SEPTEMBER 2009

FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 89 NOTTAWASAGA RIVER BRIDGE REHABILITATION/WIDENING & RETAINING WALL AND CUT SLOPE AT INTERSECTION OF ESSA 5TH LINE AND HIGHWAY 89
SIMCOE COUNTY, ONTARIO
G.W.P.2503-04-00

Submitted to:
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REPORT

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FOUNDATION INVESTIGATION REPORT
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AND CUT SLOPE AT THE INTERSECTION OF
ESSA 5TH LINE AND HIGHWAY 89
SIMCOE COUNTY, ONTARIO
G.W.P. 2503-04-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the reconstruction/rehabilitation of Highway 89 from Rosemont to 0.9 km east of County Road 13 and at the Nottawasaga River Bridge (MTO Structure Site No. 30-250) in Dufferin and Simcoe Counties. Foundation engineering services are required for the widening of the Nottawasaga River Bridge, construction of a new retaining wall to the northwest of the widened bridge structure, replacement of an existing concrete box culvert east of County Road 13 (Culvert 30-545C), and for the proposed cut slope at the intersection of Essa 5^{th} Line and Highway 89.

This report addresses the foundation investigations carried out for the Nottawasaga River Bridge widening and proposed retaining wall and for the proposed cut slope at the intersection of Essa 5^{th} Line and Highway 89.

The terms of reference and scope of work for the foundation investigation are outlined in: MTO's Request for Proposal for Agreement No. 2004-E-0032, issued in April 2005, and in Section 6.8 of MRC's *Technical Proposal* for G.W.P. 2503-04-00; Golder's proposal letters, dated January 22, 2007 for additional foundation engineering services relating to the proposed retaining wall and culvert replacement; January 28, 2008 for additional foundation engineering services relating to the proposed cut slope at the intersection of Essa 5^{th} Line and Highway 89; and August 15, 2008 for a supplementary foundation investigation at the location of the proposed Nottawasaga River Bridge pier widening.

2.0 SITE DESCRIPTION

The Nottawasaga River Bridge is located along Highway 89 between Kindlers Road and Essa Fifth Line in Simcoe County, Ontario. The existing bridge structure is a three-span concrete slab on steel girders, currently proposed to be widened to the north.

The existing Highway 89 profile varies between approximately Elevation 210 m and Elevation 210.8 m at the bridge approaches. The Highway 89 embankments are about 4 m to 6 m high relative to the existing centerline. Ground cover vegetation within the vicinity of the existing bridge consists primarily of grasses, with some small shrubs and trees.

According to preliminary General Arrangement Drawing No. P1 entitled "Highway 89 Nottawasaga River Bridge Rehabilitation- Preliminary General Arrangement", dated January 2007 and a drawing entitled "Foundation Layout", dated April 1960 by Laughlin, Wyllie & Ufnal, provided by MRC, the riverbed is at approximately Elevation 201.8 m and the water level as indicated on the above noted Drawing No. P1 was at about Elevation 205 m on January 14, 2006. The surface of the riverbed measured in boreholes advanced within the Nottawasaga River during the current field investigation, ranges from about Elevation 202.8 m to Elevation 203.7 m; the water level was measured at about Elevation 204.6 m on November 12, 2008.

The site of the proposed embankment cut slope is located on the north side of the intersection of Essa 5^{th} Line and Highway 89, about 120 m east of the Nottawasaga River Bridge. Highway 89 in this area is approximately 7.5 m wide consisting of two lanes with approximately 3 m wide shoulders on both sides of the highway. The existing slope is approximately 8 m to 10 m high, inclined on average at about 2.2H: 1V, and is generally covered with grasses and small shrubs.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Nottawasaga River Bridge site between July 3 and 12, 2007, at which time six boreholes (Boreholes 07-1 to 07-6) were advanced at the site using a track-mounted drill rig and portable drilling equipment, supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. A supplementary subsurface investigation was subsequently carried out in the Nottawasaga River at the locations of the proposed east and west pier widening between November 11 and 14, 2008 during which time a total of four boreholes (Boreholes 08-1 to 08-4) were advanced using a D-25 drill rig on a drilling platform supported on a barge, provided and operated by Walker Drilling Limited. The borehole locations are shown on Drawing 1.

An additional subsurface investigation was carried out at the site of the proposed cut slope on April 9, 2009, at which time two boreholes (Boreholes 09-1 and 09-2) were advanced using a track-mounted D-50 drill rig equipped with hollow stem augers, supplied and operated by Walker Drilling Limited. These borehole locations are shown on Drawing 3.

Boreholes 07-1 to 07-3, 07-5 and 07-6 were advanced using hollow stem or solid stem augers, to depths ranging from 6.4 m to 18.7 m below the existing ground surface. Borehole 07-4 was advanced to a depth of 7.9 m by NQ wash boring methods using portable drilling equipment. Boreholes 08-1 and 08-2 were advanced by wash boring methods to depths of 10.9 m and 8.5 m below the surface of the river bed, respectively and Boreholes 08-3 and 08-4 were advanced through the existing pier footings using BQ size coring equipment to depths of about 3 m and 3.3 m below the surface of the river bed, respectively. A starter casing was installed from the working deck of the platform to the river bed and/or top of the existing pier footings to permit re-entry of drilling and sampling equipment into the boreholes. After the starter casing was set firmly, the top of starter casing was used as a reference point for measuring the depths of the boreholes and soil sampling levels. The cuttings and returning water from the boreholes advanced in the Nottawasaga River were contained on the barge in a sedimentation container to prevent any spillage into the river. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure.

The water level in the open boreholes was observed throughout the drilling operations, and piezometers were installed in Boreholes 07-3 and Borehole 07-5 to permit monitoring of the groundwater level at the site. The piezometers consist of 50 mm diameter PVC pipe with a 1.5 m long slotted screen installed within a 3 m long sand filter pack. Upon completion of drilling, all boreholes not instrumented with a piezometer were backfilled to the ground surface using bentonite pellets, in accordance with the requirements of O.Reg. 903. Boreholes 08-3 and 08-4 were backfilled from the bottom of the borehole to the top of footing using cement grout. Borehole soil and concrete cuttings were disposed away from the river.

Boreholes 09-1 and 09-2 were advanced adjacent to the crest and the toe, respectively, of the existing embankment slope to be cut-back along Highway 89 between approximately Station 16+300 and Station 16+400. Borehole 09-1 was advanced to a depth of 15.9 m below the ground surface (i.e. Elevation 205.1 m) and Borehole 09-2 was advanced to a depth of 5.2 m below the ground surface (i.e. Elevation 206.5 m). Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m intervals of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. The groundwater conditions were observed in the open boreholes during drilling and a monitoring well comprised of 50 mm diameter Schedule 40 PVC pipe was installed in Borehole 09-1. The annulus between the borehole wall and the piezometer pipe above the screen in Borehole 09-1 and Borehole 09-2 was backfilled to the ground surface with bentonite in accordance with MOE Regulation 903.

All of the field work was monitored on a full time basis by members of Golder's technical staff who located the boreholes in the field, directed the sampling, 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 geotechnical laboratory testing. Index and classification tests, consisting of water content determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. Organic content tests were also carried out on selected samples.

The elevations of Boreholes 07-1 to 07-6 were measured in the field by members of Golder's technical staff, relative to a geodetic bench mark (BM No 00819798409) established on the south-east wing wall of the bridge and the borehole locations were measured relative to site features. The location of each of Boreholes 08-1 to 08-4 was measured relative to the edges of the existing piers and recorded by GPS; the collar elevation of each borehole was surveyed relative to temporary bench marks located on the shores of the river and correlated to the geodetic bench mark BM No. 00819798409.

The locations of Boreholes 09-1 and 09-2 were measured relative to existing site features and the ground surface elevation was surveyed relative to two temporary bench marks located under the fence line on the crest of the slope (i.e. TBM1) and at the end of the concrete curb besides the shoulder of Highway 89 (i.e. TBM2). The Geodetic elevations for these two TBMs (i.e. Elevation 211.66 m for TBM1 and Elevation 220.93 m for TBM2) were provided by MRC on April 3, 2009.

The borehole locations (including MTM NAD83 northing and easting coordinates), and ground surface or river bed elevation (referenced to geodetic datum) are summarized below and are shown on Drawing 1 and Drawing 2 for boreholes advanced at the Nottawasaga River Bridge Site and shown on Drawing 3 for boreholes advanced at the site of the proposed embankment cut-back.

Borehole Number	Borehole Locations	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface/ River Bed Elevation (m)
07-1	Retaining Wall	4891904.8	280029.5	205.6
07-2	Retaining Wall / West Approach	4891914.8	280057.6	205.4
07-3	Retaining Wall / West Abutment	4891920.8	280085.4	206.4
07-4	West Pier	4891920.5	280099.9	205.4
07-5	East Pier/East Abutment	4891937.5	280151.9	210.1
07-6	East Approach	4891943.4	280171.3	210.5
08-1	Proposed West Pier Widening	4891924.3	280108.8	203.7
08-2	Proposed East Pier Widening	4891931.6	280133.4	203.1
08-3	North End of Existing West Pier	4891920.8	280113.2	202.8
08-4	North End of Existing East Pier	4891928.4	280137.6	203.3
09-1	Crest of the existing north embankment slope	4892001.4	280261.4	220.9
09-2	Adjacent to toe of the existing slope	4891968.8	280257.3	211.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The area of the Highway 89 Nottawasaga River Bridge and proposed cut slope lies within the Simcoe Lowlands physiographic region, as delineated in *The Physiography of Southern Ontario* [1]

The Simcoe Lowlands comprise the lowlands bordering Georgian Bay and Lake Simcoe. To the west are plains lying between Elevation 176 m and Elevation 228 m, draining into Nottawasaga Bay by way of the Nottawasaga River and are referred to as the Nottawasaga Basin. To the east are the lowlands surrounding Lake Simcoe lying between Elevation 219 m and Elevation 259 m and are referred to as the Lake Simcoe Basin.

Within the Nottawasaga Basin in the Alliston area are the Essa Flats where the Nottawasaga River Bridge site is located. Most of the Nottawasaga Basin was at one time part of the floor of Lake Algonquin and its surface beds are of deltaic and lacustrine origin. The Essa Flats portion of the Basin comprises a sandy loam soil.

4.2 Subsurface Conditions

A total of ten boreholes (Boreholes 07-1 to 07-6 and 08-1 to 08-4) were advanced at the site of the Nottawasaga River Bridge widening and proposed retaining wall as shown on Drawing 1. Five boreholes (Boreholes 07-2 to 07-6) were drilled in the vicinity of the existing abutments and approach embankments, four boreholes (Boreholes 08-1 to 08-4) were advanced in the Nottawasaga River within the footprint of the proposed east and west bridge pier widening, and another borehole (Borehole 07-1) was drilled near the west end of the proposed 45 m long retaining wall to the northwest of the bridge structure.

Two additional boreholes (Boreholes 09-1 and 09-2) were advanced at the site of the proposed cut slope at the intersection of Essa 5th Line and Highway 89.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are presented on the Record of Borehole sheets and on Figures 1 to 15. A stratigraphic profile and cross-sections for the Nottawasaga River Bridge site are shown on Drawings 1 and 2. These cross-sections include borehole information from the 1959 subsurface investigation for the existing bridge structure (Appendix A). A stratigraphic cross-section for the cut slope site is shown on Drawing 3. The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile and cross sections are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the native subsoils underlying topsoil and/or fill materials at the site of the Nottawasaga River Bridge approach / abutment widening and retaining wall (Boreholes 07-1 to 07-6) consists of: (1) a deposit of sand and silt to silty sand containing various amounts of organic material, encountered in all boreholes except in Borehole 07-6 at the east end of the site. This deposit extends to between Elevation 204.5 m and Elevation 205.5 m and has a compact to very dense relative density except for the portion of the deposit containing organics, which typically has a very loose to loose relative density; (2) a deposit of sand underlying the sand and silt to silty sand layer, encountered in Boreholes 07-2 to 07-4 (west side of the Nottawasaga River). This deposit extends between Elevation 201.9 m and Elevation 197.3 m and is typically of a dense to very dense relative density. A thin layer of sand and gravel was encountered above the sand deposit in Borehole 07-2; and (3) a silt to clayey silt deposit, encountered in Boreholes 07-5 and 07-6 (east side of the Nottawasaga River) at Elevation 203 m and 206.8 m, respectively, and at Elevation 188 m in Borehole 07-2 (west side of the river).

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

The subsurface deposits within the footprints of the proposed bridge pier widening at Boreholes 08-1 and 08-2 generally consist of loose to very dense sand and gravel and very dense sand overlying hard clayey silt. Boreholes 08-3 and 08-4, advanced at the location of the existing bridge pier footings, penetrated the concrete footings and encountered approximately 0.6 m of silty sand fill at the river bed. The existing western pier is founded on a very dense sand deposit and the existing east pier is founded on a hard clayey silt deposit.

The 1959 subsurface investigation included the drilling of six (6) boreholes at the then proposed abutments and central pier locations (refer to Drawing 1 for the approximate borehole locations). The subsurface conditions encountered in these boreholes are presented on the borehole record sheets included in Appendix A and consist of a surficial layer of loose fine silty sand with organic matter underlain by very dense clay silt described as a glacial till deposit. A layer of sand and gravel was occasionally found between the loose sand and the very dense clay silt till

The subsurface conditions of the existing slope at the intersection of Essa 5th Line and Highway 89 (i.e. Borehole 09-1) consist of a thin layer of topsoil underlain by about 2 m of loose sandy silt and about 2.4 m of compact sandy silt, underlain by strata of very stiff silty clay, dense to very dense silty sand and very dense silt and sand. Borehole 09-2 advanced near the toe of the slope encountered a surficial layer of fill materials underlain by strata of firm silty clay, non plastic silts and silt and sand.

A more detailed description of the subsurface conditions encountered in the boreholes during the July 2007, November 2008, and April 2009 investigations, is provided in the following sections.

4.2.1 Subsurface Conditions at the Bridge Approach/Abutment Widening, and Retaining Wall

4.2.1.1 Topsoil

A layer of topsoil was encountered at the ground surface in all of the boreholes except in Borehole 07-6. The layer of topsoil ranges in thickness from 100 mm to 600 mm and is essentially loose in relative density.

4.2.1.2 Fill

Fill materials were encountered in Boreholes 07-1, 07-3, 07-5, and 07-6 immediately underlying the topsoil or at the ground surface. The fill is comprised of sand and silt to silty sand and is approximately 0.7 m and 0.9 m thick in Boreholes 07-1 and 07-3 on the west side of the Nottawasaga River, and 3.7 m and 5.4 m thick in Boreholes 07-5 and 07-6 on the east side of the River.

The measured SPT “N” values within the fill materials range from 4 to 16 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The results of a grain size distribution test carried out on a sample of the fill, as shown on Figure 1, indicate that the material is comprised of sand and silt, some gravel. The measured water contents on samples of the fill materials vary between 6 and 14 percent.

4.2.1.3 Sand and Silt to Silty Sand

A deposit of sand and silt to silty sand was encountered below the topsoil and/or fill materials in all of the boreholes except in Borehole 07-6 at the east end of the site. This deposit was encountered between Elevation 204.5 m and Elevation 205.5 m and varies in thickness between about 1.5 m and 8 m.

The sand and silt to silty sand deposit was found to contain variable amounts of organic matter and therefore, six organic content tests were carried out on selected samples of this deposit chosen based on visual and olfactory indication of organics. The highest organic contents were measured on samples collected in Boreholes 07-3 and 07-4 near the west bank of the River, between Elevation 205 m and Elevation 200 m, ranging from 2.6 percent to 10.1 percent.

The measured water contents on samples of this deposit range between 13 and 64 percent. Measured SPT “N” values within the sand and silt to silty sand range from 26 to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense relative density; however within the portion of this deposit containing organic matter, measured SPT “N” values range from 1 to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The results of five grain size distribution tests carried out on selected samples of this deposit, shown on Figure 2, indicate that the material grades from sand and silt to silty sand. An Atterberg limit test on a sample of the silty sand containing organics indicates that this material is non-plastic, confirming the non-cohesive composition of the material.

4.2.1.4 Sand

A deposit of sand was encountered in Boreholes 07-2, 07-3, and 07-4 underlying the sand and silt to silty sand. The surface of the sand deposit was found between Elevations 197.3 m and 201.9 m in these boreholes and extended for a thickness of 13.9 m to Elevation 188 m in Borehole 07-2. Boreholes 07-3 and 07-4 were terminated within the sand deposit at depths of 11 m and 8 m below the ground surface, having extended into the sand deposit for a thickness of 1.8 m and 0.9 m, respectively. A 400 mm thick layer of sand and gravel was encountered in Borehole 07-2 immediately overlying the sand deposit.

Measured SPT “N” values within the sand deposit range from 25 blows per 0.3 m of penetration to 100 blows for 0.1 m of penetration, indicating a compact to very dense relative density.

The result of three grain size distribution tests carried out on selected samples of this deposit are shown on Figure 3 and indicate that this deposit consists of sand, trace gravel and silt. The measured water contents on samples of the sand deposit were between approximately 16 percent and 24 percent.

4.2.1.5 Silt to Clayey Silt

A silt to clayey silt deposit was encountered underlying the sand deposit in Borehole 07-2 at Elevation 188 m, and in Boreholes 07-5 and 07-6 at Elevation 203 m and Elevation 206.8 m respectively. All three boreholes were terminated within this deposit.

The measured SPT “N” values range from 43 blows per 0.3 m of penetration to 100 blows for 0.1 m of penetration, indicating a hard consistency to very dense relative density.

The results of two grain size distribution tests carried out on selected samples of the clayey silt deposit are shown on Figure 4. Atterberg limits tests carried out on three samples of this deposit, yielded plastic limits of 15 to 16 percent, liquid limits of 21 to 24 percent, and corresponding plasticity indices of 5 to 8 percent for the clayey silt portion of this deposit, whereas yielded a non-plastic result for the silt, trace clay portion of the deposit. The results, plotted on Figure 5, confirm that this deposit is a silt to clayey silt of low plasticity. The measured water contents on samples of this deposit range between approximately 20 percent and 24 percent.

4.2.2 Subsurface Conditions at the Bridge Pier Widening

4.2.2.1 Fill

Fill materials, or river bottom sediments, were encountered in Borehole 08-3 at the west pier footing at Elevation 202.8 m and in Borehole 08-4 at the east pier footing at Elevation 203.3 m, immediately beneath the surface of the river bed overlying the existing bridge pier footings. The fill is comprised of silty sand and extends to a depth of approximately 0.6 m below river bed in both boreholes.

4.2.2.2 Concrete

In Boreholes 08-3 and 08-4, the surface of the concrete at the northern edge of the existing west and east pier footings was encountered at depths of 0.6 m and 0.7 m below the existing river bed, respectively, corresponding to Elevation 202.2 m and Elevation 202.6 m. The thickness of the concrete footing of the existing west pier is 1.7 m and extends to a depth of 2.3 m below the existing river bed in Borehole 08-3; the thickness of the concrete footing of the existing east pier is 1.5 m and extends to a depth of 2.1 m below the existing river bed in Borehole 08-4.

4.2.2.3 Sand and Gravel

A deposit of sand and gravel was encountered immediately underlying the river bed at Boreholes 08-1 and 08-2, corresponding to Elevation 203.7 m and Elevation 203.1 m, respectively. The thickness of this deposit is 3.8 m and 4 m in Boreholes 08-1 and 08-2, respectively.

Measured SPT “N” values within the sand and gravel deposit range from 4 blows per 0.3 m of penetration to 60 blows per 0.15 m of penetration, indicating a loose to very dense relative density. The lower “N” values were encountered immediately below the existing river bed and the sand and gravel deposit is typically compact to very dense in relative density.

The sand and gravel deposit was found to contain trace silt and clay and occasional cobble. Wood fragments were encountered in the near surface portion of the deposit at the location of Borehole 08-1. The results of two grain size distribution tests carried out on selected samples of this deposit are shown on Figure 6. The measured water contents on samples selected in this deposit range between 7 percent and 17 percent.

4.2.2.4 Sand

A deposit of sand was encountered underlying the sand and gravel deposit in Borehole 08-1 and immediately below the bottom of the existing west bridge pier footing at the location of Borehole 08-3, at Elevation 199.8 m and Elevation 200.5 m, respectively. In Borehole 08-1, the sand deposit is 5.4 m thick and extends to Elevation 194.5 m. Borehole 08-3 was terminated within this sand deposit at a depth of 2.9 m below the existing river bed.

Measured SPT “N” values within the sand deposit range from 72 blows per 0.3 m of penetration to 85 blows for 0.15 m of penetration, indicating a very dense relative density.

The results of one grain size distribution test carried out on selected sample of this deposit are shown on Figure 7. The measured water content of the sand deposit is about 18 percent.

4.2.2.5 Clayey Silt

A deposit of clayey silt was encountered beneath the deposits of sand and sand and gravel in Borehole 08-1 at Elevation 194.5 m, and in Borehole 08-2 at Elevation 199.1 m, as well as under the concrete footing of the east pier in Borehole 08-4 at Elevation 201.2 m. These three boreholes terminated within the clayey silt deposit at Elevation 200 m to 192.8 m.

The measured SPT “N” values range from 57 blows per 0.3 m of penetration to 100 blows for 0.1 m of penetration, indicating a hard consistency.

The results of a grain size distribution test carried out on a selected sample of the clayey silt deposit are shown on Figure 8. The measured water contents on selected samples of this deposit range between approximately 18 percent and 23 percent. The results of Atterberg limits tests carried out on three samples of this deposit are shown on Figure 9 and indicate liquid limits ranging from about 22 percent to 23 percent, plastic limits ranging from about 16 percent to 18 percent, and corresponding plasticity indices ranging from about 5 percent to 6 percent. These results indicate that the deposit consists of clayey silt of low plasticity.

4.2.3 Subsurface Conditions at the Proposed Cut Slope at Essa 5th Line and Highway 89

4.2.3.1 Topsoil

A 0.3 m thick deposit of topsoil was encountered at the ground surface in Borehole 09-1.

4.2.3.2 Fill

Fill consisting of silty sand, some gravel, trace organic matter was encountered in Borehole 09-2 at the ground surface and extends to a depth of 0.8 m below ground surface (i.e.Elevation 210.9 m).

4.2.3.3 Sandy Silt

A deposit of sandy silt was encountered immediately below the topsoil at the location of Borehole 09-1 and extends to a depth of 4.7 m below ground surface (i.e. Elevation 216.2 m). The thickness of this deposit is 4.4 m.

Measured SPT “N” values within the sandy silt deposit range from 4 blows to 23 blows per 0.3 m of penetration, indicating a loose to compact relative density. The lower “N” values were encountered within the upper 2 m of the deposit below the existing ground surface.

The deposit was found to contain trace to some clay. The results of one grain size distribution test carried out on a selected sample of this deposit are shown on Figure 10. The measured water contents on five samples selected in this deposit range between 15 percent and 18 percent.

4.2.3.4 Silty Clay and Clayey Silt

Deposits of silty clay and clayey silt were encountered underlying the deposits of sandy silt in Borehole 09-1 and underlying the fill in Borehole 09-2, at Elevation 216.2 m and Elevation 210.9 m, respectively. In Borehole 09-1, the silty clay deposit extends to a depth of 5.6 m below ground surface (i.e. Elevation 215.3 m). In Borehole 09-2, the clayey silt deposit extends to a depth of 1.4 m below ground surface (i.e. Elevation 210.3 m). The thickness of this deposit is 0.9 m in Borehole 09-1 and 0.6 m in Borehole 09-2.

A measured SPT “N” value within the silty clay deposit was 16 blows per 0.3 m of penetration indicating a very stiff consistency. An SPT “N” value of 4 blows per 0.3 m of penetration was measured within the clayey silt deposit in Borehole 09-2, indicating a firm consistency.

The results of a grain size distribution test carried out on a selected sample of the silty clay deposit are shown on Figure 11. Measured water contents on a selected sample of the silty clay and clayey silt deposit were about 23 percent. The results of two Atterberg limits tests carried out on samples of silty clay and clayey silt deposits are shown on Figure 12. The liquid limits are about 32 percent and 37 percent, the plastic limits are about 16 percent and 20 percent, and the corresponding plasticity indices are about 16 percent and 17 percent. These results indicate that these deposits consist of clayey silt and silty clay of low to intermediate plasticity.

4.2.3.5 Silty Sand

A deposit of silty sand was encountered underlying the silty clay deposit in Borehole 09-1 at Elevation 215.3 m. The silty sand deposit is 5.7 m thick and extends to Elevation 209.6 m.

Measured SPT “N” values within the silty sand deposit range from 43 blows per 0.3 m of penetration to 113 blows for 0.3 m of penetration, indicating a dense to very dense relative density.

The results of one grain size distribution test carried out on a selected sample of this deposit are shown on Figure 13. The measured water contents of the silty sand deposit range from about 6 percent to 14 percent.

4.2.3.6 Silt and Sand to Silt

Deposits of silt and sand to silt were encountered underlying the silty sand deposit at depths of 11.3 m below ground surface in Borehole 09-1 and 1.4 m below ground surface in Borehole 09-2. Borehole 09-1 was terminated within the silt and sand deposit at a depth of 15.9 m below the existing ground surface (i.e. Elevation 205.1 m). Borehole 09-2 was terminated within a silt deposit at a depth of 5.2 m below the existing ground surface (i.e. Elevation 206.5 m).

Measured SPT “N” values within these deposits range from 20 blows per 0.3 m of penetration to 128 blows for 0.3 m of penetration, indicating a compact to very dense relative density.

The silt and sand deposit was found to contain trace clay. The silt deposit was found to contain some clay. The results of three grain size distribution tests carried out on selected samples of these deposits are shown on Figure 14 and Figure 15. The measured water contents of the silt and sand deposit range from about 13 percent to 16 percent, and the measured water contents of the silt deposits range from about 16 percent to 19 percent. Atterberg limits tests carried out on two samples of the silt deposit indicate that the silt is non-plastic.

4.2.4 Groundwater Conditions

The water level observed in the open Boreholes 07-1 to 07-6 upon completion of drilling ranged from 0.8 m to 1.5 m below ground surface (Elevation 204.6 m to Elevation 204.1 m) except in Borehole 07-6 which was dry upon completion of drilling. Piezometers were installed in Boreholes 07-3 and 07-5 to monitor the groundwater level at the site and the water level measurements taken in these piezometers are summarized below:

Borehole Number	Ground Surface Elevation	Measured Groundwater Elevation		
		July 12, 2007	July 31, 2007	August 29, 2007
07-3	206.4	204.8	204.7	204.8
07-5	210.1	205.3	205.4	205.3

The water levels observed in the open Boreholes 08-1 to 08-4 upon completion of drilling were the same as the river level (i.e., Elevation 204.6 m to Elevation 204.7 m).

At the site of the proposed cut slope, groundwater seepage was observed at a depth of 1.5 m below ground surface during drilling of Boreholes 09-1 and 09-2. The groundwater level in the monitoring well installed in Borehole 09-1 was measured on May 8, 2009 at a depth of 10 m below the present ground surface (i.e. Elevation 210.9 m). The water level observed in the open Borehole 09-2 upon completion of drilling was at a depth of 2.1 m below ground surface (i.e. Elevation 209.6 m).

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. Groundwater levels may be influenced by the water level in the adjacent Nottawasaga River and perched groundwater conditions should be anticipated within the surficial silty sand and sand and silt deposits in Boreholes 07-1 to 07-6, especially during the wetter months of the year. It is further noted that the river level is also expected to fluctuate seasonally.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Sen Hu, and reviewed by Ms. Houda Jadi, Ph.D., P.Eng., a Geotechnical Engineer with Golder. Mr. Jorge Costa, P.Eng, a Designated MTO Contact and Principal with Golder conducted an independent review and provided quality control of the report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 89 NOTTAWASAGA RIVER BRIDGE
REHABILITATION/WIDENING & RETAINING WALL
AND CUT SLOPE AT INTERSECTION OF
ESSA 5TH LINE AND HIGHWAY 89
SIMCOE COUNTY, ONTARIO
G.W.P. 2503-04-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed widening of the Highway 89 Bridge over the Nottawasaga River and associated retaining wall to the northwest of the proposed bridge widening as well as geotechnical recommendations for the design of the proposed embankment cut slope at the intersection of Essa 5th Line and Highway 89, in Simcoe County. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations carried out in July 2007, November 2008, and April, 2009, as well as data obtained from the 1959 subsurface investigation for the original bridge structure, as appropriate. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the foundations for the proposed structure widening and to provide the required geotechnical information for the design of the proposed cut slope. Where comments are made on construction they are provided in order to highlight those aspects which could affect the design of the project and for which special provisions or operational constraints 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 it may affect equipment selection, proposed construction methods, scheduling, and the like.

6.2 Bridge Widening Foundation Options

The existing Nottawasaga River Bridge is a three span structure (section lengths of 18 m, 25.6 m and 18 m) consisting of a concrete slab on steel girders. According to the Preliminary General Arrangement drawing for the proposed bridge widening provided by MRC (drawing entitled “Highway 89, Nottawasaga River Bridge Rehabilitation, Preliminary General Arrangement”, dated January, 2007), the existing bridge abutments are founded on steel H-piles whereas the east and west piers are founded on spread footings enclosed within steel sheet piles. Based on the 1959 geotechnical investigation report for the design of the existing bridge structure (Report No. 59-F-98, W.P. 218-59), the east abutment and then proposed central pier were to be founded at or below Elevation 200.9 m (659 ft) and the west abutment was to be founded at or below Elevation 202 m (663 ft). Existing Department of Highways-Ontario drawings (Drawing No. D4486-2, titled “Nottawasaga River Bridge, Foundation Layout, dated April, 1960, by Laughlin, Wyllie & Ufnal) provided by MRC, indicate that the existing bottom elevation of the east pier footing is at about Elevation 201 m and that of the west pier footing is at about Elevation 200.9 m. Based on the results of the 2008 borehole investigation carried out at the locations of the existing piers and pier widening, the founding level of the existing east pier is at about Elevation 201.2 m and that of the west pier is at about Elevation 200.5 m. These founding levels are generally compatible with those shown on the Department of Highways-Ontario drawings.

The existing bridge structure is to be widened by approximately 9 m to the north to accommodate one additional future eastbound and westbound lanes and shoulders. The widened portion of Highway 89 will be maintained at approximately the existing profile grade within the limits of the structure and its immediate embankments, requiring placement of up to 4 m of new fill materials along the existing northwest embankment side slope and construction of a retaining wall to accommodate the embankment widening in proximity to the river bank. The retaining wall will be located along the northwest side of the widened approach embankment extending between approximately Station 16+085 and Station 16+130 (approximately 45 m long) with a maximum height of about 5 m. West of the proposed wall (between Station 16+020 and Station 16+085), the widened embankment will be sloped downward to the north. The existing ground surface to the northeast of the bridge structure is between about Elevation 210 m and Elevation 211 m, therefore, minor grade raise will be required for the embankment widening immediately to the northeast of the bridge. It is understood that a cut-back of the existing embankment slope between approximately Station 16+300 and Station 16+400 is required to accommodate the widening along this section of the highway.

The subsoils on the west side of the Nottawasaga River consist of an upper deposit of loose sand and silt to silty sand containing organics, found directly below topsoil and/or fill materials. The sand and silt to silty sand soils are underlain by an up to 13.9 m thick layer of sand of compact to very dense relative density. Where fully penetrated, the sand deposit was underlain by a deposit of very dense silt. On the east side of the Nottawasaga River, an up to 5.6 m thick layer of fill materials was encountered at the ground surface and is underlain by very dense silty sand and/or silt to clayey silt of hard consistency.

The surficial sand and silt to silty sand soils on the west side of the Nottawasaga River are loose and contain variable amounts of organic matter. These soils are not suitable for support of shallow foundations. Spread footings could be founded on the underlying very dense silty sand to sand below Elevation 198.5 m for the widening of the west abutment. Spread footings for the east abutment should be founded on the very dense/ hard silt to clayey silt deposit below Elevation 201 m. However, these founding elevations would involve significant excavations for the abutment foundations (7 m to 10 m in depth) which would have to be carried out in close proximity to the existing abutment pile caps, requiring extensive temporary excavation support. It is therefore recommended that the bridge abutment widening be supported on driven steel H-piles or drilled caissons founded within the “100-blow” silty sand /sand deposits or silt to clayey silt deposit. From a foundations perspective, driven steel H-piles are considered to be the most practicable option for the bridge abutments; these can be used in either a conventional or integral abutment configuration and are compatible with the existing bridge abutment foundations.

Within the footprints of the proposed widening of the pier footings, a sand and gravel deposit was encountered immediately below the river bed in Boreholes 08-1 and 08-2. The relative density of this deposit is typically compact to very dense, except for the upper 1 m of the deposit which is in a loose state. This surficial portion of the sand and gravel deposit is not suitable for support of shallow foundations. Based on Boreholes 08-3 and 08-4, the existing west bridge pier footing is founded on a very dense sand deposit at about Elevation 200.5 m, and the existing east pier footing is founded on a hard clayey silt deposit at about Elevation 201.2 m.

The bridge pier foundations for the widening of the existing structure should be consistent with the existing pier foundations, that is spread footings. The footings should be founded at the same levels as the existing footings, on the compact to very dense sand and sand and gravel deposits at Elevation 200.5 m for the west pier footing on the very dense sand and gravel and hard clayey silt deposits at Elevation 201.2 m for the east pier footing.

Alternative foundations for the proposed pier widening could consist of piles driven to refusal within the “100-blow” materials, however, pile driving operations adjacent to the existing shallow foundation could result in disturbance of the existing foundations. Therefore this alternative is not considered suitable. Micropiles drilled for a sufficient depth into the “100-blow” material could also provide a suitable foundation alternative, however a micropile foundation is significantly more costly and is therefore not addressed further in this report. If the micropile alternative is to be pursued further, based on other considerations such as potential environmental impacts or if higher geotechnical resistances are required, a site specific micropile design would need to be developed.

The advantages and disadvantages for the various bridge widening foundation options are summarized in Table 1. Recommendations for the above shallow foundations and pile foundation alternatives for the bridge widening, are provided in the following sections.

Foundation alternatives and design recommendations for the proposed retaining wall to the northwest of the bridge structure are discussed separately under Section 6.6.

6.3 Spread Footings

Spread footings are recommended for support of the west and east pier widening to be consistent with the existing pier foundations. Based on Boreholes 08-1 and 08-2, the present river bed level at the west pier widening is at about Elevation 203.7 m and that at the east pier widening is at about Elevation 203.1 m. The footings for the east and west pier widening should be founded at the same levels as the existing footings, on the compact to very dense sand and gravel deposit at Elevation 200.5 m for the west pier widening on the very dense sand and gravel and hard clayey silt deposit at Elevation 201.2 m for the east pier widening.

In order to found the footing of the west pier widening at Elevation 200.5 m, an excavation about 3.2 m deep below the river bed (about 4.1 m below the river water level) would be required. Similarly, in order to found the footing of the east pier widening at Elevation 201.2 m, an excavation about 1.9 m deep below the riverbed (about 3.4 m below the river water level) would be required. Such excavations will require water control measures in the form of a temporary sheet pile cofferdam, a tremied concrete seal, and dewatering to be able to complete construction of the footings in the dry. The sheet piling could be left in place similar to the sheet piles associated with the existing pier footings to provide scour protection as required. The sheet piles should be driven to a minimum depth of 2 m below the base of the footings to accommodate a tremie concrete plug of adequate thickness to counteract hydrostatic pressures or 2 m below the calculated scour depth, as determined by the bridge designer, whichever is deeper.

The founding level of the west bridge pier footing widening may be lowered to Elevation 199.8 m if a higher geotechnical resistance is required. In this case, subexcavation of an additional 0.7 m below the existing footing level should start about 1 m beyond the existing west pier footing followed by controlled subexcavation on a 1H: 1V slope away from and immediately adjacent to the existing footing to remove the existing compact sand and gravel material and then backfilling the subexcavated area with tremie concrete to Elevation 200.5 m. The poured tremie concrete will also function as part of the tremie concrete plug to counteract the hydrostatic pressure as described above.

The following founding elevations and geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) may be used for the design of 3 m wide spread footings placed on the native deposits:

Foundation Element	Borehole Number	Founding Soil	Maximum Founding Elevation	Factored Geotechnical Resistance at ULS	Geotechnical Resistance At SLS*
West Pier Widening	BH 08-1 and BH 08-3	Compact Sand and Gravel	200.5 m	400 kPa	300 kPa
		Very Dense Sand	199.8 m	500 kPa	350 kPa
East Pier Widening	BH 08-2 and BH 08-4	Very Dense Sand and Gravel or Hard Clayey Silt	201.2 m	500 kPa	350 kPa

* For 25 mm of settlement, assuming a 3 m wide footing.

It is noted that the ULS resistance and the magnitude of settlement are dependent on the footing founding depth, size, configuration and applied loads. If the existing pier footings are to be widened to support the new piers at a different elevation than that of the existing footings, geotechnical resistances for spread footings should be reviewed once the base elevations of the existing pier footings have been surveyed and the final geometry of the foundations has been established.

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

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC.* The coefficient of friction, tan φ', between cast-in-place concrete footings and the undisturbed sand and gravel and clayey silt may be taken as 0.45. This represents an unfactored value and in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

All footings should be provided with a minimum 1.2 m of soil cover for frost protection, however this condition may not be applicable for this site where the footings may be founded below the ice cover in the river.

6.4 Steel H-Pile Foundations

The widened abutments should be supported on steel H-piles driven to found within the "100-blow" silty sand / sand deposit for the west abutment and within the "100-blow" silt to clayey silt deposit for the east abutment, below Elevation 198.5 m and Elevation 200 m, respectively. For design, the following pile tip levels may be assumed, based on the borehole results and providing a 2 m depth of penetration into the "100-blow" materials. These tip elevations may be assumed for determining pile lengths; however, provision should be made in the contract to deal with greater pile lengths in the event that the piles penetrate deeper into the founding strata.

Foundation Element	Borehole Number	Approximate Pile Tip Elevation
West Abutment – North	07-3	196.5 m
East Abutment – North	07-5	198.0 m

In the installation of steel H-piles, consideration must be given to the possible presence of cobbles and/or boulders within the soil deposits at this site, as encountered in one of the current boreholes. The piles should be stiffened with MTO flange plates for protection during driving, in accordance with OPSS 903.07.05.04. Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing bridge structure are maintained within tolerable ranges (refer to Section 6.10.4). The pile driving criteria may have to be adjusted depending on the results of the vibration monitoring.

6.4.1 Axial Geotechnical Resistance

For HP 310x110 piles driven at least 2 m into the "100-blow" lower silty sand or sand deposit at the location of the west abutment widening or into the "100-blow" silt to clayey silt deposit at the east abutment widening location , a factored axial resistance at Ultimate Limit States (ULS) of 1400 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1000 kN.

The pile capacity must be verified in the field by the use of the Hiley formula (Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity of 2800 kN. For piles driven into the

"100-blow" silty sand/sand or silt to clayey silt deposits, the following note should be shown on the Contract drawing assuming that a resistance factor of 0.5 (in accordance with MTO Foundations requirements) is applied to the use of the Hiley:

"Piles to be driven in accordance with Standard SS-103-11 using an ultimate capacity of 2800 kN per pile."

Pile installation should be in accordance with MTO's Special Provision SP903S01. 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.

6.4.2 Downdrag Load (Negative Skin Friction)

The widened embankment loading to the northwest of the bridge will cause up to 60 mm of settlement within the upper loose sand and silt to silty sand deposit containing organics; however, this settlement is expected to occur during and immediately upon completion of the embankment construction. Some secondary compression / creep may occur due to the potential decay of the organic matter within the upper silty sand layer with time; however, this settlement is expected to be negligible due to the dispersion of the organic matter within the sand deposit at organic contents measured in the borehole samples (3 percent to 10 percent by weight). Post-construction consolidation settlement within the hard clayey silt deposit, for the east abutment under the anticipated 0.8 m to 1.8 m new embankment loading will also be negligible Therefore, downdrag loads for the abutment foundations need not be taken into consideration.

6.4.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM 1992[2] as noted in CHBDC C6.8.7.1):

For cohesionless soils:

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

where

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

For cohesive soils:

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

where

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

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. Approximate elevation intervals are given for each deposit; however, the deposit boundaries may vary at each of the foundation elements and reference should be made to the borehole records and to the interpreted stratigraphic section on Drawings 1 and 2.

[2] Canadian Geotechnical Society, 1992, Canadian Foundation Engineering Manual, 3rd Edition

Structure	Relevant Borehole	Soil Unit	n_h	s_u
West Abutment	07-3	Very loose to loose sand and silt to silty sand containing organics above Elev. 199 m	1000 kPa/m	–
		Very dense silty sand between Elev. 197 m and Elev. 199 m	15000 kPa/m	–
		Very dense sand below Elev. 197 m	15000 kPa/m	–
East Abutment	07-5	loose to compact sand and silt fill above Elev. 204.5 m	1300 kPa/m	–
		Very dense silty sand between Elev. 203 m and Elev. 204.5 m	15000 kPa/m	–
		Hard clayey silt below Elev. 203 m	–	200 kPa

A maximum factored lateral resistance of 160 kN at ULS, and a maximum lateral resistance of 65 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 C 6.8.7.1(a) of the *Commentary* to the *CHBDC*.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, 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

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

6.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.5 Caissons

Consideration could be given to the use of caissons founded within the "100-blow" silty sand / sand or silt to clayey silt deposits for support of the abutment widenings. The following design base elevations may be used at the abutments, based on approximately 2 m of embedment within the "100-blow" soils:

Foundation Element	Relevant Boreholes	Estimated Elevation Of 100-Blow Deposit	Estimated Caisson Base Elevation
West Abutment Widening	07-3	199 m	196.5 m
East Abutment Widening	07-5	200 m	198 m

Running or flowing of water-bearing cohesionless strata could occur during or after drilling of the caissons, and therefore if caisson foundations are adopted for this site, temporary or permanent caisson liners would be required to support the soils during construction and permit cleaning and inspection of the caisson base. However, construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the "100-blow" material and that such difficulty can result in "necking" of the caisson. As such, permanent liners would be preferred for the construction of the caissons in these soil conditions.

If caisson foundations are adopted for this site, an NSSP will be developed to address the need for control of the ground and groundwater during caisson construction.

6.5.1 Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on "socketting" approximately 2 m into the "100 blow" deposits. Using the design elevations given above, and assuming that all caisson excavations are properly cleaned and are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS are given below for various caisson diameters:

Caisson Diameter	Axial Geotechnical Resistance	
	ULS	SLS
0.9 m	2,300 kN	1,900 kN
1.2 m	4,200 kN	3,500 kN
1.5 m	7,500 kN	6,000 kN

If permanent liners are used for construction of the caissons, the geotechnical resistances provided above would have to be reduced to neglect the component of shaft friction over the "socket" within the 100-blow soil.

6.5.2 Downdrag Load (Negative Skin Friction)

As discussed in Section 6.4.2, post-construction settlements of the foundation soils are expected to be negligible and therefore no significant downdrag loads would be applied to the caissons.

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as detailed in Section 6.4.3

A maximum factored lateral resistance of 350 kN at ULS, and a maximum lateral resistance of 200 kN at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons, based on MTO caisson lateral load test results at Leslie Street and Highway 401 in Toronto, modified to reflect subsurface conditions at this site. Values for alternative caisson diameters can be provided if larger diameter caisson foundations are adopted at this site.

6.6 Retaining Wall Options

The subsoil conditions along the proposed retaining wall alignment, except for the west end of the wall, consist of the upper very loose to loose sand and silt to silty sand soils containing variable amounts of organic matter, generally underlain by a relatively thick layer of sand of compact to very dense relative density. The loose sand and silt/silty sand layer, containing organics is approximately 6.5 m thick immediately adjacent to the bridge structure (i.e. Borehole 07-3) and reduces in thickness westerly to about 3 m at the location of Borehole 07-2 and is absent at the location of Borehole 07-1. The retaining wall height and additional embankment fill thickness varies between about 3 m on the west end of the wall to approximately 5 m adjacent to the bridge structure. The upper loose sand and silt to silty sand soils are expected to undergo an estimated elastic settlement of up to 60 mm under the additional embankment loading during and immediately after completion of construction, with up to about 50 mm of differential settlement estimated to occur between the east and west ends of the retaining wall.

Given the above estimated total and differential settlements, shallow foundations will not be feasible on the upper loose sand and silt to silty sand soils. Therefore, the options for the retaining wall are:

- a concrete retaining wall on strip footing foundation after subexcavation of the upper loose sand and silt to silty sand soils containing organics; this option however, will require deep excavations;
- a pile-supported concrete retaining wall founded within the “100-blow” silty sand or sand deposits; or
- a retained soil system (RSS) wall which would accommodate predicted total and differential settlements along the wall during the embankment / wall backfill construction.

The following sections provide further discussion and geotechnical recommendations regarding the retaining wall foundation options outlined above. The advantages and disadvantages for the various retaining wall options are summarized in Table 2.

6.6.1 Concrete Retaining Wall on Shallow Footings

A concrete retaining wall supported on shallow foundations can be considered provided the loose silty sand /sand and silt soils containing organics are subexcavated and replaced with properly compacted granular fill in order to limit the total settlement under foundations to less than 25 mm. However, excavations up to 7 m deep at the eastern end of the wall (i.e., immediately west of the bridge structure near Borehole 07-3) and 3 m deep at the western end of the wall (i.e., between Boreholes 07-2 and 07-1 would be required.

The retaining wall footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration.

For the design of spread footings placed on properly compacted Granular “A” fill, a geotechnical resistance at Ultimate Limit States (ULS) of 450 kPa and at Serviceability Limit States (SLS) (for 25 mm of settlement) of 300 kPa may be used for a 2 m wide strip footing.

The ULS resistance and the magnitude of settlement are dependent on the footing size, configuration and applied loads. Therefore, if this option is adopted, geotechnical resistances for spread footings should be reviewed as the detail design progresses.

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

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan φ', between cast-in-place concrete footings and the properly prepared granular fill may be taken as 0.45. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.6.2 Pile-Supported Concrete Retaining Wall

Consideration could be given to the use of a concrete retaining wall supported on pile foundations that extend through the loose silty sand soils, to reduce the potential settlement / differential settlement of the concrete retaining wall during and immediately after construction of the additional embankment fill materials. Based on the information from Boreholes 07-1, 07-2, and 07-3, the piles should be driven to found within the very dense, "100-blow" lower silty sand or sand deposits. The surface of the "100-blow" soils was encountered between about Elevation 190 m and Elevation 202.5 m, as summarized in the table below. For design, the following pile tip levels may be assumed based on 2 m of penetration into the "100-blow" lower sandy deposits.

Foundation Element	Relevant Boreholes	Estimated Elevation of "100-Blow" Soil	Estimated Pile Tip Elevation
Concrete Wall -East Section	07-3	198.5 m	196.5 m
Concrete Wall- Central Section	07-2	190.0 m	188.0 m
Concrete Wall-West Section	07-1	202.5 m	200.5 m

In the installation of steel H-piles, consideration must be given to the possible presence of cobbles and/or boulders within the sand, silt/silty sand and sand and gravel layers. Steel H-piles should be stiffened with MTO flange plates for protection during driving, in accordance with OPSS 903.07.05.04.

Geotechnical design recommendations for HP 310 x 110 piles driven to within the "100 blow" lower sandy soils are provided is Sections 6.4.1 through 6.4.4.

6.6.3 Reinforced Soil System (RSS) Wall

A mechanically-reinforced soil retaining system (retained soils system or RSS wall) is also considered suitable for this site and could be an overall most cost-effective wall option. Post-construction settlement is estimated to be negligible at the site, though up to 60 mm of elastic settlement could occur during and immediately after completion of the embankment/wall backfill construction.

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The footing must be founded below any topsoil, loose fill or unsuitable native soils. A properly compacted granular pad, at least 0.6 m thick should be placed below the wall front facing footing. For an assumed width of 0.6 m for the facing footing and assuming the footing is placed on a granular levelling pad or properly prepared undisturbed subgrade, such as the dense sand and silt near the west end of the proposed wall, a factored geotechnical resistance at ULS of 100 kPa may be used for design of the facing footing. The facing footing should be founded 1.2 m below final adjacent ground surface for protection against frost penetration.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, the factored geotechnical resistances at ULS below may be used for assessment of the reinforced mass founded on the properly prepared embankment fill materials (or on the properly prepared native sand and silt deposit if it becomes exposed during excavation of the existing embankment slope for construction of the lower portion of the wall).

Wall Height	Assumed Reinforced Width	Factored Geotechnical Resistance at ULS
2.5 m	1.7 m	150 kPa
5.0 m	3.3 m	200 kPa

The resistance to lateral forces/sliding resistance between the compacted backfill 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 fills of the RSS wall and the properly prepared subgrade may be taken as 0.6. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The global stability of the RSS wall is discussed in Section 6.8.2 in association with the approach embankment stability.

6.7 Lateral Earth Pressure for Design

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

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

- Select free draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 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. This fill should be compacted in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in 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 in Figure C6.20(b) of the *Commentary* to the *CHBDC)*.

- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for the new portions of the approach embankments:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, k_a	0.35
At rest, k_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) 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	0.27	0.27
At rest, k_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the abutment stem and retaining wall, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design.

6.7.1 Seismic Considerations

Seismic (earthquake) loading must be considered in the design of the abutment stems and retaining walls in accordance with Section 4.6 of CHBDC as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and 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, which is to be added to the static earth 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:

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

Where K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);

K_{AE} = the seismic active earth pressure coefficient determined in accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*;

γ' = the effective unit weight of the soil (kN/m^3)

- taken as the soil unit weight given above for the fill materials;

- taken as 19 kN/m^3 for the loose native deposits and 21 kN/ m^3 for the very dense / hard deposits
- taken as 20 kN/ m^3 for the existing fill, where encountered;

d = the depth below the top of the wall (m); and

H = the height of the wall (m).

The peak zonal acceleration used for this site is 0.065 g, which is based on a zonal acceleration of 0.05 g (based on a zonal acceleration ratio of 0.05 for Alliston, Ontario, CHBDC, 2001) multiplied by an amplification factor of 1.3 for the types of soils found at the site (in accordance with Section 4.1.8.4 and Table 4.1.8.4 B of the NBC (2006) for Class D soils and 5 % percent damped spectral acceleration Sa (0.2) less than or equal to 0.25). Using the amplified zonal acceleration ratio of 0.065g, the seismic lateral earth pressure coefficients (KAE) for both yielding and non-yielding walls considering earth or granular fills were determined in accordance with Sections 4.6.4 and C 4.6.4 of the CHBDC and its *Commentary* and are presented in the table below. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	**Granular A**	**Granular B Type II**
Yielding wall [1]	0.30	0.26	0.26
Non-yielding wall	0.34	0.30	0.30

[1] 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 0.065.

6.8 Approach Embankments

The widened portion of Highway 89 on the east side of the Nottawasaga River Bridge site will require a grade raise between approximately 0.8 m to 1.8 m, with embankment earth fill side slopes of 2Horizontal:1Vertical. To the west of the bridge structure however, up to 5 m of additional embankment fill will be placed atop the existing embankment north side slope to accommodate the approximately 9 m widening to the north.

6.8.1 Subgrade Preparation and Embankment Construction

In order to minimize differential settlement between the existing and widened portions of the approach embankments, it is recommended that all topsoil and softened / loosened soils and soils containing significant amounts of organics, be stripped from the existing embankment side slopes below the widening areas. All subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment fill should be placed and compacted in accordance with MTO's Special Provision 105S10.

Additionally, to minimize differential settlement between the widened portions of the approach embankments due to settlement of the fill itself, the use of granular fill is recommended rather than the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

To reduce the potential for erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankment.

6.8.2 Approach Embankment Stability

Due to the limited footprint for widening adjacent to the Nottawasaga River bank, immediately west of the bridge structure, an approximately 45 m long retaining wall will be constructed along the north side of the widened embankment between approximately Station 16+085 and Station 16+130 (reference: MRC's drawing entitled "Option 2 – Widening to the North", dated April 26, 2007). From about Station 16+020 to Station 16+085, the widened west approach embankment will be sloped downward to the north to meet the surrounding grades.

Slope stability analyses for the new section of the west embankment side slopes between Station 16+020 and Station 16+085, and global stability analyses for the proposed retaining wall between Stations 16+085 and 16+130, have been carried out using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. Effective stress parameters were employed in the analyses assuming drained conditions for the soils. The effective angle of friction for these soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests, in conjunction with engineering judgement considering experience in similar soil conditions. The following parameters have been used:

Soil Type	Unit Weight (kN/m^3)	Effective Angle of Friction
Embankment fill (granular fill assumed)	20	30°
Surficial sand and silt to silty sand , containing organics (very loose to loose)	19	30°
Silty sand or sand and silt (very dense)	20	32°
Sand (very dense)	21	35°

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the approximately 3 m to 5 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety greater than 1.3 against deep-seated slope instability under static and seismic conditions, as shown on Figures B1 and B2, respectively, in Appendix B.

The global stability of the proposed retaining wall for the cross section configuration at about Station 16+120 has been analyzed assuming both a concrete retaining wall and an RSS wall. Based on the results of the analyses, the factor of safety against global instability of the concrete retaining wall and the RSS wall (assuming that the reinforcing strips have a length of at least two-thirds of the height of the wall) is greater than 1.5 under static conditions and greater than 1.4 under seismic conditions, as shown on Figures B3 through B6, respectively, in Appendix B.

6.8.3 Approach Embankment Settlement

Settlement of the approximately 3 m to 6.5 m thick deposit of very loose to loose sand and silt to silty sand containing organics, will occur as a result of the widening of the existing northwest approach embankment. To estimate the magnitude of settlement, analyses were carried out using the commercially-available computer program *Unisettle* as well as hand calculations, assuming the use of conventional earth or granular fill. The settlement of the founding soils has been estimated using the elastic deformation moduli given below, based on correlations with the relevant SPT "N" values (Bowles, 1984):

Soil Unit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Embankment fill (parameters assumed for granular fill)	20	–
Surficial sand and silt to silty sand with organics (very loose to loose)	19	5
Silty sand or sand and silt (very dense)	20	30
Sand (very dense)	21	50

Based on placement of a "wedge" of fill, having a maximum thickness of 5 m, on the existing embankment side slopes, the maximum settlement estimated below the widening footprint and outside portion of the existing embankment, assuming the use of conventional earth or granular embankment fill is 60 mm. This settlement is expected to occur during and immediately upon completion of construction as it is chiefly attributed to the elastic deformation of the existing upper loose silty sand/sand and silt soils.

6.9 Liquefaction Potential and Seismic Analysis

The liquefaction susceptibility of the soil deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions for the Nottawasaga River bridge site was assessed using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary* (2001) based on papers by Seed and Idriss (1971) and Seed et al. (1984), which compares the cyclic resistance ratio (CRR) of the soils to the cyclic stress ratio (CSR) caused by an earthquake. If liquefaction of the subsoils under embankment loading is not anticipated, the stability of the embankment slope may be assessed using conventional pseudo-static methods of slope stability analysis under earthquake-induced peak ground acceleration. Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual shear strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT 'N' values. For free-draining soils, the seismic loading is applied to the long-term (drained) conditions.

Using the methods outlined in above and using the amplified peak ground acceleration value for this site of 0.065g (refer to Section 6.7.1), the soils at this site have a low risk of liquefaction. This assessment corresponds to a characteristic earthquake of magnitude 7 representing approximately 10 to 15 effective cycles of loading and has been established based on historical earthquake data and de-aggregation of seismic risk carried out for other projects in the general region, and taking into consideration that smaller magnitude events (i.e. ≤ M5) do not contribute to liquefaction damage.

A factor of safety greater than 1.0 against embankment instability under seismic conditions is obtained with an earthquake-induced peak ground acceleration equal to 0.065g using the commercially available program SLOPE/W (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The results of the embankment slope stability analyses are shown on Figures B1 through B6 in Appendix B.

6.10 Construction Considerations

6.10.1 Excavation

The excavation for the east abutment pile cap will extend mainly through the existing embankment fill, whereas excavations for the west abutment pile cap and for the retaining wall foundations will extend through or into the surficial deposit of sand and silt to silty sand containing organics. 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 and the upper sand and silt soils are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

At the pier locations, a sheet pile cofferdam and dewatering would be required to allow construction of the pier footings in the dry. A tremie concrete seal will be required at the base of the excavation to allow for dewatering within the sheet pile cofferdam for construction of the reinforced concrete footings. The Contractor should be responsible for determining the actual length of the sheetpiles required for internal stability of the cofferdam; however, if the sheetpiles are to be kept in place after construction for scour protection, they should be driven to a minimum depth of 2 m below the footings base elevations or below the scour depth as determined by the bridge designer. In this regard, it should be noted that the depth to which the existing sheet pile wall cofferdam was driven is not known, however, it is considered that sheet piles may be difficult to drive within the very dense / hard "100-blow" strata encountered below Elevation 200.5 m at the east pier widening location and below Elevation 199.8 m at the west pier location.

6.10.2 Groundwater and Surface Water Control

The groundwater level at this site is typically between 1.6 m and 5 m below ground surface. It is noted that the granular (sand and silt) fill and the surficial silty sand deposit may be water-bearing, particularly during wet periods of the year. It is anticipated that the groundwater seepage into foundation excavations for the abutment widening can be adequately controlled by pumping from properly filtered sumps.

At the pier locations, in order to construct the widened pier footings in the dry, the water must be adequately lowered within a sheet pile cofferdam and tremie plug as described in Section 6.10.1. Once the sheet pile cofferdam and tremie plug have been placed and the excavation adequately sealed against water infiltration, properly filtered sump pumps can be used to pump out the remaining water and control minor seepage to allow construction of the footings in the dry.

6.10.3 Obstructions During Pile Driving

It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the contractor of the possible presence of cobbles and/or boulders within the overburden soils, as such obstructions were encountered at one borehole location and may affect the installation of steel H-piles for abutment widenings. A sample NSSP is provided in Appendix C.

6.10.4 Vibration Monitoring During Pile Installation

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing bridge structure are maintained below tolerable levels. An NSSP should be included in the Contract Documents for this purpose. A sample NSSP is provided in Appendix C.

A maximum peak particle velocity (PPV) of 50 mm/s is recommended at the existing bridge structure. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structure and if necessary, alter the pile driving criteria for the remaining piles.

6.11 Geotechnical Recommendations for the Cut Slope at Essa 5TH Line and Highway 89

It is understood that the current design for the proposed road widening and intersection improvement in the area of Essa 5th Line and Highway 89 for the section along the Highway from about Station 16+300 to Station 16+400 requires a cut-back of the existing north embankment slope to accommodate the proposed highway widening. Based on preliminary design drawings provided by MRC on January 15, 2008, the existing slope is between approximately 8 m and 10 m high and is inclined at about 2.2H: 1V.

The design drawings provided by MRC on May 8, 2009 indicate that the proposed cut slope would be inclined at 2 Horizontal to 1 Vertical (2H: 1V) with a 2 m wide bench at a height of about 6 m above the proposed toe of the slope, except at Station 16+400 where the proposed cut slope will be 9 m high without a mid-slope benh. It should be noted that MTO Standards are to include a bench on all earth slopes greater than 8 m high (as per OPSD 202.010).

6.11.1 Slope Stability Analyses

Static and seismic global slope stability analyses were carried out for an initially proposed slope configuration at Station 16+325 where the proposed cut slope would be 10.5 m high, inclined at 2H:1V and with a 2 m wide bench on the slope. The analyses were carried out using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis and using the soil parameters given below as interpreted from the subsurface information at Boreholes 09-1 and 09-2.

Soil Deposit	Bulk Unit Weight	Drained Shear Strength		Undrained Shear Strength
		c' (kPa)	ϕ'	
Loose Sandy Silt	19 kN/m^3	0	28°	–
Compact Sandy Silt	19 kN/m^3	0	33°	–
Very Stiff Silty Clay	17 kN/m^3	0	34°	120 kPa
Dense to very dense Silty Sand	20 kN/m^3	0	35°	–
Firm Silty Clay	17 kN/m^3	0	28°	50 kPa
Very dense Silt and Sand	19 kN/m^3	0	36°	–
Compact Silt	19 kN/m^3	0	32°	–
Very dense Silt	19 kN/m^3	0	35°	–

The input parameters for the soils described above were estimated from empirical correlations using the results of in situ Standard Penetration Tests, in conjunction with engineering judgment considering experience in similar soil conditions.

Effective stress parameters were employed in the analyses assuming drained conditions of the soils for the static condition. Undrained parameters of the cohesive soils were used under seismic loading conditions with horizontal peak ground acceleration (HPGA) equal to 0.05g. The design target Factor of Safety against global slope failure is 1.3 for the static condition and 1.1 for the seismic condition.

The results of the initial analyses indicate that the minimum Factor of Safety against surficial failure on the upper 4.5 m of the 2H:1V slope as proposed/shown on the design drawings is 1.0 under static condition and 0.9 under seismic condition, both of which are below the target design criteria. For the lower section of the 2H:1V slope below the bench, the minimum Factor of Safety against surficial failure was 1.4 under static condition and 1.3 under seismic condition, which are both acceptable.

Examples of surficial failure can be observed on the upper portion of the existing 2H:1V slope about 25 m to 30 m east of Borehole 09-2. Surficial erosion can be also observed on the slope above the existing gas control valves beside Essa 5th Line. The results of our analyses, presented above, suggest that slope instability similar to those conditions observed on the existing slopes will occur for the proposed 2H:1V upper slope.

Based on the above results, it was recommended to MRC that the upper 4.5 m portion of the slope be flattened to 2.5 H:1V to provide a stable slope in the long-term and achieve the target acceptable Factor of Safety (i.e. FS equal or greater than 1.3) against surficial failure. Alternative mitigation measures to stabilize the upper slope against surficial failure were also discussed with MRC. As a result, a revised configuration of the proposed cut slope at Station 16+325 was provided by MRC on May 21, 2009. The revised configuration consists of a 10.5 m high slope with a 2 m wide bench, inclined at 2H: 1V in the lower portion and at 2.5H: 1V in the upper portion. The location of the bench on the slope was not fixed to a specific elevation. Also, it was indicated by MRC that a 4.5 m wide setback distance between the crest edge of the slope and the north Right-of-Way of Highway 89 is required. Golder was requested by MRC to carry out additional slope stability analyses based on the revised configuration and provide a range of heights for the location of the bench relative to the upper and lower slopes.

Global slope stability analyses were carried out for the revised slope configuration provided at Station 16+325 with a 2 m wide bench that is sloped downward away from the upper slope at a 3 percent inclination as indicated in a typical cut slope cross section provided by MRC. The soil parameters provided above were used for the analyses. The analyses were carried out considering the required setback criteria from the crest edge of the slope and using different locations for the bench to obtain a slope configuration that satisfies the minimum Factor of Safety criteria under static and seismic conditions. The results of the slope stability analyses are summarized below and are shown on Figures D-1 to D-4 in Appendix D.

Slope Configuration (North Cut Slope Sta. 16+325)	**Min. FS Static Conditions**	**Min. FS Seismic Conditions (HPGA= 0.05g)**	**Distance from the edge of the slope crest to Highway 89 ROW (m)**
Upper slope (2.5H:1V) 2.5 m high; lower slope (2H:1V) 8 m high (Bench Elevation 218.3 m)	1.31 (Figure D-1)	1.16 (Figure D-3)	4.5m
Upper slope (2.5H:1V) 2.2 m and (2H:1V) 2 m high; lower slope (2H:1V) 6.3 m high (Bench Elevation 216.6 m)	1.34 (Figure D-2)	1.18 (Figure D-4)	4.5m

Based on the results of the slope stability analyses under static conditions, a 2 m wide bench located at about Elevation 218.3 m to accommodate a 2.5 m high upper slope at 2.5H:1V and a maximum 8 m high lower slope at 2H:1V (refer to Figure D-1, Appendix D) will achieve a Factor of Safety greater than 1.3. Alternatively, a 2 m wide bench located at about Elevation 216.6 m to accommodate a combination of a 2.2 m high upper slope at 2.5H:1V to Elevation 218.6 m and a 2 m high slope at 2H:1V below this elevation (refer to Figure D-2, Appendix D) will also achieve a Factor of Safety greater than 1.3. This configuration results in a 6.3 m high lower slope at 2H: 1V. The results of the corresponding slope stability analyses under seismic conditions are presented on Figures D-3 and D-4 in Appendix D.

The location of the proposed bench will be dependent on the finalized geometry of the road widening design which is being undertaken. Nonetheless, the upper slope must be configured at 2.5H:1V above Elevation 218.6 m, or lower, and the lower slope below this elevation may be configured at 2H:1V. The bench can be located on the slope at a convenient height below the above recommended 2.5H:1V upper slope to a maximum continuous slope height of 8 m measured either from the crest or from the toe of the slope.

An interceptor ditch should be provided at the top of the upper slope to reduce the potential for overflow of surface water over the slope. Provision should be made for vegetating the entire surface of the cut slope as well as placement of suitable erosion protection measures to protect against surficial erosion.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Houda Jadi, Ph.D., P.Eng. and Mr. Sen Hu, and reviewed by Mr. Jorge Costa, P.Eng. a Principal of Golder and a Designated MTO Contact, with technical input from Mr. Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Mr. Costa also provided a quality control review of this report for conformance with the project Terms of Reference.

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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
NOTTAWASAGA RIVER BRIDGE WIDENING, G.W.P. 2503-04-00

Foundation Element	Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs*
Abutment Widening	Spread footings founded beneath the surficial fills and loose silty sand soils	• Feasible	• Relatively lower costs than deep foundation elements • Standard/well understood construction methods; no specialized construction equipment required.	• Significant depth of excavation required with extensive temporary excavation support adjacent to the existing pile caps. • Differential settlement along length of abutment footings due to variable embankment loading. • Lower geotechnical resistance. • Different foundation system than existing abutments may not be compatible.	• High subexcavation and temporary support costs • Estimated cost $45,000 based on $300/m^3 for supply and placement of concrete footings
	Steel H-pile foundations driven to found within 100-blow silt to clayey silt or silty sand/sand	• Feasible and considered most appropriate from a foundations perspective	• Avoids differential settlement between foundation elements • Allows for integral / semi-integral abutment design • Compatible with existing abutment foundations	• Possible difficulty with cobbles/boulders within soil deposits • Requires contractor with pile installation experience • New piles have to be properly positioned to avoid interference with existing piles.	• Higher cost than spread footings • Estimated $50,000 based on 10 piles per abutment, 12 m long at $150/m and $300/m^3 for supply and placement of concrete pile caps
	Caisson foundations bearing within 100-blow silt to clayey silt or silty sand/sand	• Feasible	• Avoids differential settlement between foundation elements • Relatively higher bearing resistances than for steel H-piles	• Liner required due to soil conditions. Permanent liner recommended over temporary liner, to avoid difficulties with withdrawal of temporary liner due to length of caissons and presence of hard/very dense soils near caisson base, and to avoid “necking” of the caissons. • Possible difficulty with cobbles/boulders within soil deposits • Compatible with but not same as existing foundations • Potential for unbalanced hydrostatic head during installation requires tremie seal methods.	• Higher cost than steel H-piles, plus cost of permanent liner • Estimated cost $300,000 based on 6 caissons per abutment, 12 m long at $2,000 / m
Pier Widening	Spread footings founded on compact to very dense sand and gravel	• Feasible	• Relatively lower cost than deep foundations • Compatible with existing pier foundations	• Lower geotechnical resistance than deep foundations • Temporary cofferdam required • 3 m – 4 m underwater excavation required • Adequate tremie concrete pad or thick footing required.	• Considered least expensive option for the piers widening • Estimated cost $ 35,000 based on $300/m^3 for supply and placement of concrete footings (not including sheet piles)

TABLE 1 - continued
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
NOTTAWASAGA RIVER BRIDGE WIDENING, G.W.P. 2503-04-00

Foundation Element	Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs*
Pier Widening (Cont'd)	Steel H-pile foundation driven to found within 100-blow silt to clayey silt or silty sand/sand	• Not feasible	• Higher geotechnical resistance	• Longer construction time and high costs • Requires a specialized Contractor • Short length of pile (6m) may not be adequate to provide for scour protection • Potential vibrations induced on existing foundations from pile driving operations • Not same foundation support as existing foundation	• Higher costs than spread footings • $48,000 based on 7.5m long piles x $250/m and $300/m^3 for supply and placement of concrete pile caps
	Micropiles	• Not cost effective in comparison to the spread footing alternative	• Small diameter drilled piles avoid any significant disturbance to existing foundations	• Not same foundations support as existing foundation • Require deeper penetration than other deep foundations • Specialized contractor required • Supplementary study will be required	• Much higher costs than either spread footings or steel H-piles • about $450,000 based on piles installed to about 10 m depths plus supplementary study.

* Cost estimates are provided for foundation elements only and do not include mobilization costs, earthworks including excavation support structures, site access conditions, etc.

TABLE 2
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
RETAINING WALL, STATION 16+085 to STATION 16+130, G.W.P. 2503-04-00

Retaining Wall System / Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs*
Concrete Retaining Wall On Shallow Foundations	• Feasible with subexcavation of the loose sand and silt/silty sand with organics	• Relatively lower costs than deep foundation elements • Standard construction	• Significant depth of subexcavation with requirement for temporary excavation support at east end adjacent to the River • Costs for imported granular fill for replacement and disposal of existing soils containing organics • Relatively longer construction time • Potential for differential settlement due to variable density of subsoils at/below founding level	• Deep subexcavation, new material and temporary excavation support costs • Approximately $35,000 for supply and placement of concrete for retaining wall and foundation at $300/m^3.
Concrete Retaining Wall On Steel H-piles driven to found within 100-blow silty sand/sand	• Feasible	• Avoids deep excavations • Minimizes settlement of wall due to embankment widening construction	• Possible difficulty with cobbles/boulders within soil deposits • May require long (17 m) piles for small loading conditions	• Approximate cost $55,000 based on 20 piles, 10 m deep at $150/m and $300/m^3 for supply and placement of concrete wall and pile cap at $300/m^3
RSS Wall	• Feasible and considered most appropriate option from a foundations perspective	• Can accommodate differential settlement along wall length • Relatively easy construction and less expensive option compared to concrete retaining wall	• Specialized / proprietary design / construction	• Approximate cost $50,000 based on $350/m^2 of wall for design and supply of the wall materials.

* Cost estimates are provided for wall / foundation elements only and do not include mobilization costs, excavations including temporary support, site access conditions, etc.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	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.

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

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 ($\sigma' = \sigma - \mu$)
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
μ	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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
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 $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

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

Golder Associates

RECORD OF BOREHOLE No 07-1

 METRIC

PROJECT 05-1111-034			
W.P. 2503-04-00	LOCATION N 4891904.8 ;E 280029.5	ORIGINATED BY SB
DIST Central HWY 89	BOREHOLE TYPE Power Auger, 108 mm Hollow Stem Augers	COMPILED BY DD
DATUM Geodetic	DATE July 6, 2007	CHECKED BY JB/HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.6	GROUND SURFACE							
0.0 / 0.1	TOPSOIL							
	Silty sand to sand with roots and organics (FILL) Brown Moist					205		
204.8 / 0.8	SAND and SILT, trace clay and gravel Dense to very dense Brown to grey Wet 0.15 m cobble at 1.5 m (no sample recovery)	1	SS	31			o	
		~~2~~	~~SS~~	~~50/0.0~~	▽	204		
		3	SS	67		203	o	1 61 34 4
		4	SS	103		202		
		5	SS	112		201	o	
		6	SS	107		200		
199.2		7	SS	94/0.15			o	
6.4	END OF BOREHOLE NOTE: 1. Water level measured in open borehole upon completion of drilling at 1.5 m below ground surface (Elevation 204.1 m).							

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

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

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RECORD OF BOREHOLE No 07-2

 METRIC

PROJECT 05-1111-034			
W.P. 2503-04-00	LOCATION N 4891914.8 ;E 280057.6	ORIGINATED BY SB	
DIST Central HWY 89	BOREHOLE TYPE Power Auger, 108 mm Hollow Stem Augers	COMPILED BY DD	
DATUM Geodetic	DATE July 5, 2007	CHECKED BY JB/HJ	

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
205.4	GROUND SURFACE
0.0	TOPSOIL
0.2	SAND and SILT, containing organics, trace clay Loose Brown to grey Moist to wet
202.3 / 3.1	SAND and GRAVEL Compact Grey Wet
201.9 / 3.5	SAND, trace to some silt, trace to some gravel, trace clay Compact to very dense Grey Moist to wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	1	
3	SS	1	OC 3.7%
4	SS	23	
5	SS	26	8 81 10 1
6	SS	41	
7	SS	27	
8	SS	28	
9	SS	25	
10	SS	63	0 90 8 2
11	SS	38	
12	SS	58	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; 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: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No 07-2

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00 LOCATION N 4891914.8 ;E 280057.6 ORIGINATED BY SB
DIST Central HWY 89 BOREHOLE TYPE Power Auger, 108 mm Hollow Stem Augers COMPILED BY DD
DATUM Geodetic DATE July 5, 2007 CHECKED BY JB/HJ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT w (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SAND, trace to some silt, trace to some gravel, trace clay Compact to very dense Grey Moist to wet	13	SS	100/0.10		190	~15		11	81	6	2
						189						
		14	SS	100/0.15								
188.0 17.4	SILT, trace clay Very dense Grey Moist					188						
		15	SS	100/0.10		187	~20		NP			
186.7 18.7	END OF BOREHOLE NOTE: 1. Water level measured in open borehole upon completion of drilling at 0.8 m below ground surface (Elevation 204.6 m).											

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

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

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RECORD OF BOREHOLE No 07-3

 METRIC

PROJECT: 05-1111-034			
W.P.: 2503-04-00	LOCATION: N 4891920.8 ;E 280085.4	ORIGINATED BY: SB	
DIST: Central	HWY: 89	BOREHOLE TYPE: Power Auger, 108 mm Hollow Stem Augers	COMPILED BY: DD
DATUM: Geodetic	DATE: July 5, 2007	CHECKED BY: JB/HJ	

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS / GR SA SI CL
206.4	GROUND SURFACE						
0.0	TOPSOIL				206		
0.2	Silty sand (FILL) Brown Moist						
205.5 0.9	Silty SAND, trace clay, containing organics Very loose to loose Black and grey Moist	1	SS	5			
		2	SS	5	205		
		3	SS	5	204	59.5	OC 10.1%
		4	SS	3	203		0 70 25 5
		5	SS	3	202		
		6	SS	4		64.1	OC 7.7%
					201		
	Contains some gravel and slightly organic at 6.1 m depth	7	SS	6	200		OC 2.6%
199.1 7.3	Silty SAND, trace clay Very dense Brown to grey Moist				199		
		8	SS	107	198		0 76 22 2
197.3 9.1	SAND, some silt Very dense Grey Moist	9	SS	105/0.25	197		
					196		
195.4 10.9	END OF BOREHOLE	10	SS	100/0.125			

NOTES:

1. Water level measured in open borehole upon completion of drilling at 1.85 m below ground surface. (Elevation 204.5 m).

2. Water level measured in piezometer on July 12 at 1.6 m below ground surface (Elevation 204.8 m).

3. Water level measured in piezometer on July 31 at 1.7 m below ground surface (Elevation 204.7 m).

4. Water level measured in piezometer on August 29 at 1.6 m below ground surface (Elevation 204.8 m).

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

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RECORD OF BOREHOLE No 07-4

 METRIC

PROJECT 05-1111-034			
W.P. 2503-04-00	LOCATION N 4891920.5 ;E 280099.9		ORIGINATED BY SB
DIST Central	HWY 89	BOREHOLE TYPE Tripod, HQ Wash Boring	COMPILED BY DD
DATUM Geodetic	DATE July 12, 2007		CHECKED BY JB/HJ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
205.4	GROUND SURFACE
0.0	Sandy TOPSOIL, roots Very loose Brown Moist
204.8 0.6	SAND and SILT, trace clay, shells Very loose to loose, containing organics Grey to brown Moist Wet below 1.2 m depth
202.3 3.1	Silty SAND, trace clay, containing organics Very loose to loose Grey to black Wet
200.8 4.6	SILT and SAND, trace gravel and clay Compact to very dense Brown Wet
198.4 7.0	SAND, trace to some silt Very dense Reddish brown to grey Wet
197.5 7.9	END OF BOREHOLE

NOTE:

1. Water level measured in open borehole upon completion of drilling at 0.8 m below ground surface. (Elevation 204.6 m).

SAMPLES

NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3		
2	SS	3		
3	SS	8		OC 3.6%
4	SS	4		0 60 35 5
5	SS	4		OC 9.5%
6	SS	7	71.4	NP
7	SS	26		
8	SS	114		3 35 60 2
9	SS	90/0.15		

GROUND WATER CONDITIONS: ∇ (at 0.8 m depth)

ELEVATION SCALE: 205, 204, 203, 202, 201, 200, 199, 198

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

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

MIS-MTO 001 05-1111-034.GPJ GAL-MISS.GDT 9/16/09 DD

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RECORD OF BOREHOLE No 07-5

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00
LOCATION N 4891937.5 ;E 280151.9
ORIGINATED BY SB
DIST Central HWY 89
BOREHOLE TYPE Power Auger, 108 mm Hollow Stem Augers
COMPILED BY DD
DATUM Geodetic
DATE July 3, 2007
CHECKED BY JB/HJ

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
210.1	GROUND SURFACE								
0.0	TOPSOIL								
0.2	Sand and silt, some gravel, trace clay, (FILL) Loose to compact Brown Moist								
		1	SS	6	209				
		2	SS	5	208				
		3	SS	13	207				
		4	SS	10					
		5	SS	13	206	12	50	32	6
		6	SS	14	205				
204.5 / 5.6	Silty SAND, some gravel, trace clay Very dense Brown Moist				204				
		7	SS	80					
203.0 / 7.1	SILT to CLAYEY SILT Hard Grey Wet				203				
		8	SS	66	202				
		9	SS	93	201	0	0	85	15
					200				
		10	SS	123	199				
		11	SS	114	198				
					197				
196.0 / 14.0		12	SS	95/0.15					

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

Continued Next Page

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

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RECORD OF BOREHOLE No 07-5

 METRIC

PROJECT 05-1111-034			
W.P. 2503-04-00	LOCATION N 4891937.5 ;E 280151.9		ORIGINATED BY SB
DIST Central	HWY 89	BOREHOLE TYPE Power Auger, 108 mm Hollow Stem Augers	COMPILED BY DD
DATUM Geodetic		DATE July 3, 2007	CHECKED BY JB/HJ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTES: 1. Water level measured in open borehole upon completion of drilling at 9.1 m below ground surface. (Elevation 201.0 m). 2. Water level measured in piezometer on July 12 at 4.8 m below ground surface (Elevation 205.3 m). 3. Water level measured in piezometer on July 31 at 4.7 m below ground surface (Elevation 205.4 m). 4. Water level measured in piezometer on August 29 at 4.8 m below ground surface (Elevation 205.3 m).										

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

RECORD OF BOREHOLE No 07-6

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00
LOCATION N 4891943.4 ;E 280171.3
ORIGINATED BY SB
DIST Central HWY 89
BOREHOLE TYPE Power Auger, 101 mm Solid Stem Augers
COMPILED BY DD
DATUM Geodetic
DATE July 3, 2007
CHECKED BY JB/HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.5	GROUND SURFACE					
0.0	Sand and silt, trace to some gravel (FILL) Loose to compact Brown Moist				210	
		1	SS	16		
		2	SS	5	209	
		3	SS	4	208	
		4	SS	4		
206.8 / 3.7	Clayey Silt, trace gravel Hard Brown to grey Moist				207	
		5	SS	43		
		6	SS	52	206	0 0 74 26
					205	
204.0	Becoming grey at 6.3 m	7	SS	94/0.25		
6.5	END OF BOREHOLE					

NOTE:

1. Borehole dry upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No 08-1

 METRIC

PROJECT 05-1111-034

W.P. 2503-04-00 | LOCATION N 4891924.3 ;E 280108.8 | ORIGINATED BY PKS

DIST Central HWY 89 | BOREHOLE TYPE Wash Rotary - NQ Casing | COMPILED BY SH

DATUM Geodetic | DATE November 12, 2008 | CHECKED BY JMAC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
204.6	RIVER SURFACE									
0.0	Water				204					
203.7 / 0.9	SAND and GRAVEL, trace silt and clay, occasional cobble, wood fragments Loose to dense Brown Wet	1	50 DO	4						
		2	50 DO	14	203		48	47	3	2
		3	50 DO	19	202					
		4	50 DO	37	201					
		5	50 DO	27	200					
199.8 / 4.7	SAND, trace to some silt, trace gravel and clay, occasional cobble Very dense Brown Wet	6	50 DO	67/0.15						
		7	50 DO	86	199					
		8	50 DO	85/0.15	198 / 197		5	85	7	3
		9	50 DO	123	196 / 195					
194.5 / 10.1	CLAYEY SILT, trace to some sand Hard Grey Wet	10	50 DO	75/0.15	194					
		11	50 DO	100/0.18	193					
192.8 / 11.8	END OF BOREHOLE									

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

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

RECORD OF BOREHOLE No 08-2

METRIC

PROJECT 05-1111-034

W.P. 2503-04-00 | LOCATION N 4891931.6 ;E 280133.4 | ORIGINATED BY PKS

DIST Central HWY 89 | BOREHOLE TYPE Wash Rotary - NQ Casing | COMPILED BY SH

DATUM Geodetic | DATE November 11, 2008 | CHECKED BY JMAC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.6	RIVER SURFACE					
0.0	Water				204	
203.1 / 1.5	SAND and GRAVEL, trace silt and clay, occasional cobble (below 3.05 m) Loose to very dense Brown Wet	1	50 DO	8	203	
		2	50 DO	32	202	
		3	50 DO	86		48 47 3 2
		4	50 DO	85	201	
		5	50 DO	60/0.15	200	
199.1 / 5.5	CLAYEY SILT, trace to some sand Hard Grey Wet	6	50 DO	86	199	
		7	50 DO	100/0.2		5 1 72 22
		8	50 DO	96/0.15	198	
					197	
		9	50 DO	100/0.10	196	
					195	
194.6 / 10.0	END OF BOREHOLE	10	50 DO	100/0.10		

SOIL PROFILE | SAMPLES | STRAT PLOT | GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No 08-3

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00
LOCATION N 4891920.8 ;E 280113.2
ORIGINATED BY MWK
DIST Central HWY 89
BOREHOLE TYPE Wash Rotary - NQ Casing
COMPILED BY SH
DATUM Geodetic
DATE November 14, 2008
CHECKED BY JMAC

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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.7	WATER SURFACE										
0.0	Water						204				
202.8 1.9	Silty sand, trace gravel, occasional decayed wood (FILL/sediment) Grey						203				
202.2 2.5	Concrete						202 201				
200.5 4.2	SAND, trace gravel Very dense Brown Wet		1	SS	72		200		~24 (O)		
199.8 4.8	END OF BOREHOLE										

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

RECORD OF BOREHOLE No 08-4

 METRIC

PROJECT 05-1111-034

W.P. 2503-04-00 LOCATION N 4891928.4 ;E 280137.6 ORIGINATED BY MWK

DIST Central HWY 89 BOREHOLE TYPE Wash Rotary - NQ Casing COMPILED BY SH

DATUM Geodetic DATE November 13, 2008 CHECKED BY JMAC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.7	WATER SURFACE						
0.0	Water						
203.3 / 1.4	Silty sand, trace gravel, occassional decayed wood, (FILL/sediment) Grey						
202.6 / 2.0	Concrete						
201.2 / 3.5	CLAYEY SILT, trace to some sand Hard Grey Wet	1	SS	57			
		2	SS	88			
200.0 / 4.7	END OF BOREHOLE						

Elevation scale: 204, 203, 202, 201, 200

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

Golder Associates

RECORD OF BOREHOLE No 09-1

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00
LOCATION N 4892001.4 ;E 280261.4
ORIGINATED BY TB
DIST Central HWY 89
BOREHOLE TYPE D-50 Track-mounted Power Auger, 210 mm O.D. Hollow Stem Augers
COMPILED BY SH
DATUM Geodetic
DATE April 9, 2009
CHECKED BY JMC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
220.9	GROUND SURFACE
0.0 / 220.6	TOPSOIL
0.3	Sandy SILT, trace to some clay Loose to compact Brown Moist to wet
216.2 / 4.7	SILTY CLAY, trace sand Very stiff Grey/brown Moist
215.3 / 5.6	Silty SAND, trace clay Dense to very dense Brown to grey Moist - Orange brown between 10.67 m and 11.28 m depth
209.6 / 11.3	SILT and SAND, trace clay Very dense Grey Moist to wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	4				
2	SS	6				
3	SS	4	0	28	62	10
4	SS	16				
5	SS	23				
6	SS	14				
7	SS	16	0	2	51	47
8	SS	43				
9	SS	103				
10	SS	113	0	68	30	2
11	SS	86				
12	SS	124				
13	SS	90	0	43	55	2

STRAT PLOT | 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

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

RECORD OF BOREHOLE No 09-1

 METRIC

PROJECT 05-1111-034

W.P. 2503-04-00 LOCATION N 4892001.4 ;E 280261.4 ORIGINATED BY TB

DIST Central HWY 89 BOREHOLE TYPE D-50 Track-mounted Power Auger, 210 mm O.D. Hollow Stem Augers COMPILED BY SH

DATUM Geodetic DATE April 9, 2009 CHECKED BY JMC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
205.1	SILT and SAND, trace clay Very dense Grey Moist to wet		14	SS	128				o		
15.9	END OF BOREHOLE						205				

Notes:

1. Groundwater seepage at a depth of 1.5 m (Elevation 219.4 m).

2. Groundwater level at a depth of 3.9 m (Elevation 217.0 m) upon completion of drilling.

3. Groundwater level measured in monitoring well on May 8, 2009 at a depth of 10.0 m below ground surface (Elevation 210.9 m).

MIS-MTO 001 05-1111-034.GPJ GAL-MISS.GDT 9/16/09 DD

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

Golder Associates

RECORD OF BOREHOLE No 09-2

 METRIC

PROJECT 05-1111-034
W.P. 2503-04-00 LOCATION N 4891968.8 ;E 280257.3 ORIGINATED BY TB
DIST Central HWY 89 BOREHOLE TYPE D-50 Track-mounted Power Auger, 108 mm O.D. Solid Stem Augers COMPILED BY SH
DATUM Geodetic DATE April 9, 2009 CHECKED BY JMC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT kN/m³	GR	SA	SI	CL
211.7	GROUND SURFACE										
0.0	Silty sand, some gravel, trace organic matter (FILL) Loose Brown Moist	1	SS	6		211					
210.9 / 0.8	CLAYEY SILT, trace gravel, trace sand Firm Grey Moist	2	SS	4							
210.3 / 1.4	SILT, some sand, some clay Compact Grey Wet	3	SS	20	∇	210	NP	0	17	69	14
209.4 / 2.3	SILT and SAND Compact Grey Wet	4	SS	23		209					
208.7 / 3.0	SILT, some clay, trace sand Very dense Grey Moist to wet	5	SS	70		208					
		6	SS	84		207					
		7	SS	90			NP	0	3	83	14
206.5 / 5.2	END OF BOREHOLE										

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

Notes:

1. Groundwater seepage at a depth of 1.5 m (Elevation 210.2 m).

2. Groundwater level at a depth of 2.1 m (Elevation 209.6 m) upon completion of drilling.

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

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

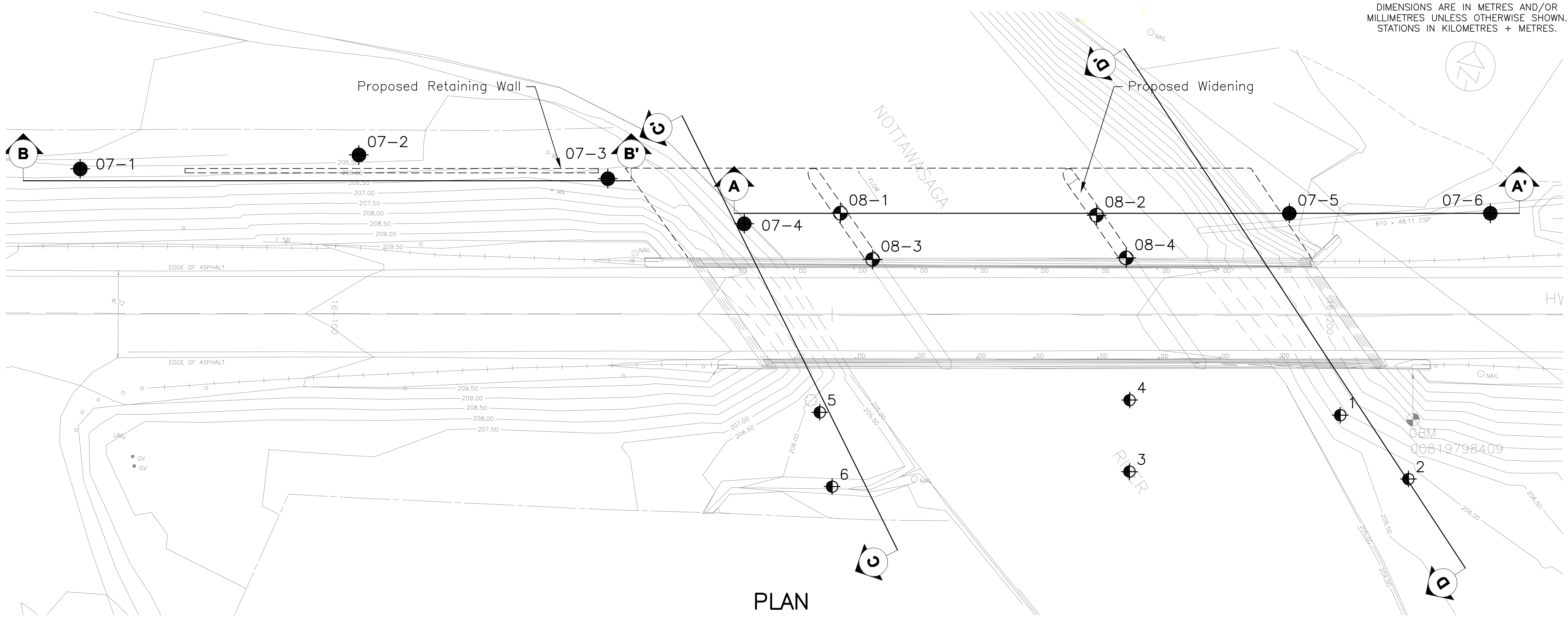
CONT No.
WP No.2503–04–00

HIGHWAY 89
NOTTAWASAGA RIVER BRIDGE WIDENING AND RETAINING WALL

BOREHOLE LOCATION AND SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



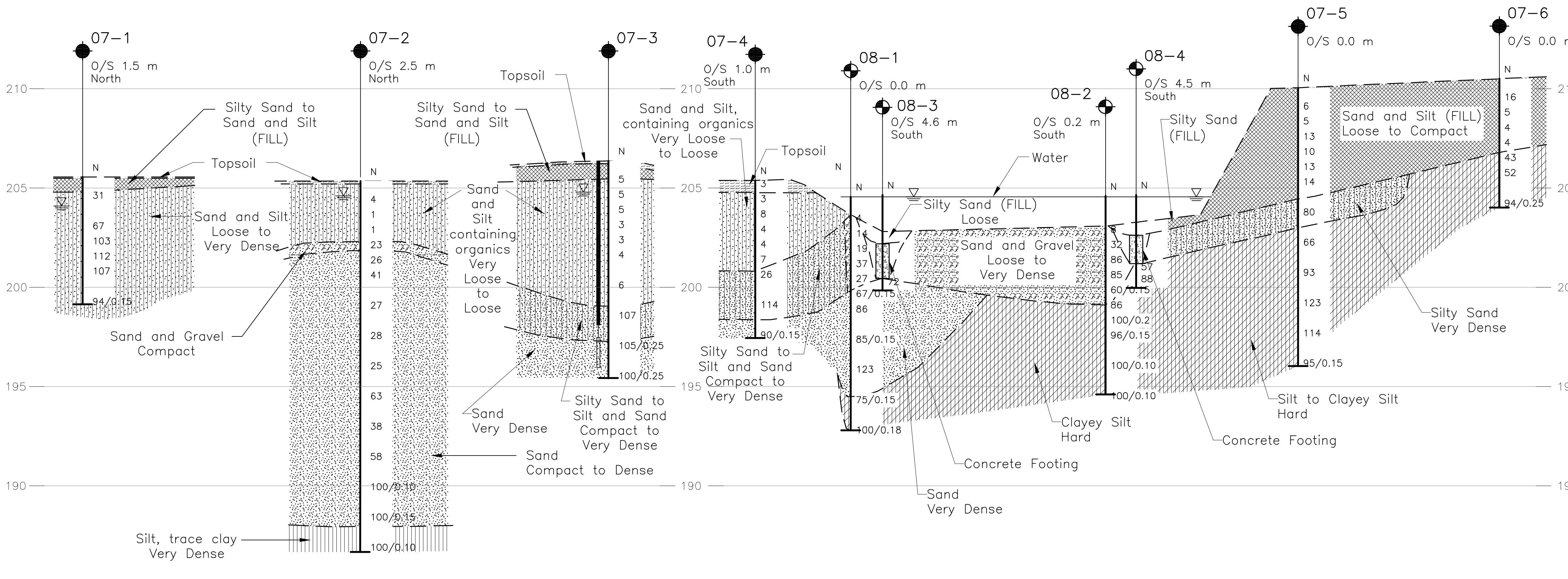
PLAN
SCALE 5 0 5 10 m

KEY PLAN
NOT TO SCALE

LEGEND

- Borehole current Investigation in 2008
- Approximate Borehole Location in 2007
- Approximate Borehole Location – 1959 Investigation
- 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 08/29/07
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
08–1	204.6	4891924.3	280108.8
08–2	204.6	4891931.6	280133.4
08–3	204.7	4891920.8	280113.2
08–4	204.7	4891928.4	280137.6
07–1	205.6	4891904.8	280029.5
07–2	205.4	4891914.8	280057.6
07–3	206.4	4891920.8	280085.4
07–4	205.4	4891920.5	280099.9
07–5	210.1	4891937.5	280151.9
07–6	210.5	4891943.4	280171.3



PROFILE B–B'

PROFILE A–A'

SCALE
HOR 5 0 5 10 m
VER 2.5 0 2.5 5 m

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by MRC, drawing file no. E–247–09–1, dated 01, 2006 by J.D. Barnes, received on 07, 23, 2007 and MRC drawing P1 entitled "Highway 89 Nottawasaga River Bridge Rehabilitation, Preliminary General Arrangement, dated Jan/07, Report No 59–F–98 W.P. 218–59 entitled "HWY #89 and Nottawasaga River DIST#5".

NO.	DATE	BY	REVISION

Geocres No. 31D–454			
HWY. 89	PROJECT NO. 05–1111–034		DIST.
SUBM'D. JB	CHKD. HJ	DATE: 16–Sep–09	SITE:
DRAWN: DD	CHKD. SH	APPD. JMAC	DWG. 1

PLOT DATE: September 16, 2009

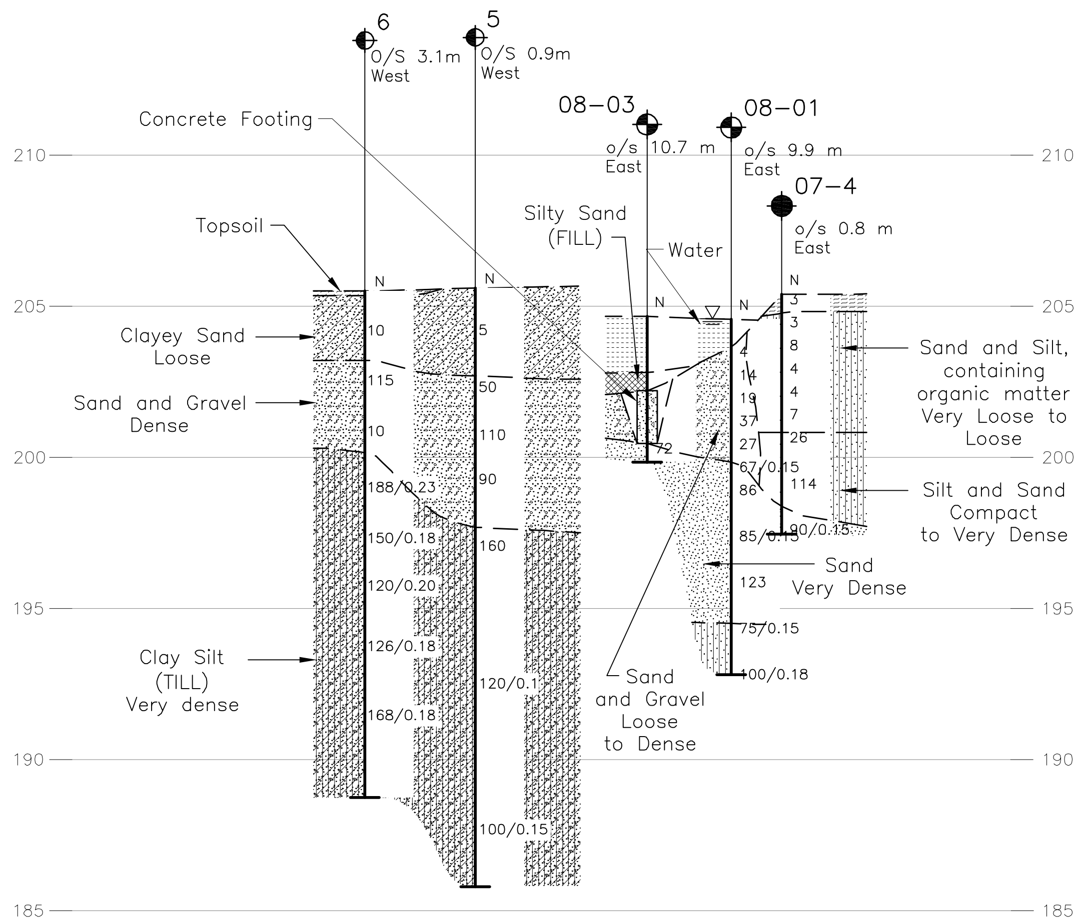
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.2503-04-00

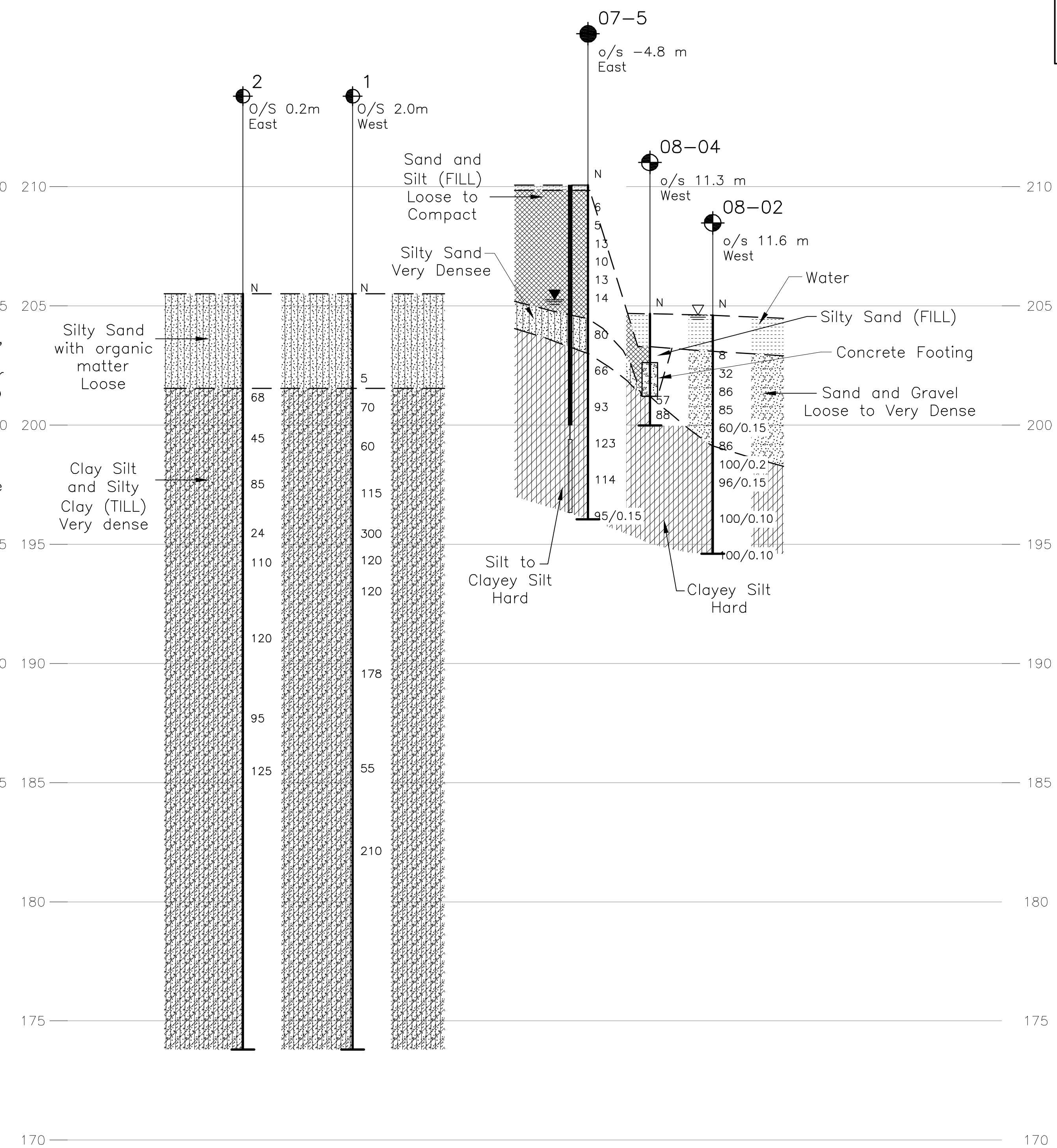
HIGHWAY 89
NOTTAWASAGA RIVER BRIDGE WIDENING AND RETAINING WALL
SOIL STRATA

SHEET

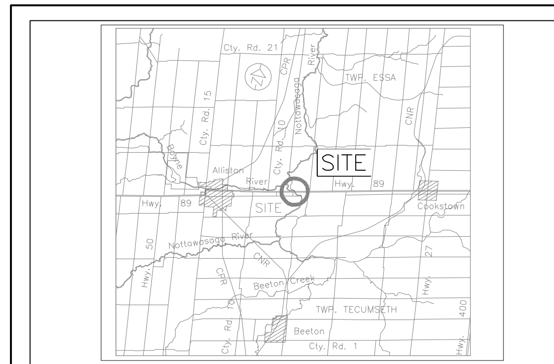
Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



CROSS SECTION C–C'



CROSS SECTION D–D'



KEY PLAN
NOT TO SCALE

LEGEND

- Borehole current Investigation in 2008
- Approximate Borehole Location in 2007
- Approximate Borehole Location – 1959 Investigation
- 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 08/29/07
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
08–1	204.6	4891924.3	280108.8
08–2	204.6	4891931.6	280133.4
08–3	204.7	4891920.8	280113.2
08–4	204.7	4891928.4	280137.6
07–1	205.6	4891904.8	280029.5
07–2	205.4	4891914.8	280057.6
07–3	206.4	4891920.8	280085.4
07–4	205.4	4891920.5	280099.9
07–5	210.1	4891937.5	280151.9
07–6	210.5	4891943.4	280171.3

NOTES

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REFERENCE

Base plans provided in digital format by MRC, drawing file no. E-247-09-1, dated 01, 2006 by J.D. Barnes, received on 07, 23, 2007 and MRC drawing P1 entitled "Highway 89 Nottawasaga River Bridge Rehabilitation, Preliminary General Arrangement, dated Jan/07, Report No 59-F-98 W.P. 218-59 entitled "HWY #89 and Nottawasaga River DIST#5".

SCALE
HOR 5 0 5 10 m
VER 2.5 0 2.5 5 m

NO.	DATE	BY	REVISION

Geocres No. 31D-454

HWY. 89	PROJECT NO. 05-1111-034	DIST.
SUBM'D. JB	CHKD. HJ / DATE: 16-Sep-09	SITE:
DRAWN: DD	CHKD. SH / APPD. JMAC	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

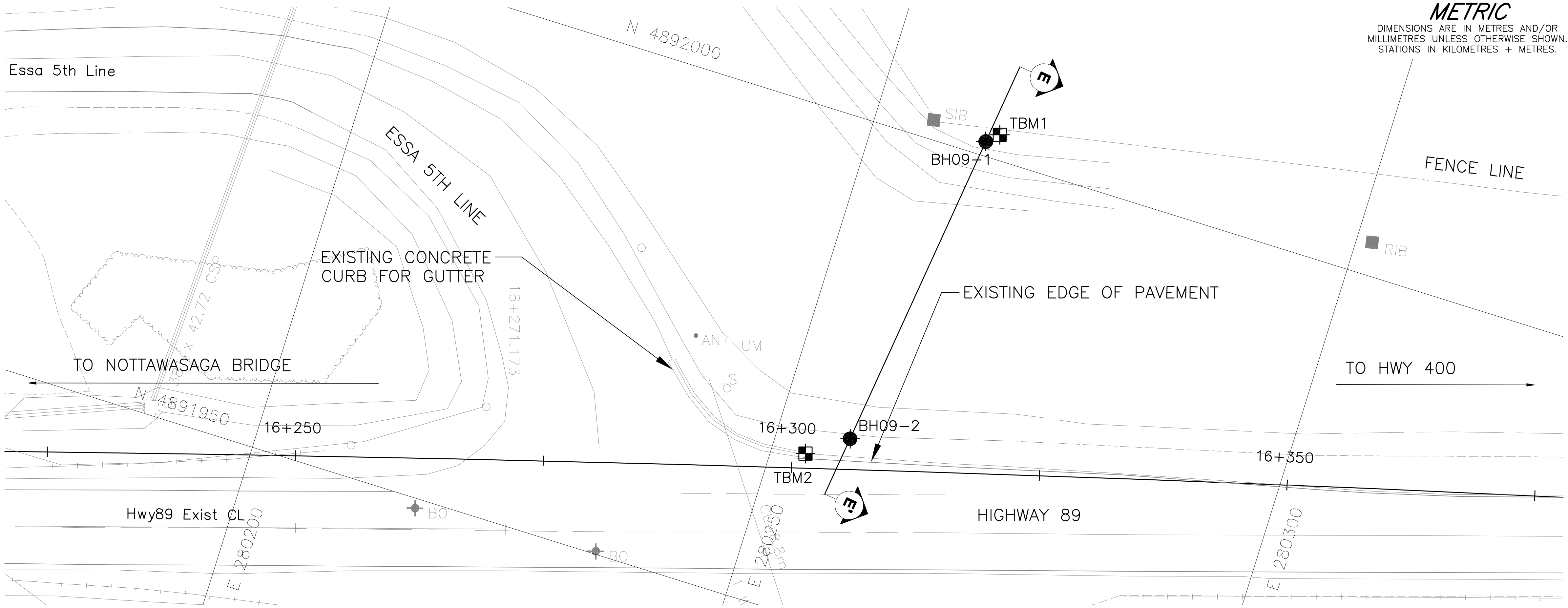
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2503-04-00

HIGHWAY 89
CUT SLOPE BETWEEN STA. 16+300 AND STA. 16+400
BOREHOLE LOCATION AND SOIL STRATA

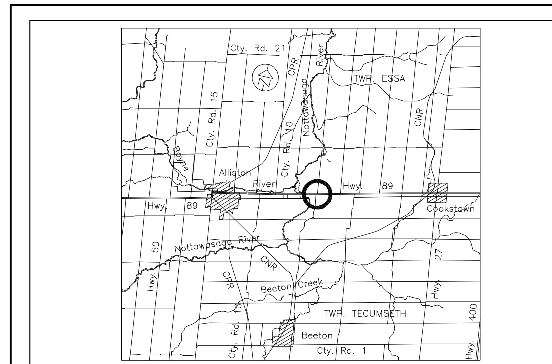
SHEET

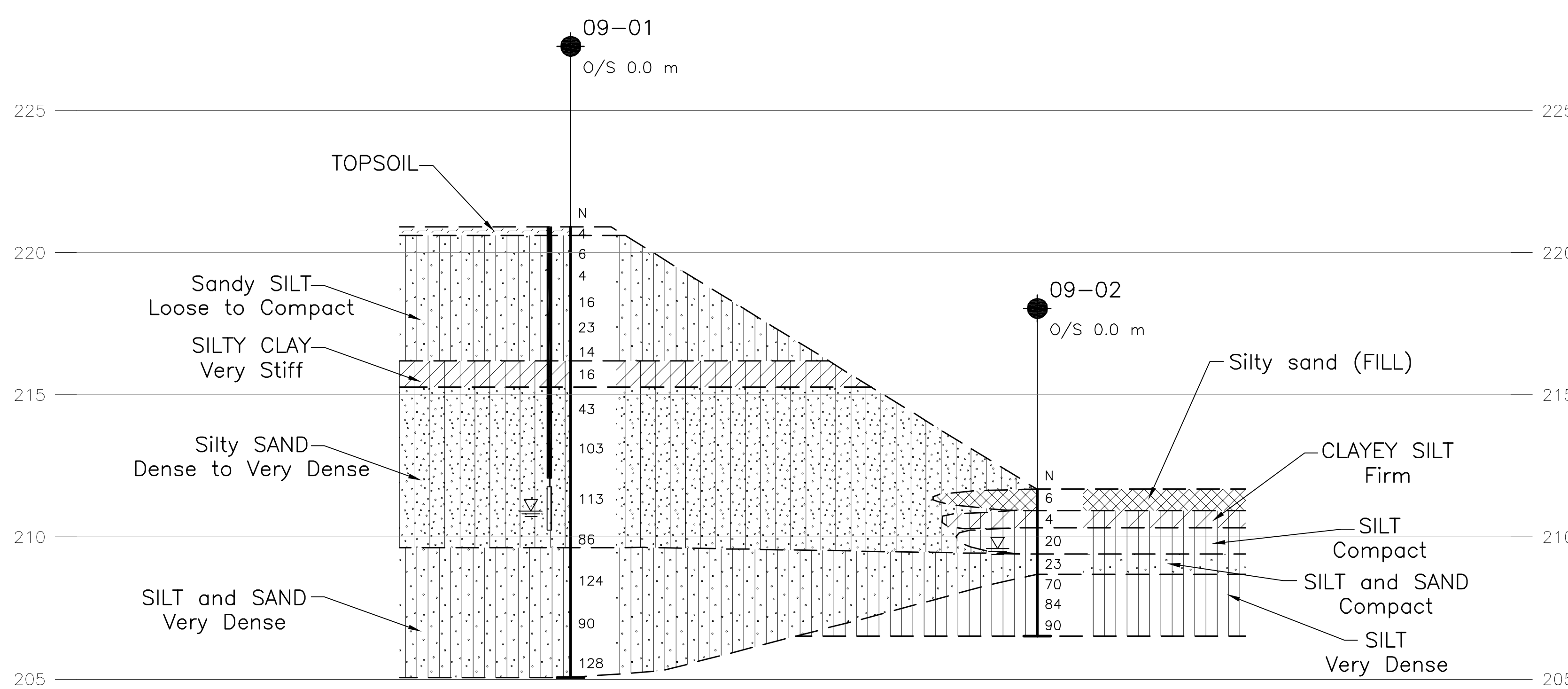
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE 5 0 5 10 m





CROSS SECTION E-E

SCALE HORIZONTAL 5 0 5 10 m
2.5 0 2.5 5 m SCALE VERICAL

LEGEND

- Borehole – Current Investigation
- Temporary Bench Mark (TBM)
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on May 8, 2009
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
09-1	220.9	4892001.4	280261.4
09-2	211.7	4891968.8	280257.3

NOTES

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REFERENCE

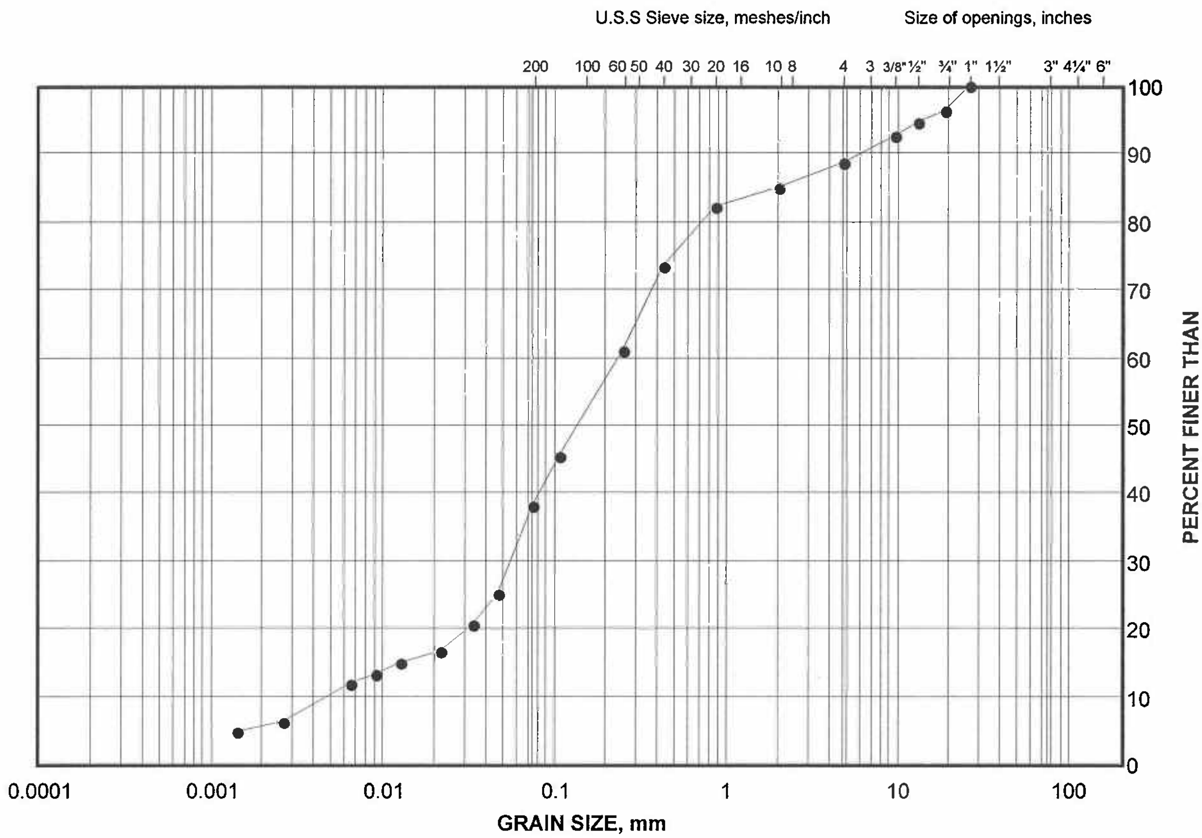
Locations and elevations of Two Temporary Bench marks were provided by MRC on April 3, 2009.

Base plans provided in digital format by MRC, drawing file nos. 6246mp-Hwy89 Nott (for Golder)-Apr16-09, received April 16, 2009.

NO.	DATE	BY	REVISION

Geocres No. 31D-454			
HWY. 89		PROJECT NO. 05-1111-034	DIST.
SUBM'D. TB	CHKD. SH	DATE: 16-Sep-09	SITE:
DRAWN: DD	CHKD. SH	APPD. JMAC	DWG. 3

GRAIN SIZE DISTRIBUTION Sand and Silt (Fill)	FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-5	5	205.9

Project Number: 05-1111-034

Checked By: HJ

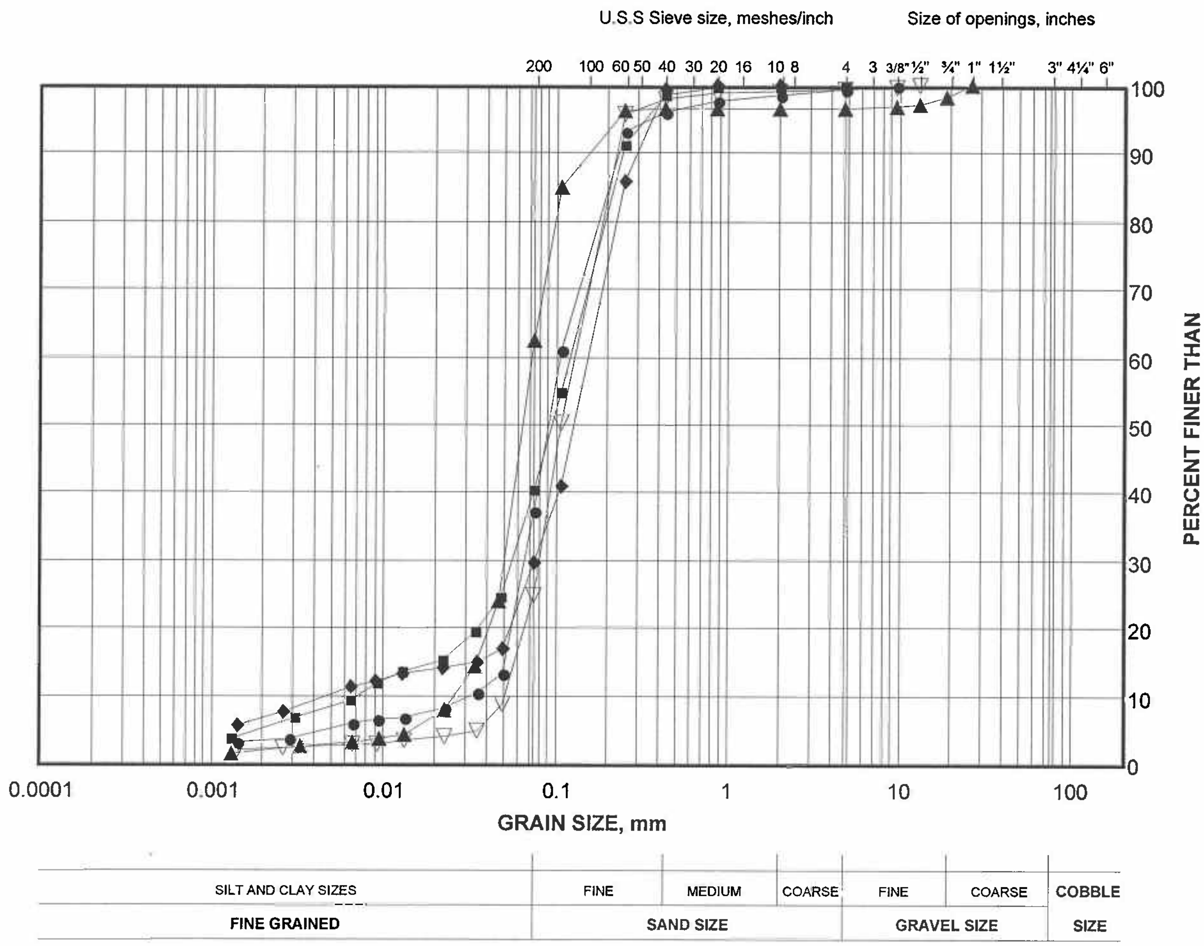
Golder Associates

Date: 25-Sep-07

GRAIN SIZE DISTRIBUTION

Sand and Silt to Silty Sand

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-1	3	202.9
■	07-4	4	202.8
◆	07-3	4	203.0
▲	07-4	8	199.0
▽	07-3	8	198.4

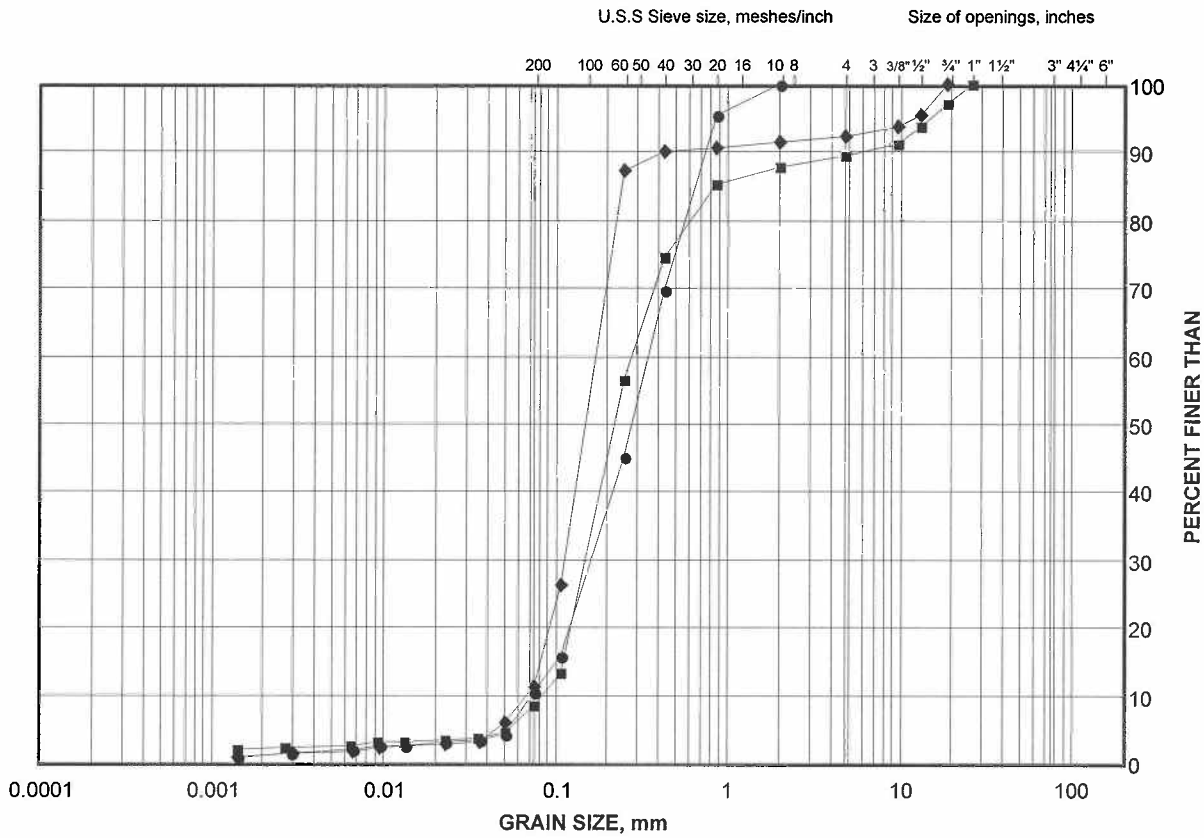
Project Number: 05-1111-034

Checked By: HJ

Golder Associates

Date: 25-Sep-07

GRAIN SIZE DISTRIBUTION Sand	FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-2	10	194.4
■	07-2	13	190.1
◆	07-2	5	201.2

Project Number: 05-1111-034

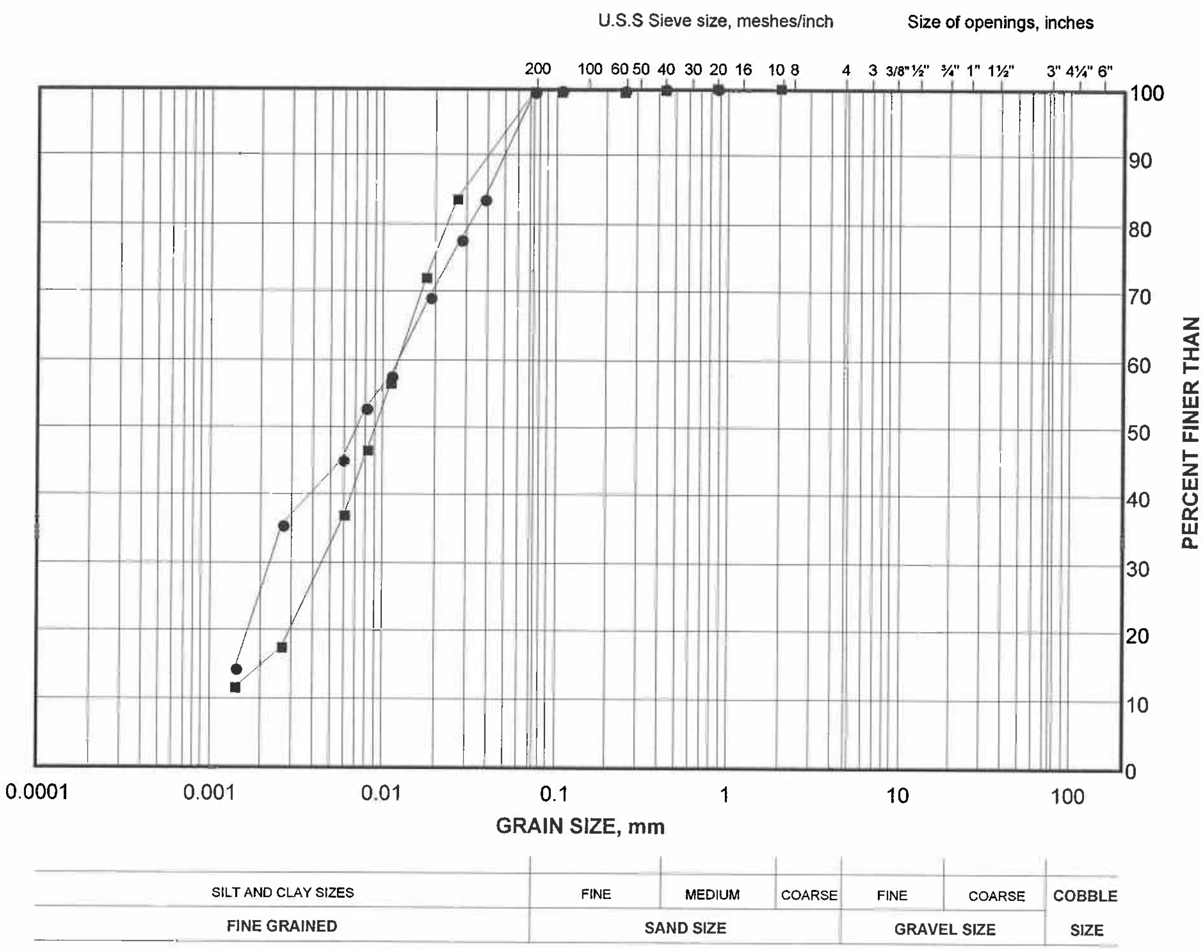
Checked By: HJ

Golder Associates

Date: 25-Sep-07

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-6	6	205.7
■	07-5	9	200.6

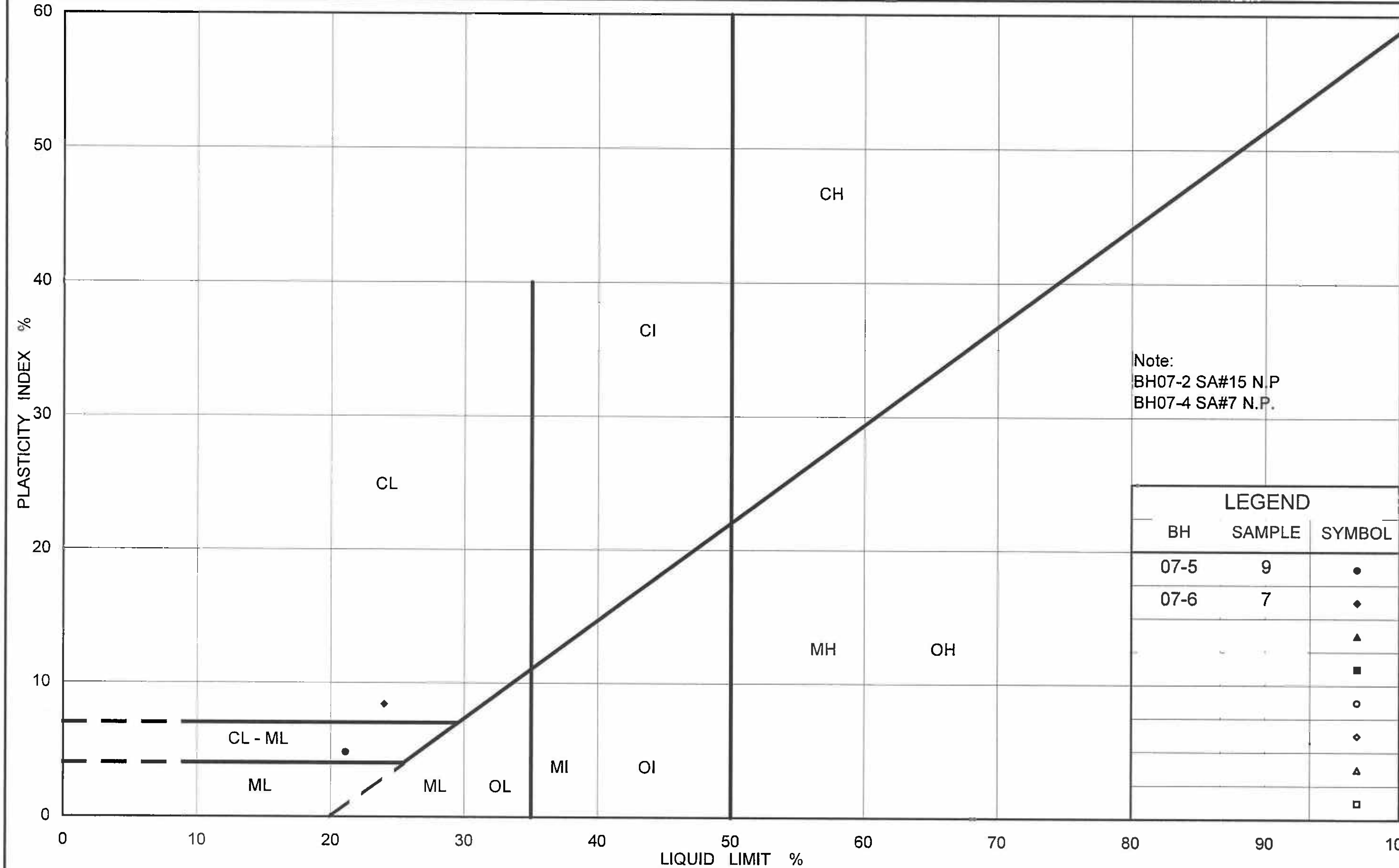
Project Number: 05-1111-034

Checked By: HJ

Golder Associates

Date: 25-Sep-07

Oct 75, FF-S-21



Note:
BH07-2 SA#15 N.P
BH07-4 SA#7 N.P.

LEGEND

BH	SAMPLE	SYMBOL
07-5	9	●
07-6	7	◆
		▲
		■
		○
		◇
		△
		□

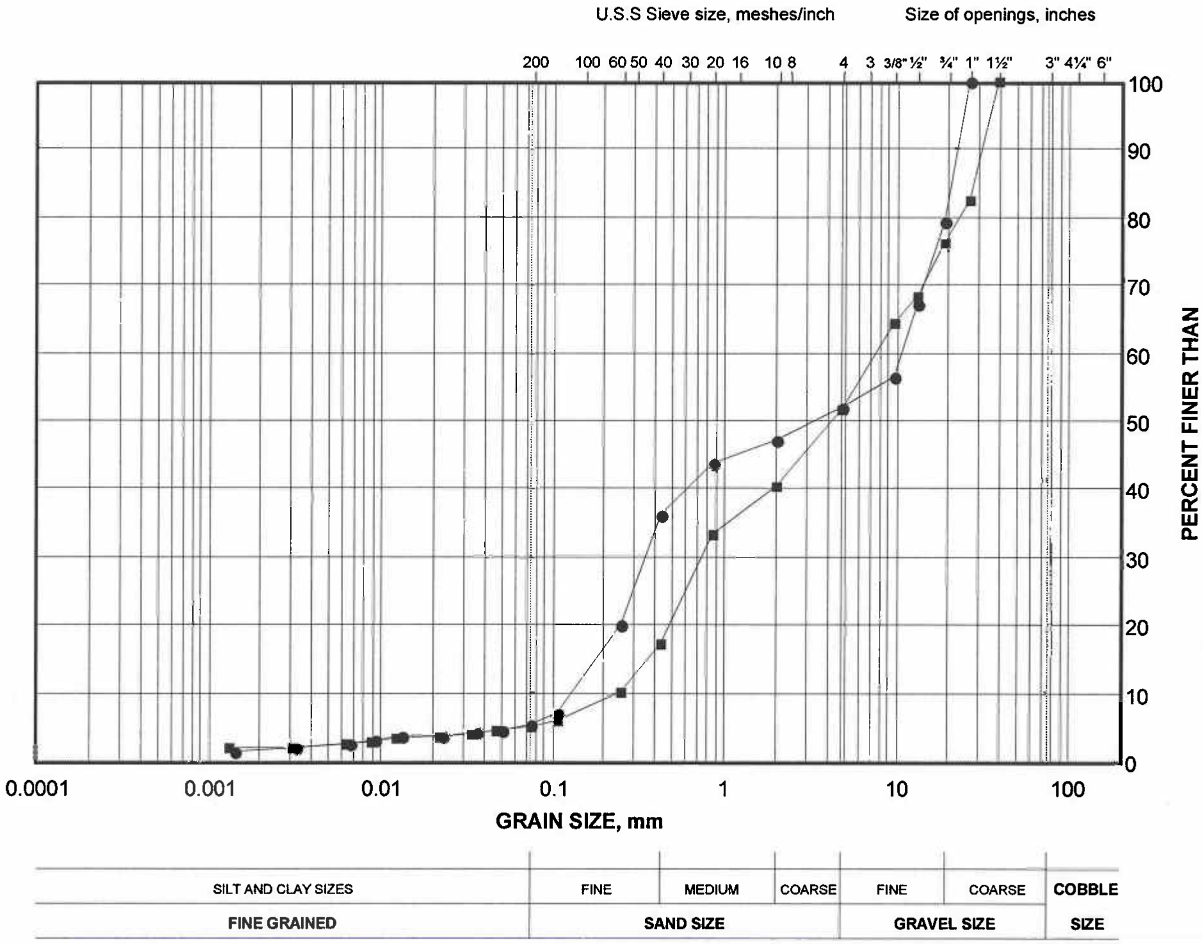
Ministry of Transportation
Ontario

PLASTICITY CHART
Silt to Clayey Silt

Figure No. 5
Project No. 05-1111-034
Checked By: HJ

GRAIN SIZE DISTRIBUTION
Sand and Gravel

FIGURE 6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	08-1	2	202.6
■	08-2	3	201.2

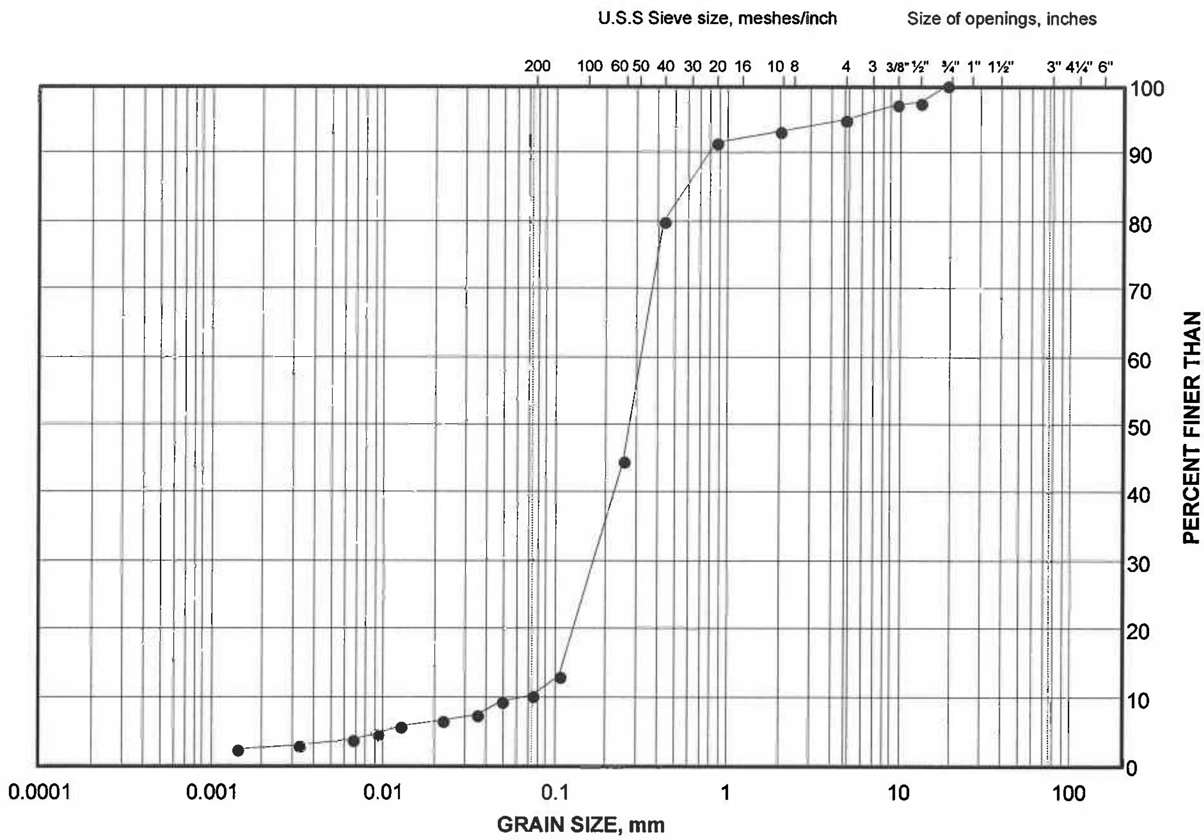
Project Number: 05-1111-034

Checked By: Sen

Golder Associates

Date: 09-Sep-09

GRAIN SIZE DISTRIBUTION Sand	FIGURE 7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	08-1	8	197.5

Project Number: 05-1111-034

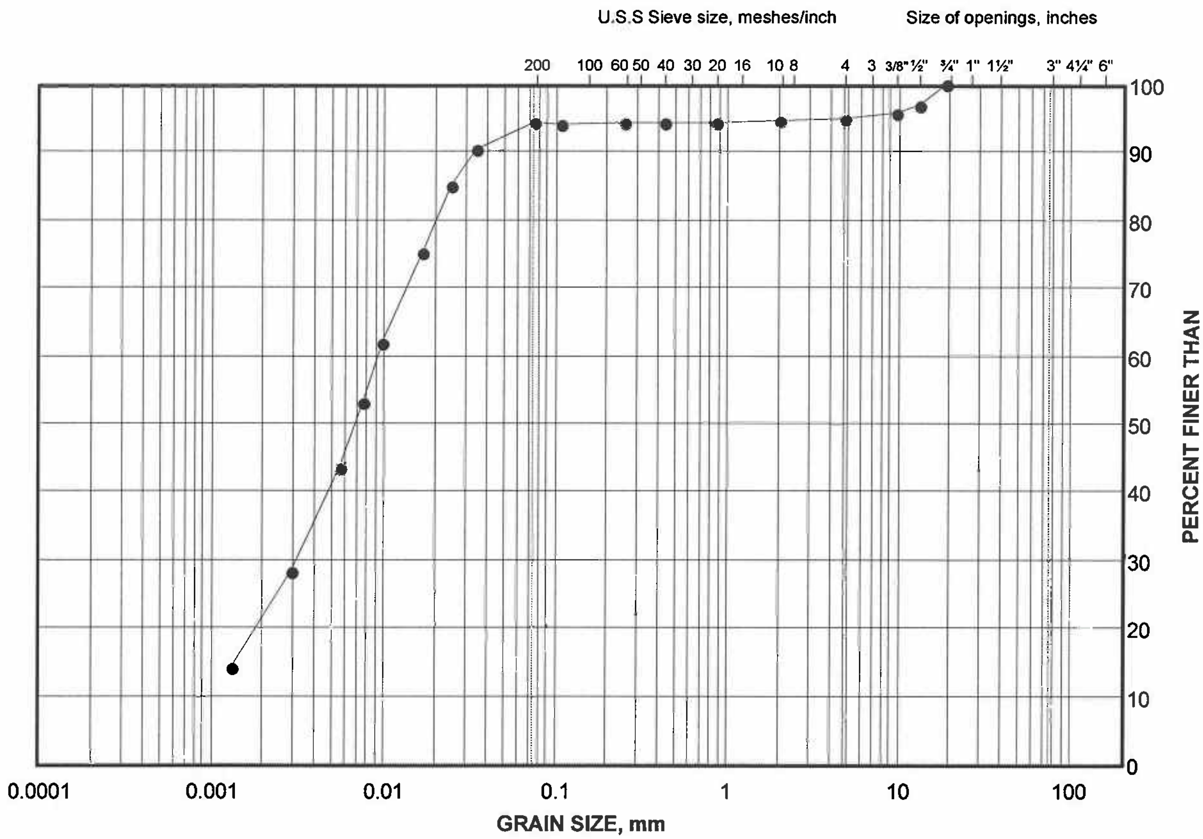
Checked By: Sen

Golder Associates

Date: 09-Sep-09

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE 8



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

LEGEND

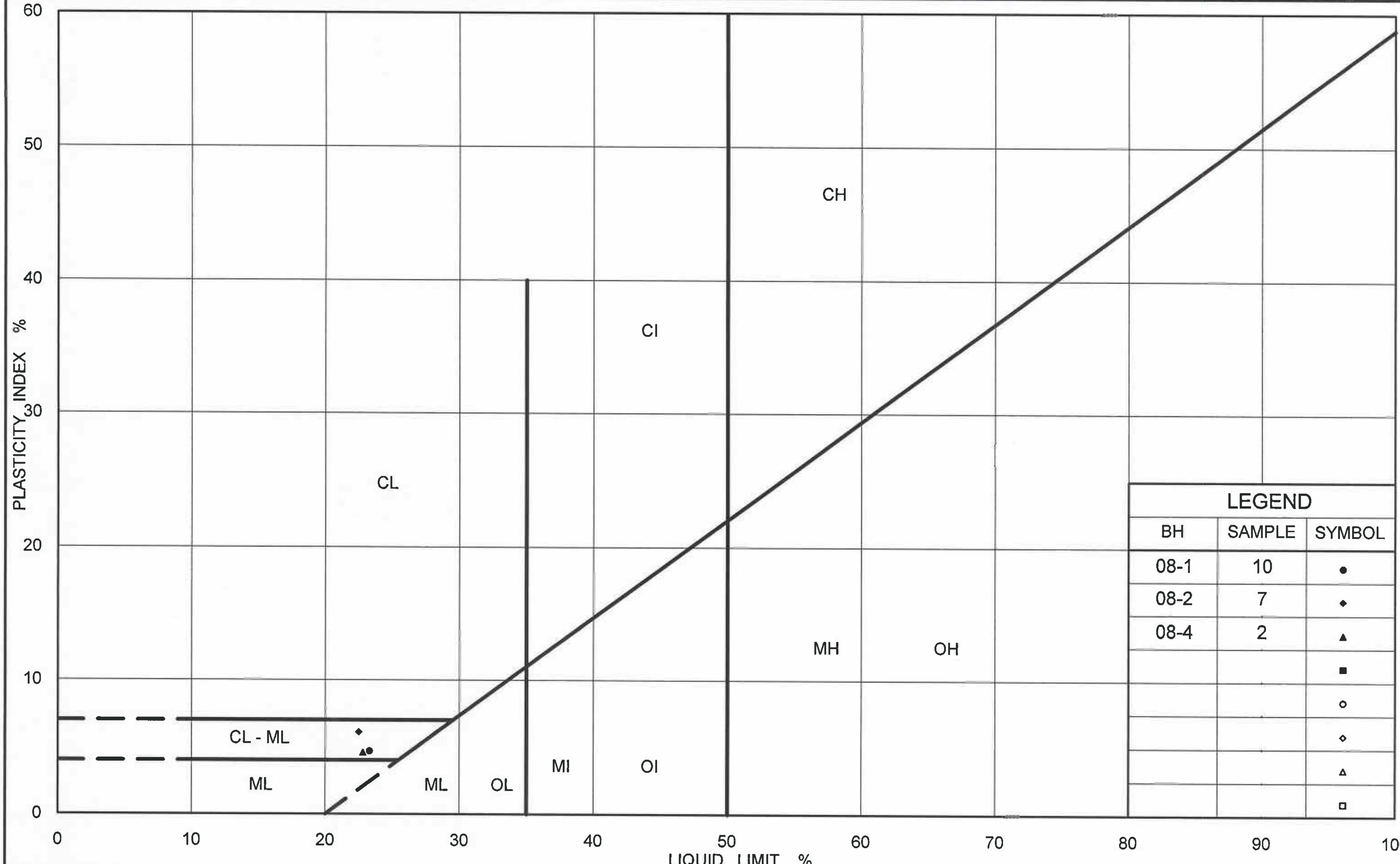
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	08-2	7	198.4

Project Number: 05-1111-034

Checked By: Sen

Golder Associates

Date: 09-Sep-09



LEGEND		
BH	SAMPLE	SYMBOL
08-1	10	●
08-2	7	◆
08-4	2	▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

Figure No. 9

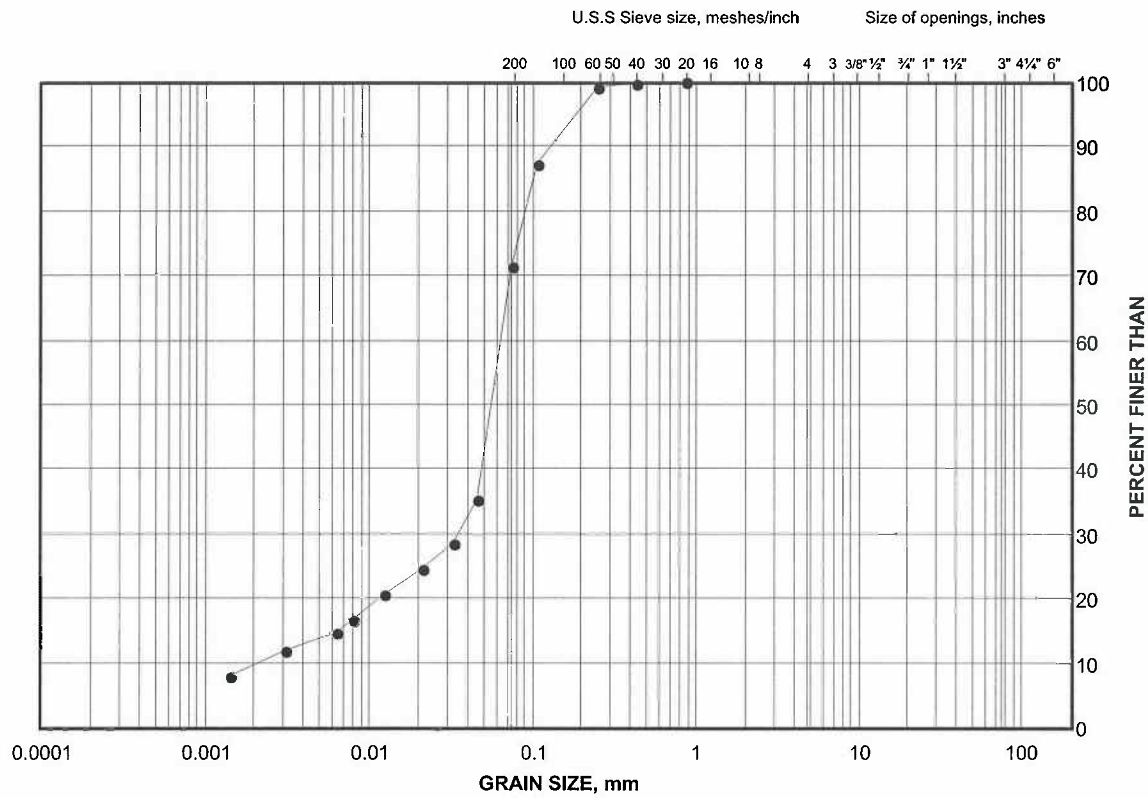
Project No. 05-1111-034

Checked By:

GRAIN SIZE DISTRIBUTION

Sandy Silt

FIGURE 10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-1	3	219.1

Project Number: 05-1111-034

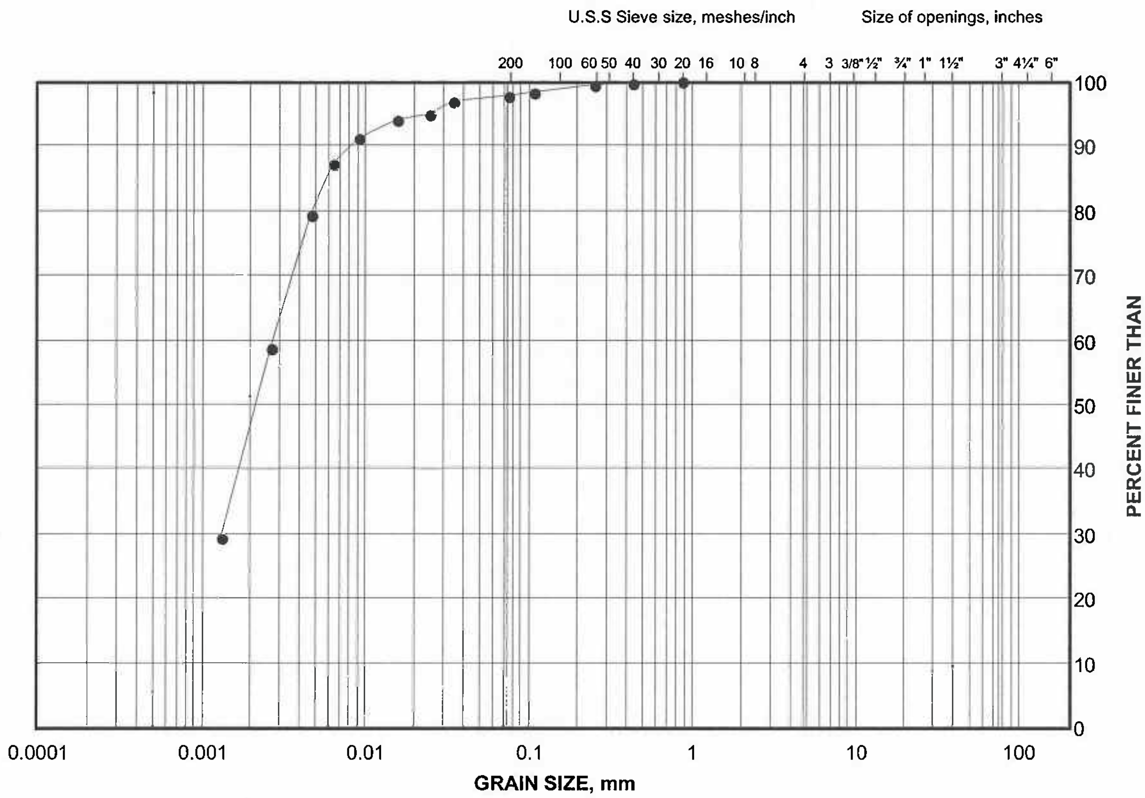
Checked By: Sen

Golder Associates

Date: 16-Sep-09

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 11



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	09-1	7	216.0

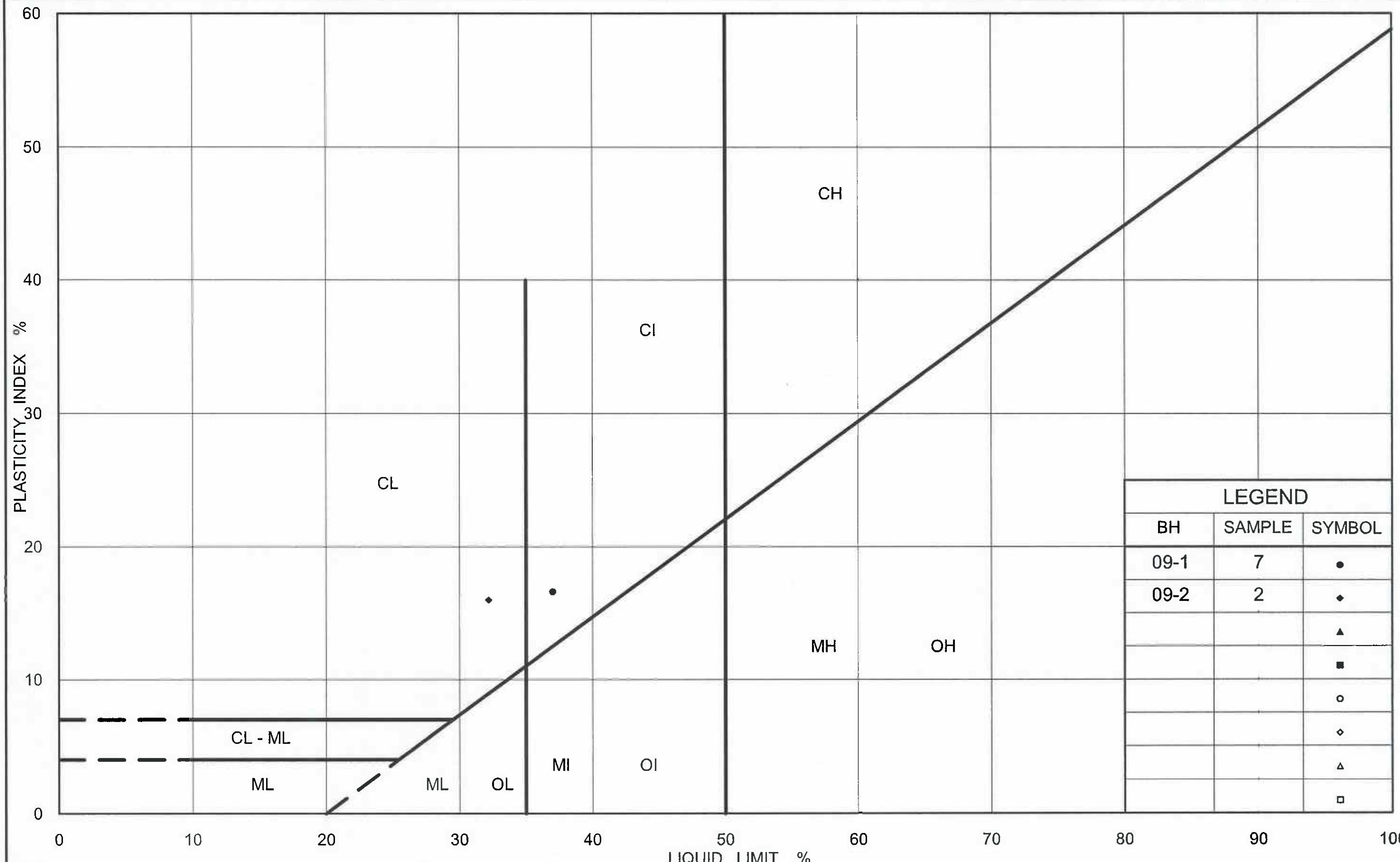
Project Number: 05-1111-034

Checked By: Sen

Golder Associates

Date: 16-Sep-09

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt and Silty Clay

Figure No. 12

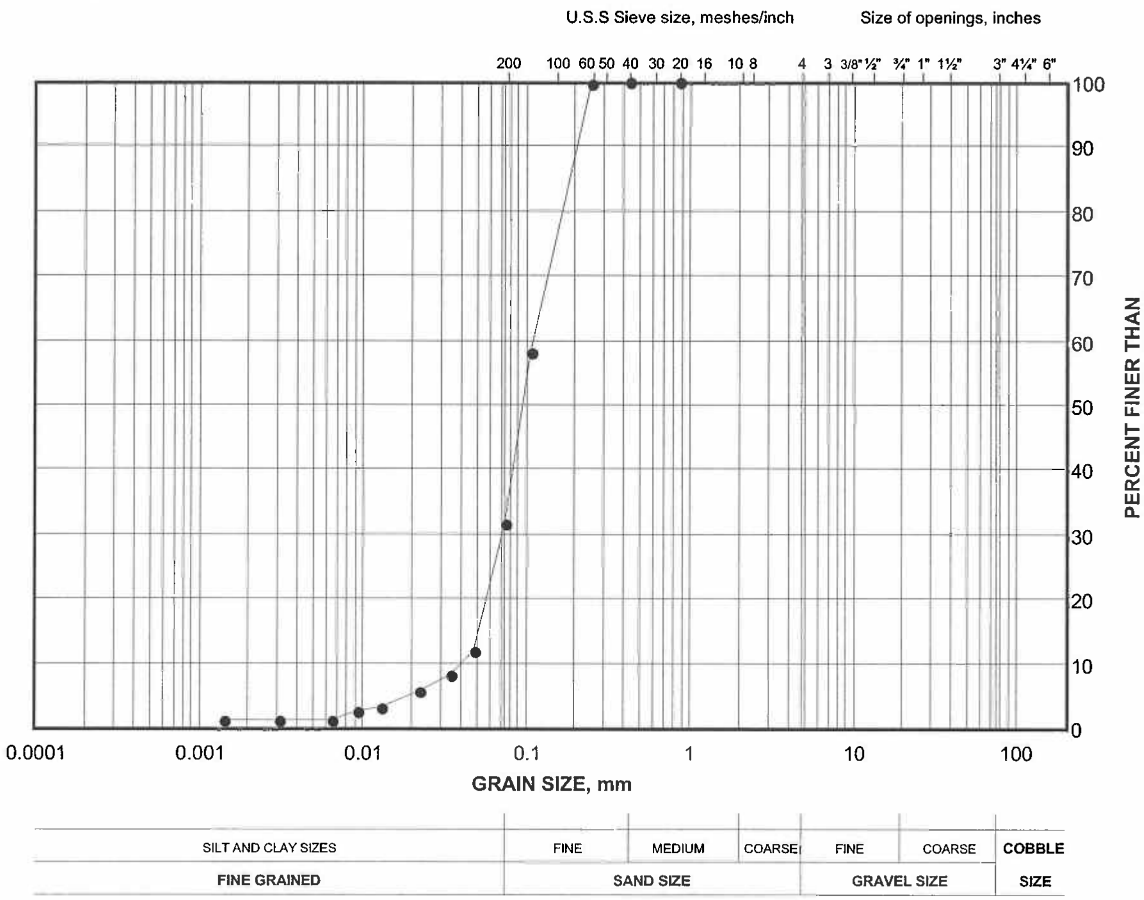
Project No. 05-1111-034

Checked By: [signature]

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE 13



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-1	10	211.5

Project Number: 05-1111-034

Checked By: Sen

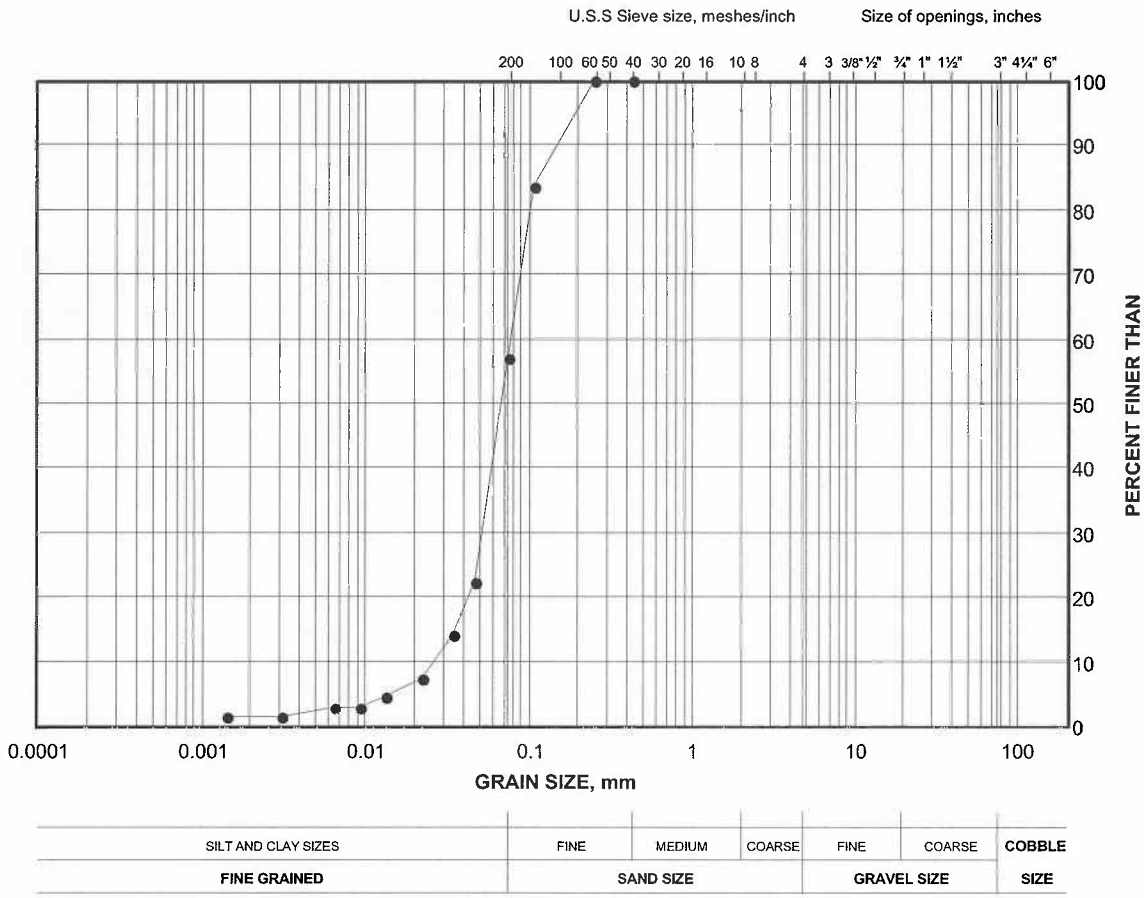
Golder Associates

Date: 16-Sep-09

GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE 14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-1	13	206.9

Project Number : 05-1111-034

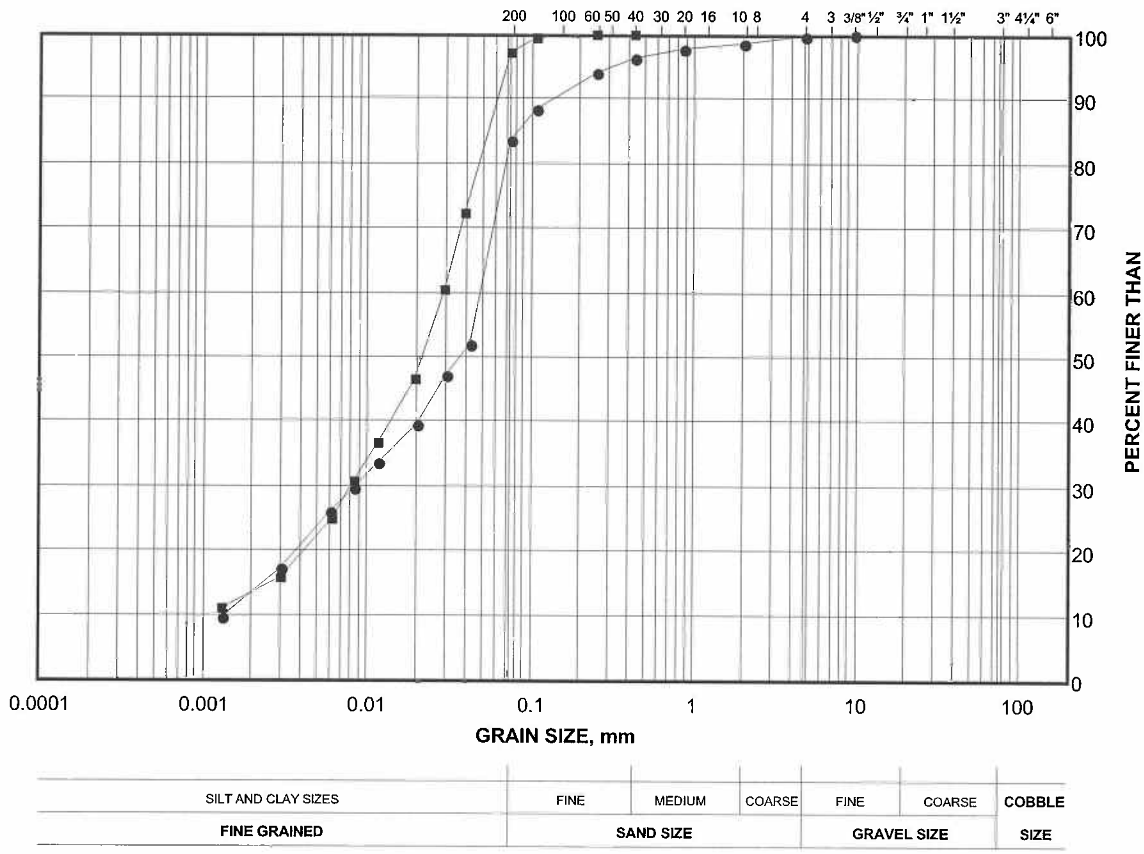
Checked By: Sen

Golder Associates

Date: 16-Sep-09

GRAIN SIZE DISTRIBUTION
Silt

FIGURE 15



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-2	3	209.9
■	09-2	7	206.8

Project Number : 05-1111-034

Checked By: Sen

Golder Associates

Date: 16-Sep-09

APPENDIX A

Borehole Records from the 1959 Investigation by The Department of Highways, Ontario Report No. 59-F-98 W.P. 218-59

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59 BORE HOLE NO. 1

JOB F59-98 STATION 291+50 (40'Lt)

DATUM 674.2' COMPILED BY B.K.

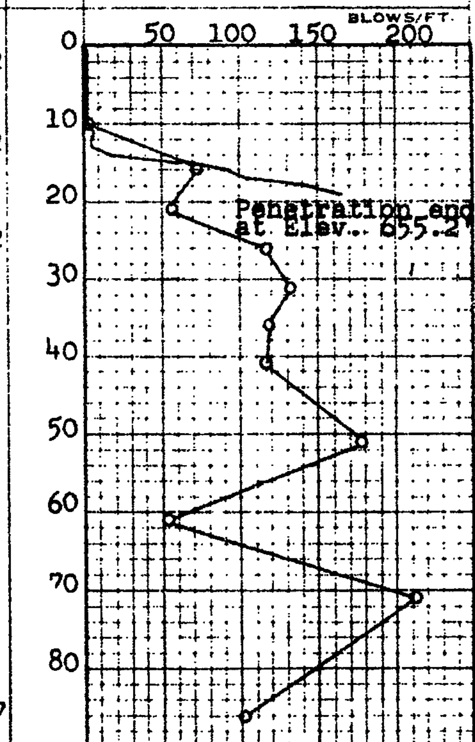
BORING DATE Oct. 6/58 CHECKED BY A.L.

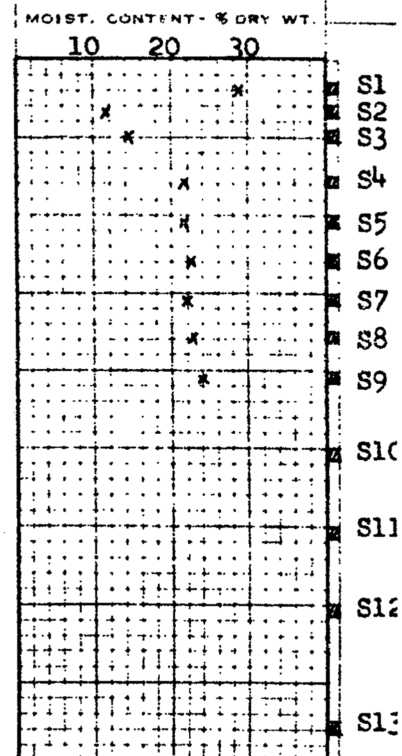
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET
	Ground level	674.2
	Loose brown fine silty sand with organic matter W.L.	671.2
		661.2
	Very dense grey clay silt (glacial till)	
		649.2
	Very dense grey silty clay and clay silt (glacial till)	
	End of borehole	587.7





Penetration resistance profile shown obtained by driving a 2" dia. cone from ground level noted with an energy of 350 ft. lb. per blow.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 218-59 BORE HOLE NO. 2
JOB F59-98 STATION 291+20 (90' Lt.)
DATUM 674.2' COMPILED BY B.K.
BORING DATE Oct. 9/59 CHECKED BY A.L.

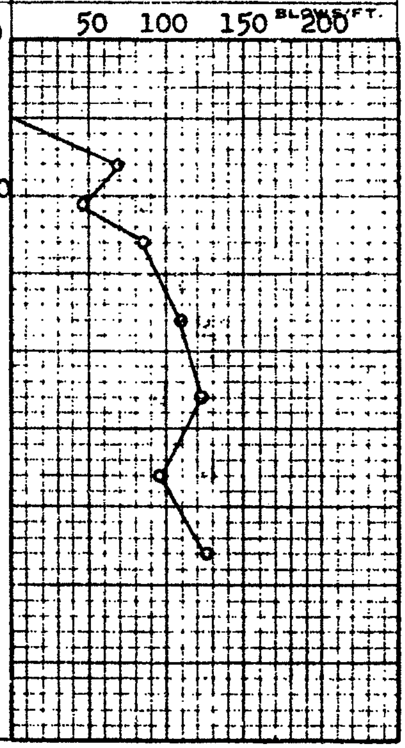
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

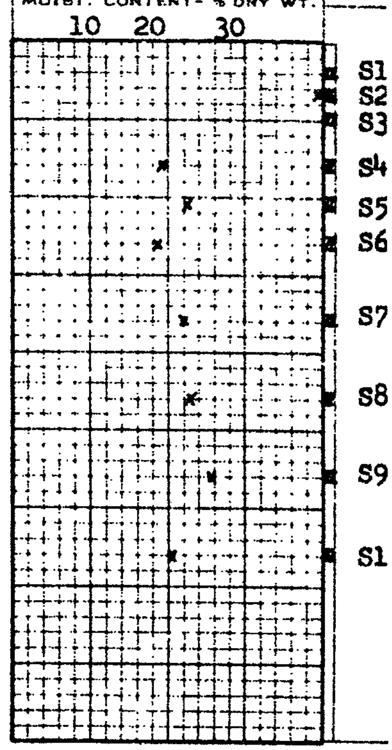
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Ground level	674.2	0
	Loose grey-brown fine silty sand with organic matter		10
		661.2	
	Very dense grey clay silt interbedded with silty clay layers (glacial till)		20
			30
			40
			50
			60
			70
			80

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT.



CONSISTENCY — MOIST. CONTENT - % DRY WT. — SAMPLE



Continued

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59
BORE HOLE NO. 2 cont'd.
JOB F 59-98
STATION 291+20 (70' LT)
DATUM 674.2
COMPILED BY B.K.
BORING DATE Oct. 9/59
CHECKED BY A.L.

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT.	CONSISTENCY / MOIST. CONTENT - % DRY WT.	SAMPL
	Borehole Continued	584.2	90			
	Same as described above		100			
		570.2				
	End of borehole		110			
			120			
			130			
			140			
			150			
			160			
			170			

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59
JOB F59-98
DATUM 671.5'
BORING DATE Oct. 13/59

BORE HOLE NO. 3
STATION 292+40 (50' Lt.)
COMPILED BY B.K.
CHECKED BY A.L.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

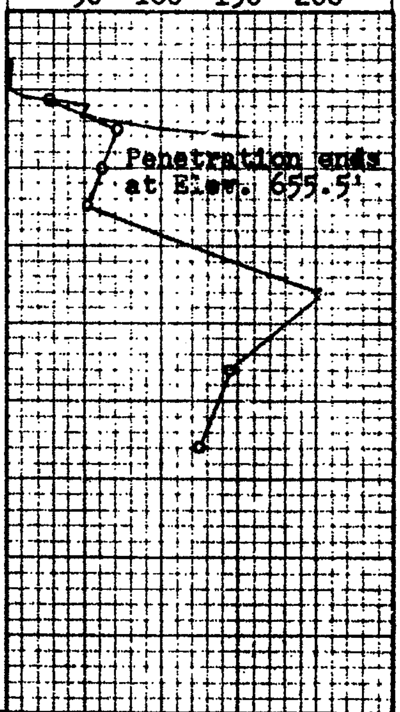
LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

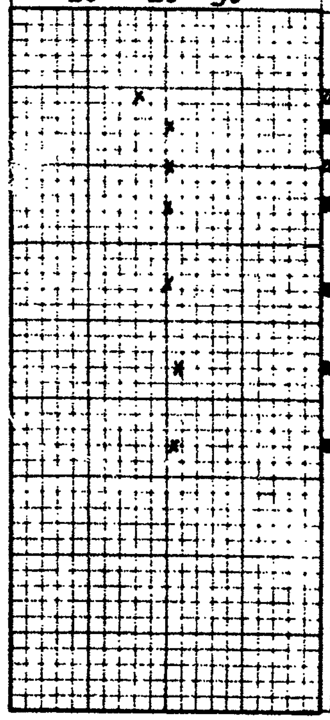
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓Water level	671.5	0
	Water	665.5	
	Medium coarse sand & fine gravel	659.5	10
	Very dense grey clay silt interbedded with silty clay layers (glacial till)		20
			30
			40
			50
			60
			70
	End of borehole	596.5	80

Penetration resistance profile shown obtained by driving a 2" dia. cone from ground level to depth noted with an energy of 350 ft. lb. per blow.

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT.



CONSISTENCY — MOIST. CONTENT - % DRY WT.



SAMPLE

S1
S2
S3
S4
S5
S6
S7

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59 BORE HOLE NO. 4

JOB F 59-98 STATION 292+40 (25'LT)

DATUM 671.5' COMPILED BY B.K.

BORING DATE Oct. 15/59 CHECKED BY A.L.

LEGEND

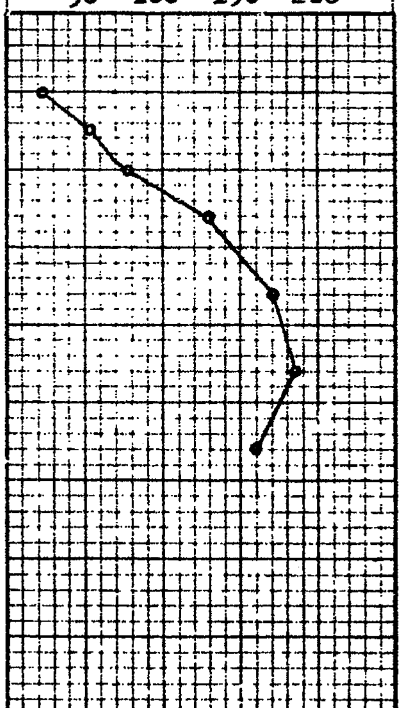
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Waterlevel	671.5	0
	Water	665.5	
	Medium Co. Sand & Fine Gravel	659.5	10
	Very dense grey clay Silt inter-bedded in the silty clay layers (glacial till).		20, 30, 40, 50, 60
	End of borehole	606.5	70, 80

STRENGTH AND PENETRATION RESISTANCE P.S.F.

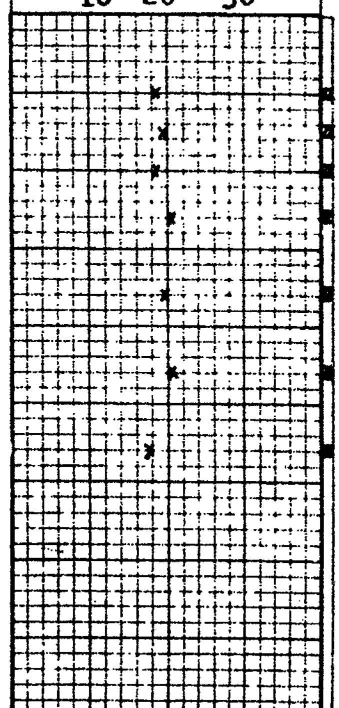
50 100 150 200 BLOWS/FT.



CONSISTENCY

MOIST. CONTENT - % DRY WT.

10 20 30



SAMPL

S1
S2
S3
S4
S5
S6
S7

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59　　BORE HOLE NO. 5

JOB F59-98　　STATION 293+30 (35' Lt)

DATUM 674.7'　　COMPILED BY B.K.

BORING DATE Oct.14/59　　CHECKED BY A.L.

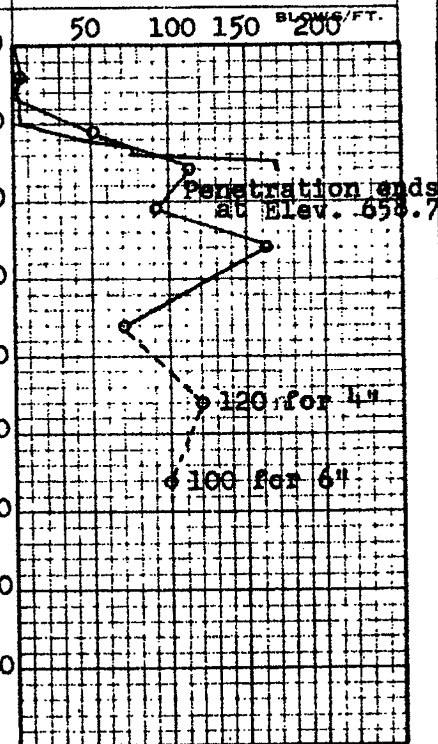
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

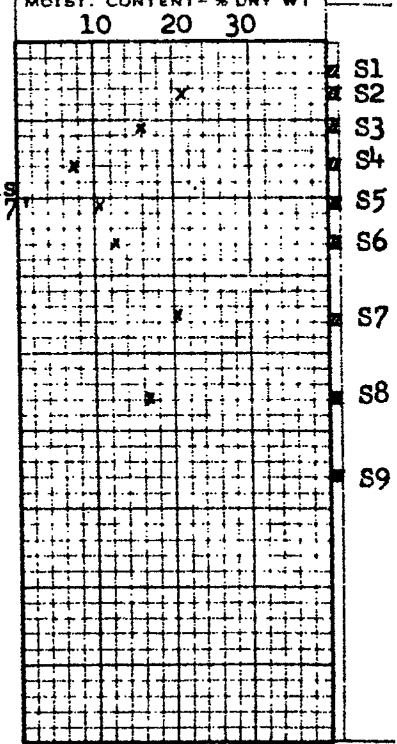
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓Ground level	674.7	0
	Loose grey-brown clayey sand with organic matter	670.2	
	Dense Sand and gravel inter-bedded with thin layers of clay	665.2	10
			20
	Very dense grey clay silt interbedded with layers of silty clay (glacial till)	648.7	30
			40
			50
			60
	End of borehole Penetration resistance profile shown obtained by driving a 2" dia cone from ground level to depth noted with an energy of 350 ft. lb. per blow.	609.7	70
			80

STRENGTH AND PENETRATION RESISTANCE　P.S.F.　BLOWS/FT.



CONSISTENCY　MOIST. CONTENT - % DRY WT　SAMPL



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 218-59 BORE HOLE NO. 6

JOB F 59-98 STATION 293+30 (60' LT).

DATUM 674.2' COMPILED BY B.K.

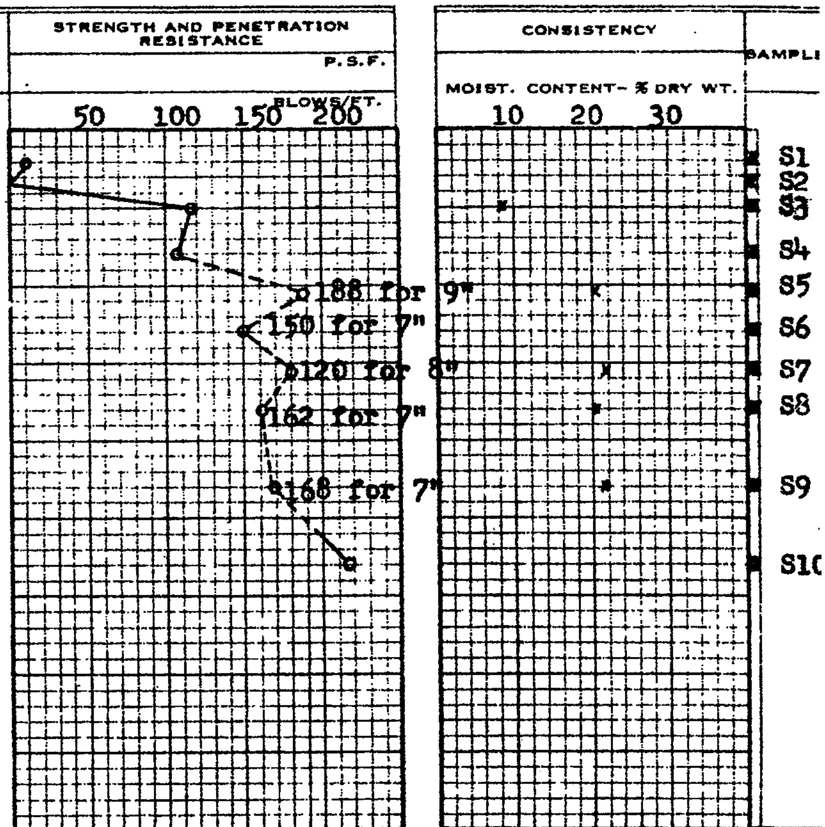
BORING DATE Oct. 19/59 CHECKED BY A.L.

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S).
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel Topsoil	674.2	0
	Loose brown clay sand with organic matter		
	Dense Sand & Gravel	666.7	10
	Very dense grey clay silt with traces of silty clay (glacial till).	656.7	20
			30
			40
			50
	End of borehole	619.2	60
			70
			80

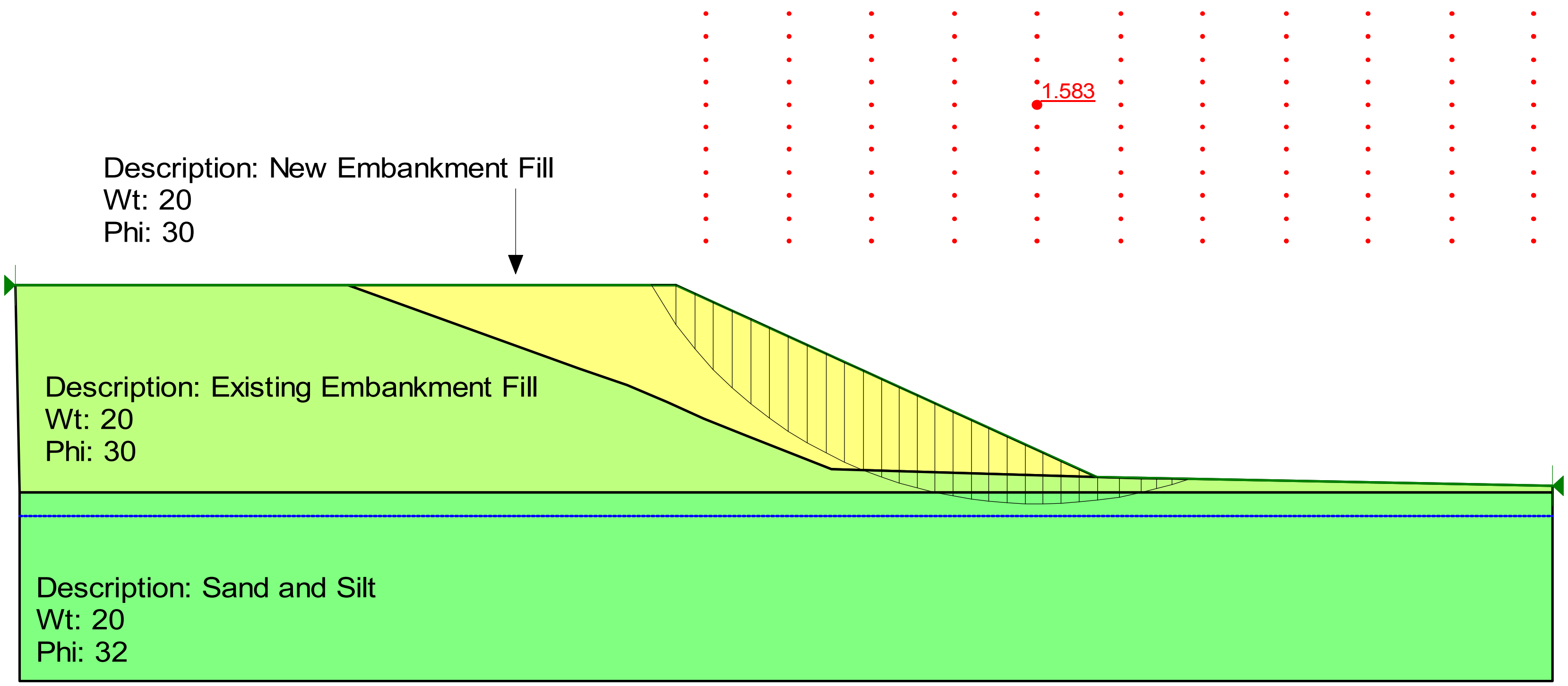


APPENDIX B

Results of Approach Embankment Stability Analyses

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
EMBANKMENT STABILITY ANALYSIS, STA. 16+020 TO STA. 16+085

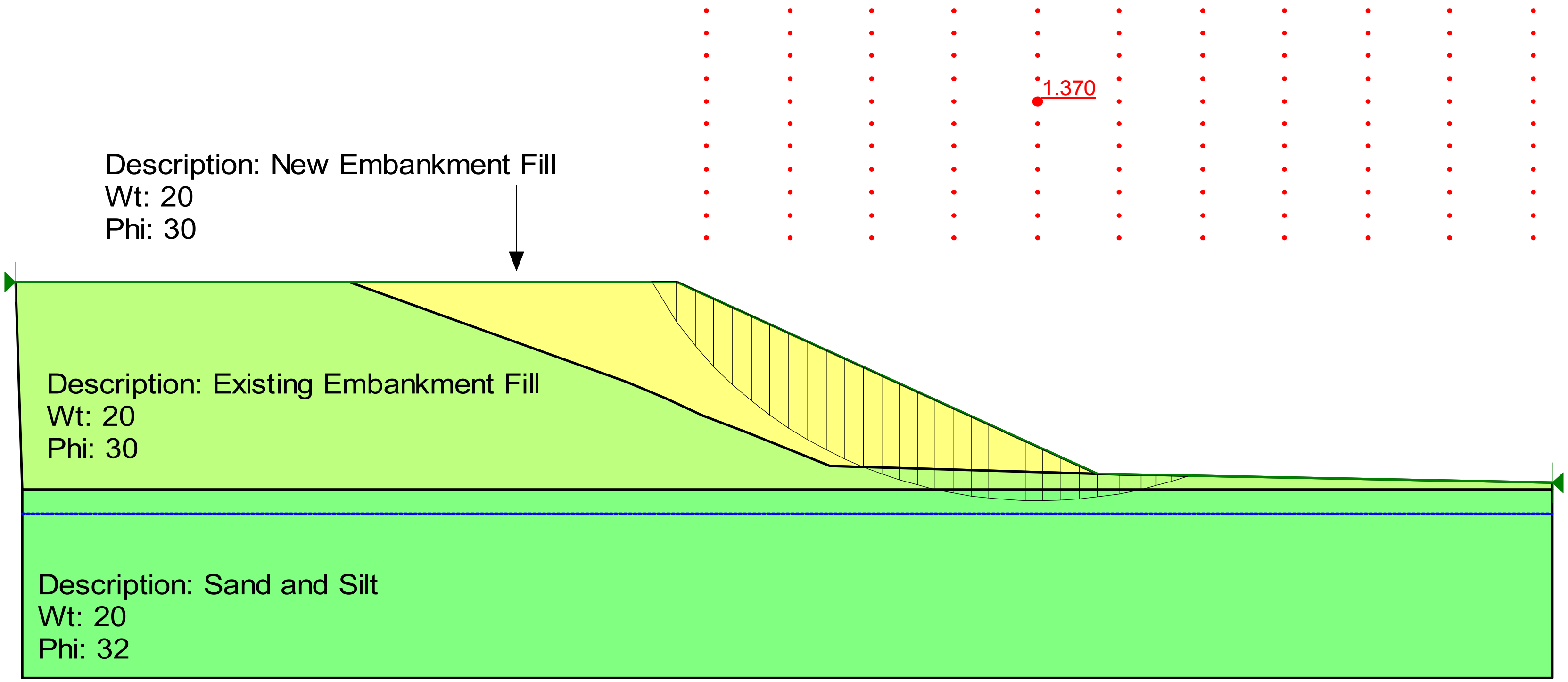
FIGURE B1



Date:	September 2007		Drawn:	NK
Project:	05-1111-034	**Golder Associates**	Checked:	HJ

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
EMBANKMENT STABILITY ANALYSIS, STA. 16+020 TO STA. 16+085
SEISMIC CONDTIONS

FIGURE B2



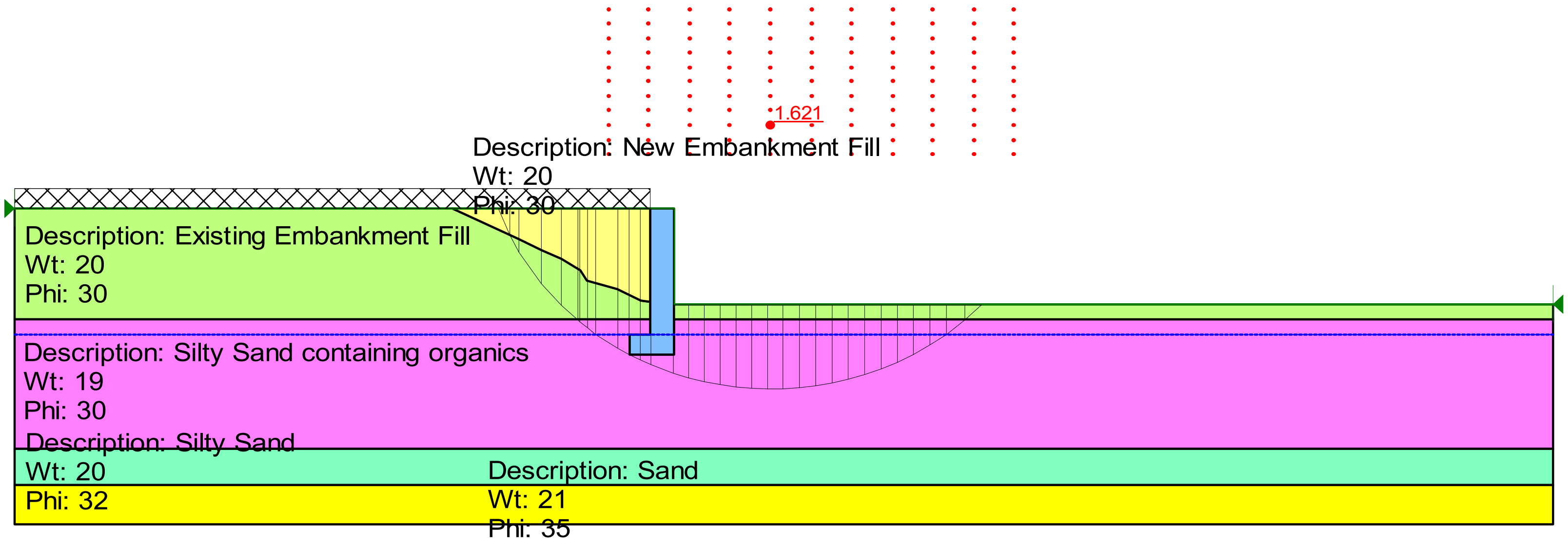
Date: September 2007
Project: 05-1111-034

Golder Associates

Drawn: NK
Checked: HJ

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
REATINING WALL GLOBAL STABILITY ANALYSIS - STA. 16+085 TO STA. 16+130

FIGURE B3



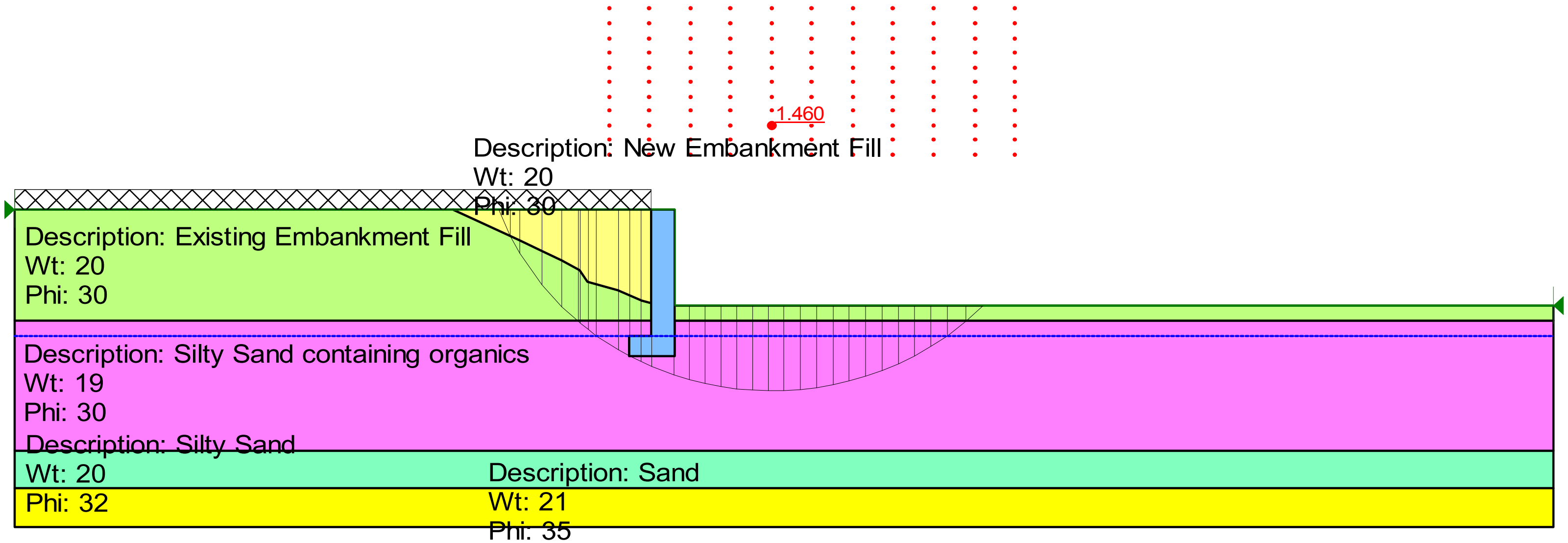
Date: September 2007
Project: 05-1111-034

Golder Associates

Drawn: NK
Checked: HJ

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
REATINING WALL GLOBAL STABILITY ANALYSIS - STA. 16+085 TO STA. 16+130
SEISMIC CONDITIONS

FIGURE B4



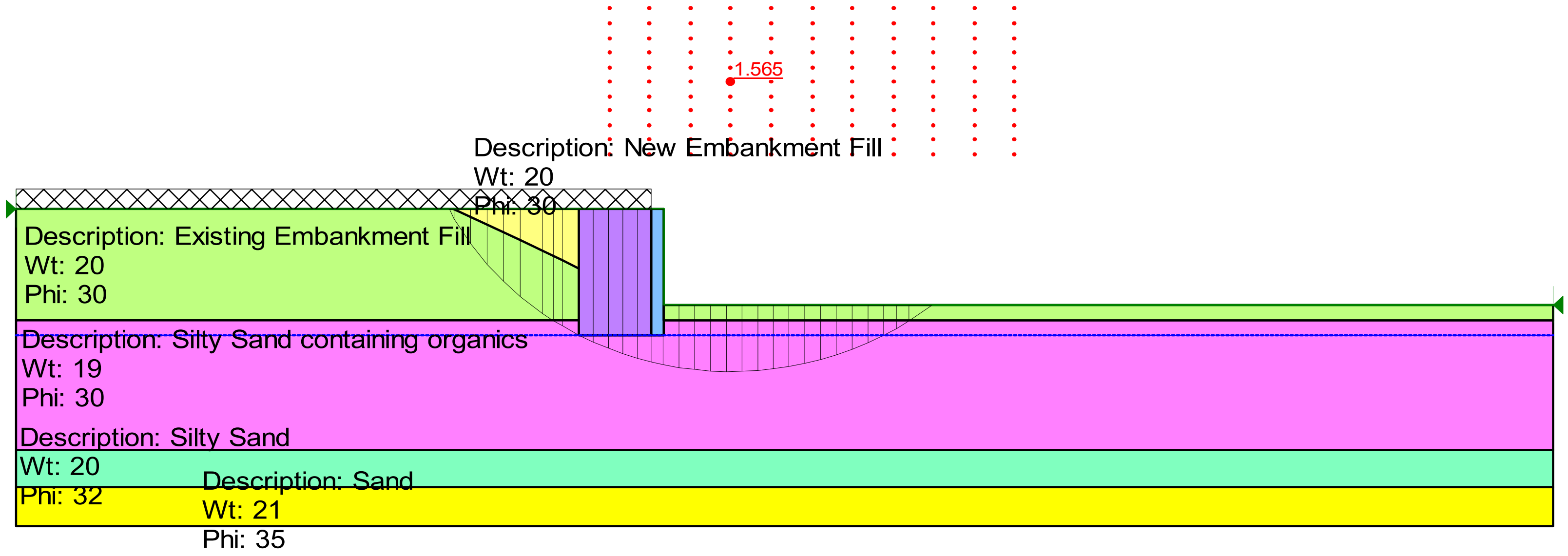
Date: September 2007
Project: 05-1111-034

Golder Associates

Drawn: NK
Checked: HJ

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
RSS WALL GLOBAL STABILITY ANALYSIS - STA. 16+085 TO STA. 16+130

FIGURE B5



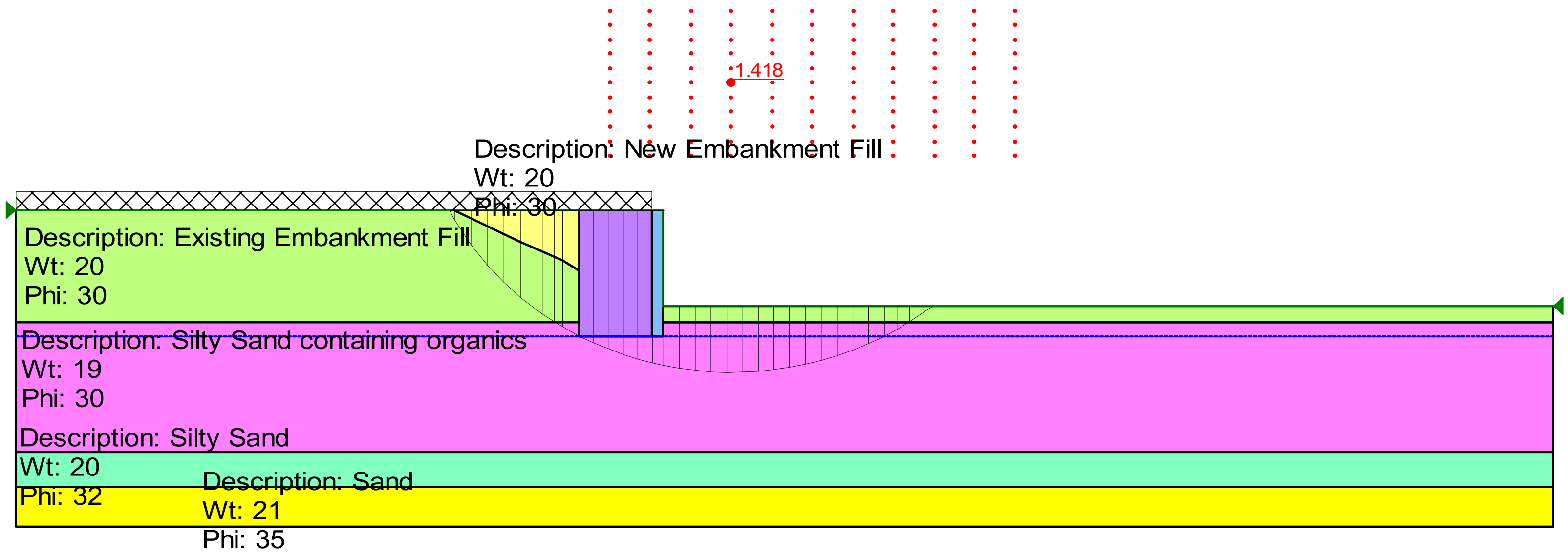
Date: September 2007
Project: 05-1111-034

Golder Associates

Drawn: NK
Checked: HJ

HWY 89 - NOTTAWASAGA RIVER BRIDGE WIDENING
RSS WALL GLOBAL STABILITY ANALYSIS - STA. 16+085 TO STA. 16+130
SEISMIC CONDITIONS

FIGURE B6



Date:	September 2007		Drawn:	NK
Project:	05-1111-034	**Golder Associates**	Checked:	HJ

APPENDIX C

Non-Standard Special Provisions

BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.

Special Provision

The soils at the site are may contain cobbles and/or boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) which may be are encountered during pile driving.

Basis of Payment

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

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during the piling installation works for the widening of the Highway 89 Bridge over the Nottawasaga River

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 2503-04-00:

- Foundation Investigation Report, Hwy 89 Nottawasaga River Bridge Rehabilitation/Widening & Retaining Wall and Cut Slope at the Intersection of Essa 5th Line and Hwy 89, Simcoe County, Ontario, G.W.P. 2503-04-00.

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 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 general conformance with the contract documents and issue certificate(s) of conformance.

Submission Requirements

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

Qualifications of vibrations monitoring specialist.
Proposed instrumentation.
Proposed location of instruments on existing Third Street overpass structure.
Proposed frequency of readings.
Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Monitoring

The vibration monitoring equipment shall be placed on the existing bridge structure, as close as possible to the piling works. The Contractor shall take readings on the existing structure during driving of each pile, starting with the pile furthest away from the existing structure for each widening area. As a minimum, the readings should be taken and recorded during the first 6 m of driving and during driving of the pile into the very dense/hard soil strata at depth.

The vibrations measured on the existing structure shall not exceed 50 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structure are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

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

END OF SECTION

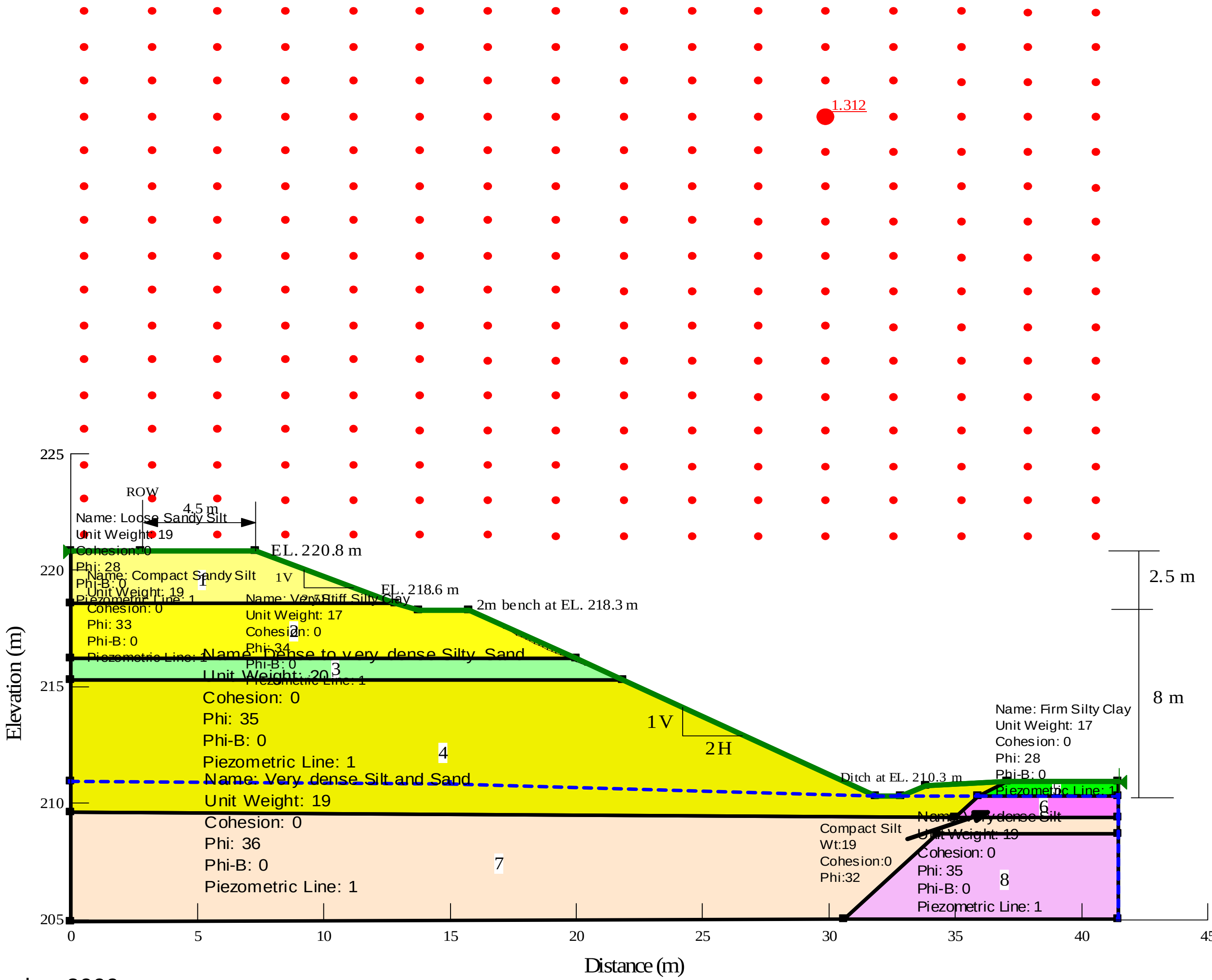
APPENDIX D

Results of Cut Slope Stability Analyses

PROPOSED NORTH CUT SLOPE AT STATION 16+325 - HIGHWAY 89
GLOBAL STABILITY - STATIC CONDITION
PROPOSED BENCH AT ELEVATION 218.3 m

FIGURE D1

Title: Title: 05-1111-034 Highway 89 Sta.16+325 Revised Slope (upper 2.5m 2.5H:1V normal)
Date: 11/09/2009



Date: September 2009
Project: 05-1111-034

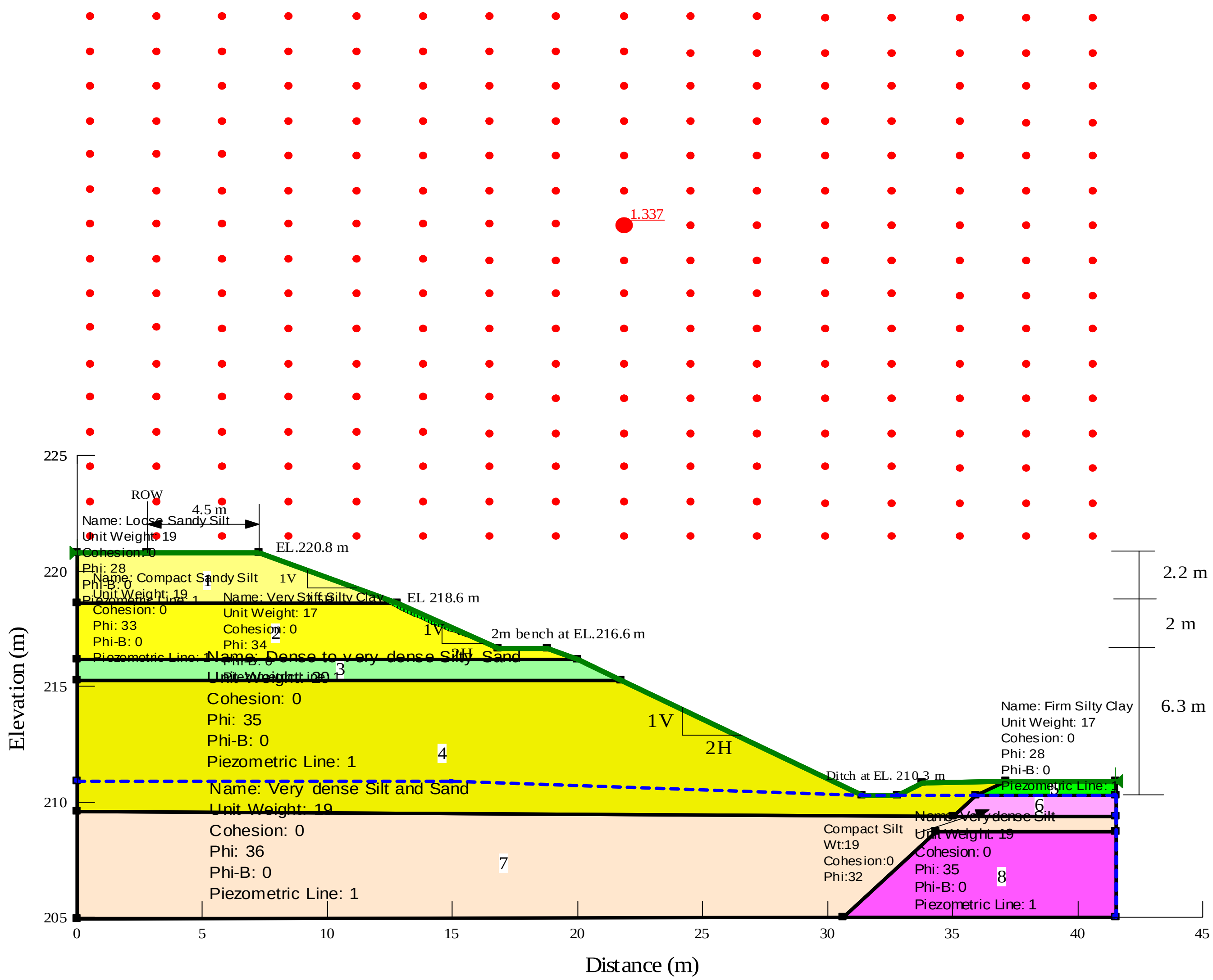
Golder Associates

Drawn: SH
Checked: JMAC

PROPOSED NORTH CUT SLOPE AT STATION 16+325 - HIGHWAY 89
GLOBAL STABILITY - STATIC CONDITION
PROPOSED BENCH AT ELEVATION 216.6 m

FIGURE D2

Title: 05-1111-034 Highway 89 Sta.16+325 Revised Slope (Upper 4.2 m 2.5to2H :1V normal
Date: 11/09/2009



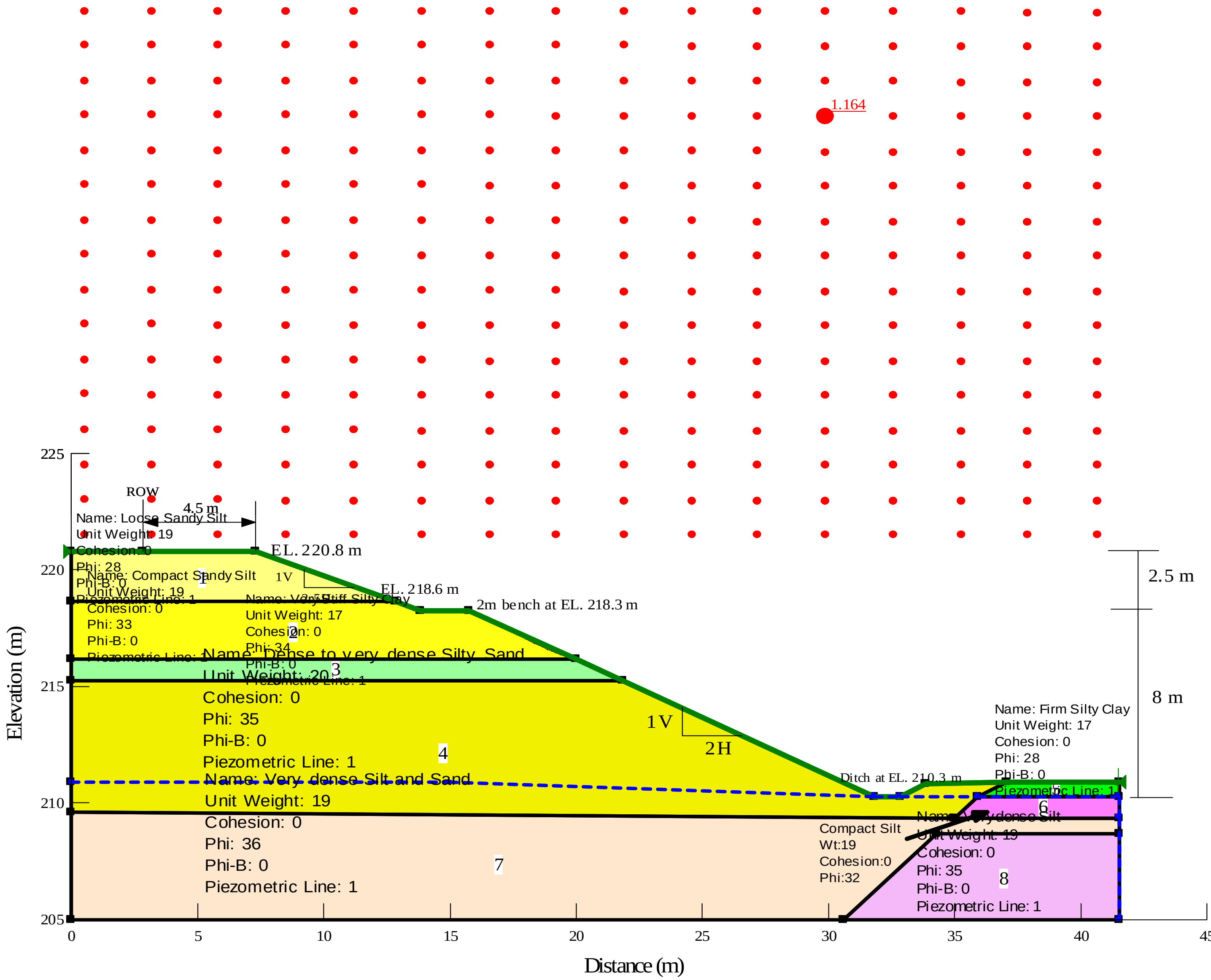
Date: September 2009
Project: 05-1111-034

Golder Associates

Drawn: SH
Checked: JMAC

PROPOSED NORTH CUT SLOPE AT STATION 16+325 - HIGHWAY 89
GLOBAL STABILITY - SEISMIC CONDITION
PROPOSED BENCH AT ELEVATION 218.3 m

FIGURE D3



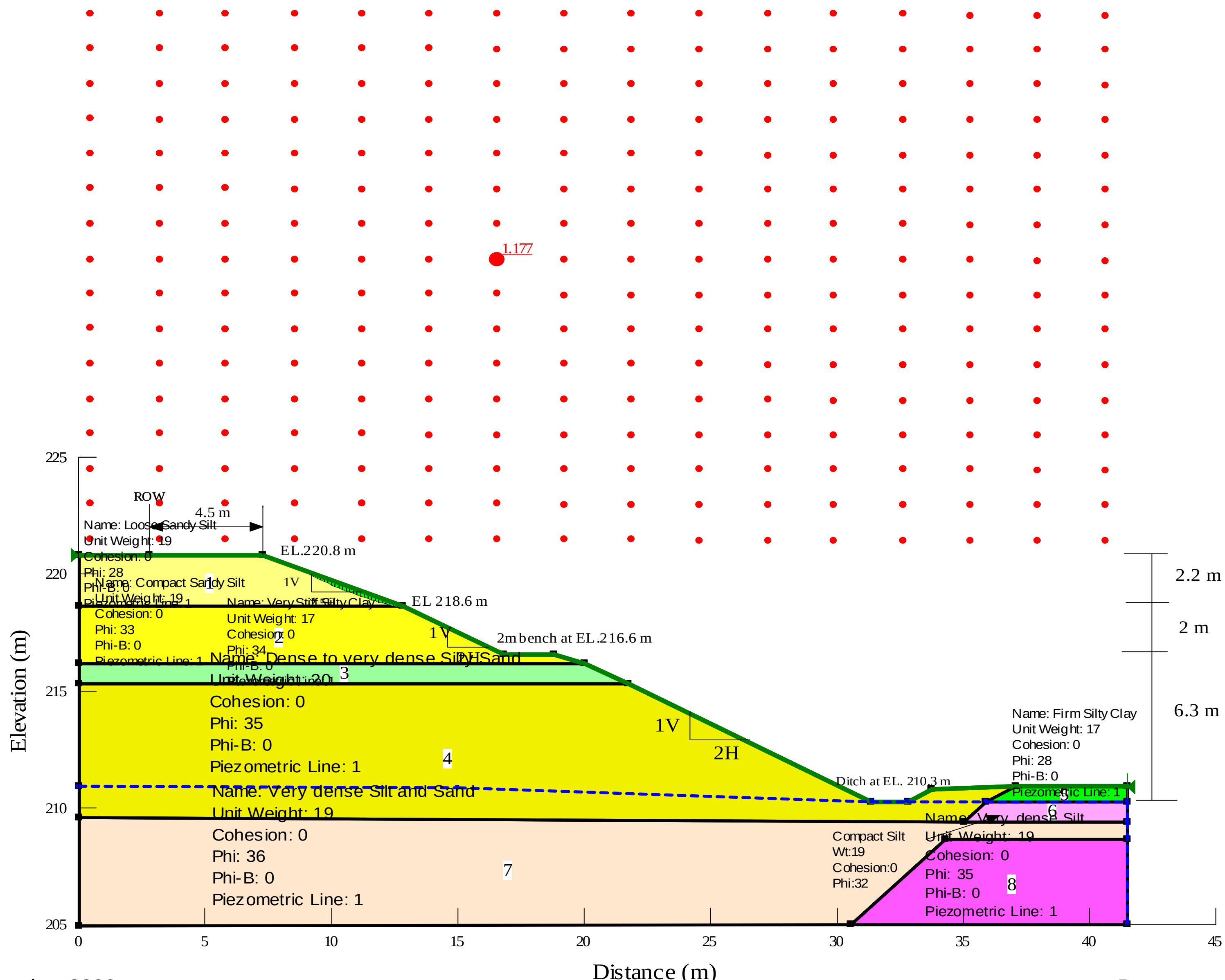
Date: September 2009
Project: 05-1111-034

Golder Associates

Drawn: SH
Checked: JMAC

PROPOSED NORTH CUT SLOPE AT STATION 16+325 - HIGHWAY 89
GLOBAL STABILITY - SEISMIC CONDITION
PROPOSED BENCH AT ELEVATION 216.6 m

FIGURE D4



Date: September 2009
Project: 05-1111-034

Golder Associates

Drawn: SH
Checked: JMAC

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

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North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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