November 2009

REVISED FOUNDATION INVESTIGATION AND DESIGN REPORT

BOAG ROAD OVERPASS - SBL STRUCTURE
HIGHWAY 404 EXTENSION FROM QUEENSVILLE SIDEROAD TO RAVENSHOE ROAD
TOWN OF EAST GWILLIMBURY
MTO W.P. 2005-07-00

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detailed design of the Highway 404-Boag Road overpass structure as part of the Highway 404 Extension project in East Gwillimbury, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P81-1069, dated February 22, 2008. The work was carried out in accordance with the Quality Control Plan for this project dated August 13, 2008.

The purpose of this investigation is to establish the subsurface conditions and shallow groundwater conditions at the proposed structure site by borehole drilling, in situ testing and laboratory testing on selected samples.

2.0 SITE DESCRIPTION

The site of the proposed overpass structure at Boag Road is located approximately 1 km west of Woodbine Avenue in the Town of East Gwillimbury in the Region of York (see key plan on Drawing 1). Boag Road, an east-west regional road, is currently a low volume, gravel surfaced two-lane road for eastbound and westbound traffic.

The overall surface topography in the area of the proposed overpass structure is generally flat-lying to gently sloping, with drumlins – elliptical "hills" formed by advancing glaciers during the last period of glaciation – present throughout the area. The existing Boag Road profile slopes downward from west of the proposed structure site to the east and flat open fields are located north and south of existing Boag Road alignment. The existing ground surface in the immediate vicinity of the proposed Boag Road structure site varies from about Elevation 240 m to 244 m and slopes downward to the southeast toward the Maskinonge River (located about 500 m east) where the ground surface at the river bank is at about Elevation 233 m. A drumlin is present to the north/northwest of the proposed structure site. This local topographic high has a maximum ground surface elevation of approximately 260 m.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the proposed Hwy 404 SBL Boag Road overpass was carried out between February 10 and 24, 2009, at which time six boreholes (Borehole BR-1 to BR-6) were advanced at approximately the locations shown on Drawing 1.

The field investigation was carried out using a track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced using 108 mm outside diameter solid stem augers (for the approach embankment holes) and 108 mm inside diameter hollow stem augers (for the abutment holes) to depths ranging from 5.2 m to 28.0 m below existing ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure.

The groundwater conditions in the open boreholes were observed throughout the drilling operations and piezometers were installed in Boreholes BR-2 and BR-3 to monitor the groundwater level at the site. The piezometers consist of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The boreholes and annulus surrounding the well pipe were backfilled to the surface with bentonite pellets in accordance with Ontario Regulation (O.Reg.) 903 as amended by O.Reg. 372/07 of the Ontario Water Resources Act. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A.

The field work was supervised on a full-time basis by a member of Golder's technical staff who arranged for service clearances, supervised the drilling, sampling and in-situ testing operations, logged the boreholes and examined and cared for the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg Limits and grain size distribution) was carried out on select soil samples. Organic content testing was carried out on one sample from borehole BR-2.

The borehole locations were surveyed in the field by J.D Barnes Ltd. prior to drilling operations. The as-drilled borehole locations (referenced to NAD83 MTM co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized below.

Borehole Number	Northing (m)	Easting (m)	Ground Surface Elevation (m)
BR-1	4893066.8	308889.1	242.7
BR-2	4893061.0	308899.6	242.3
BR-3	4893085.8	308888.5	243.5
BR-4	4893090.3	308900.0	242.8
BR-5	4893052.7	308894.9	241.7
BR-6	4893099.8	308894.7	243.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area for this investigation lies within or near two physiographic regions, delineated in *The Physiography of Southern Ontario*[1] *as*:

- Simcoe Lowlands; and
- Peterborough Drumlin Field

The surficial soils in the Simcoe Lowlands, to the south and southeast of Lake Simcoe, consist of sands, silts and clays that were deposited within a former glacial lake. It is noted that several areas of drumlinized till break the continuity of the Simcoe Lowlands plain.

The surficial soils in the Peterborough Drumlin Field consist of sandy drumlinized till. Some of the drumlins in this area have shallow coverings of silt and fine sand of thickness between about 0.5 m and 2.5 m. "Wave-washed" drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe. Localized deposits of silt, clay and peat are found in the low-lying areas between drumlins.

[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.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory tests are given on the Record of Borehole sheets and laboratory test plots provided in Appendices A and B, respectively.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes is shown on Drawings 1 and 2.

In summary, the subsoil conditions encountered at the site consist of a surficial layer of topsoil underlain by a relatively thin layer of sandy silt to silty sand, underlain by sand and silt till containing cobbles/boulders. Below the sand and silt till, a clayey silt till deposit is present. Layers of sand to silty sand are present within the native till deposits.

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

4.2.1 Topsoil / Clayey Silt containing Organics

Topsoil was encountered at the existing ground surface in all boreholes except for Borehole BR-2 where a surficial layer of clayey silt containing organics was penetrated. The topsoil and clayey silt layer ranges from 0.3 m to 1.0 m thick. It should be noted that boulders / cobbles were observed at the ground surface throughout the site.

Standard Penetration Test (SPT) ‘N’ values recorded within the topsoil and clayey silt containing organics range between 2 and 8 blows per 0.3 m of penetration, indicating a very soft to soft consistency. Drilling was performed during the winter months during which time the topsoil and near surface soils may have been frozen; therefore, SPT ‘N’ values may not be representative of thawed conditions.

A laboratory organic content test performed on a sample of the surficial clayey silt measured 2.6 percent organic content. The natural water content measured on a sample of the topsoil and clayey silt is 27 percent and 25 percent, respectively.

4.2.2 Sandy Silt to Silty Sand

A layer of sandy silt to silty sand was encountered immediately below the topsoil in Boreholes BR-1, BR-4 and BR-6 and ranges from 0.4 m to 1.1 m thick. This layer typically contains trace clay and gravel and contains rootlets / organics near the surface.

Standard Penetration Test (SPT) ‘N’ values recorded within the silty sand to sandy silt layer typically range between 2 and 14 blows per 0.3 m of penetration, indicating a very loose to compact relative density. An ‘N’ value of 44 was recorded from the ground surface in BR-1 which may be indicative of frozen ground conditions.

The natural water content measured on one sample of the silty sand to sandy silt is 14 percent.

4.2.3 Sand and Silt Till

Underlying the topsoil, clayey silt, and sandy silt to silty sand layer, a deposit of sand and silt till was encountered in all boreholes. The sand and silt till typically contains trace clay, trace to some gravel. Oxidation staining is also present within this layer indicating that the groundwater level may fluctuate throughout this layer. Cobbles and boulders are anticipated to be present throughout this layer as inferred by the grinding of augers as they advanced through the layer. The surface of the sand and silt till layer was encountered at depths ranging from 0.3 m to 1.5 m below ground surface (Elevation 243.2 m and 241.1 m) and is 3.2 m to 5.6 m thick.

Borehole BR-5 was terminated within the sand and silt till deposit at a depth of 5.2 m below ground surface (Elevation 236.5).

The measured SPT ‘N’ values within the sand and silt till deposit typically range from 14 blows per 0.3 m of penetration to 110 blows per 0.25 m of penetration, indicating a compact to very dense relative density. SPT ‘N’ values of 8 and 9 blows per 0.3 m of penetration were measured within the upper 2 m of this deposit at Borehole BR-5 and an ‘N’ value of 116 blows per 0.25 m of penetration was recorded in this borehole inferred to be on a boulder.

The results of seven grain size distribution tests performed on samples of the sand and silt till are shown on Figures B1 and B2. The measured natural water contents on samples of the sand and silt till range from 7 percent to 16 percent.

4.2.4 Silty Sand to Sand

Layers of sand, silty sand and sandy silt were encountered within and between the sand and silt till and clayey silt till deposits in Boreholes BR-1, BR-2 and BR-4. The top of the silty sand to sand layers were encountered at depths of 4.1 m (Elevation 238.6 m), 7.6 m (Elevation 234.7 m) and 12.2 m (Elevation 230.6 m) and the thickness of the layers range from 0.6 m to 1.9 m thick. The sandy silt to silty sand layers encountered at the bottom of Borehole BR-4 extended to the termination depth of 12.8 m (Elevation 230.0 m).

The measured SPT ‘N’ values within the silty sand to sand layers encountered in Boreholes BR-1, BR-2 and BR-4 range from 8 to 107 blows per 0.3 m of penetration indicating a loose to very dense relative density. The ‘N’ value of 8 may be the result of “blowing” sands during drilling operations and may not be representative of the in-situ relative density of the sand layer.

The result of a grain size distribution performed on a sample of the silty sand layer is shown on Figure B3. The measured natural water content taken on a sample of the silty sand from BR-2 is 16 percent.

4.2.5 Clayey Silt Till

A deposit of clayey silt till was encountered below the sand and silt till in all of the boreholes except BR-5 which did not extend into this deposit. The clayey silt till was encountered directly below the sand and silt till in Boreholes BR-2, BR-3, BR-4 and BR-6, and below a silty sand layer present below the sand and silt till in Borehole BR-1. The top of the clayey silt till deposit was encountered at depths ranging from 3.8 m to 7.8 m below ground surface (Elevation 239.7 m to 234.9 m) and contained trace to some sand, trace gravel. Interlayers and seams of sandy silt were present throughout the deposit. Boreholes BR-1, BR-2, BR-3 and BR-6 were terminated within this layer at depths ranging from 12.8 m to 28.0 m (Elevation 230.6 m to 214.7 m).

The measured SPT ‘N’ values within the clayey silt till range from 22 to 124 blows per 0.3 m of penetration and up to 151 blows per 0.28 m of penetration, indicating a very stiff to hard consistency.

The results of eight grain size distribution tests performed on samples of the clayey silt till are shown on Figures B4 and B5. Atterberg limits testing carried out on fourteen samples of the clayey silt till deposit measured liquid limits ranging from 20 to 25 percent, plastic limits ranging from 12 to 16 percent, and plasticity indices ranging from 6 to 10 percent. The results of the Atterberg limits testing are shown on Figure B6 and indicate that the material is a clayey silt till of low plasticity. The measured natural water contents range from 11 percent to 20 percent and was typically near the plastic limit.

4.2.6 Groundwater Conditions

Water levels were noted within the boreholes during and after the drilling operations. Piezometers were installed in Boreholes BR-2 and BR-3 to permit monitoring of the groundwater level. The piezometer installed in Borehole BR-2 was sealed within the silty sand to sand layer and the piezometer installed in BR-3 was sealed near the bottom of the borehole within the clayey silt deposit containing sandy silt interlayers. Details of the well installations are shown in the Record of Borehole sheets in Appendix A. The water levels recorded in the boreholes and piezometers are summarized below:

Borehole / Piezometer	Ground Surface Elevation (m)	Depth Below Ground Surface to Water Level (m)	Ground Water Level Elevation (m)	Date	Notes
BR-1	242.7	9.3	233.4	Feb. 19, 2009	Open Borehole
BR-2	242.3	6.4 1.4 1.3	235.9 240.9 241.0	Feb. 23, 2009 Mar. 12, 2009 May. 20, 2009	Open Borehole Piezometer Piezometer
BR-3	243.5	2.9 4.0 2.1 1.5	240.6 239.5 241.4 242.0	Feb. 11, 2009 Feb. 26, 2009 Mar. 12, 2009 May 20, 2009	Open Borehole Piezometer Piezometer Piezometer
BR-4	242.8	2.0	240.8	Feb. 13, 2009	Open Borehole
BR-5	241.7	5.2	236.5	Feb. 24, 2009	Open Borehole
BR-6	243.4	1.2	242.2	Feb. 13, 2009	Open Borehole

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. Artesian water conditions are present within the silty sand to sand layer encountered in Boreholes BR-2 and BR-3 and within the sandy silt interlayers present within the clayey silt till deposit based on the water levels observed during drilling and monitoring of subsequent water levels in the piezometers installed in BR-2 and BR-3.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Ted Beadle and reviewed by Mr. Kevin Bentley, P.Eng., a Senior Geotechnical Engineer with Golder. Mr. Jorge M.A. Costa, P.Eng., a Principal of Golder and a Designated MTO Contact for Foundations provided quality control review of this report for conformance with the project Terms of Reference.

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PART B

REVISED FOUNDATION DESIGN REPORT

BOAG ROAD OVERPASS – SBL STRUCTURE

HIGHWAY 404 EXTENSION FROM QUEENSVILLE SIDEROAD TO RAVENSHOE ROAD

TOWN OF EAST GWILLIMBURY

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations for the foundation aspects of the proposed Highway 404 SBL Boag Road overpass structure as part of the Highway 404 Extension project. The results of foundation investigation and detail design input for the NBL structure are provided in a separate report. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the General Arrangement drawing (dated March 16, 2009) provided to us, the proposed structure will be a single span reinforced concrete bridge with a total span length of 14.2 m. The proposed road grade of Hwy 404 at the proposed structure site ranges from about Elevation 246 m to 247 m. The proposed road grade of Boag Road is about Elevation 239 m and the existing road grade at Boag Road is at about Elevation 243 m; resulting in a permanent cut depth of about 4 m below existing grade. The configuration of the overpass involves lowering (cutting) the existing Boag Road and raising (filling) the approach embankment areas by up to 5 m above existing grade. The resulting north and south approach embankments will be constructed partially in cut and partially in fill and will be up to 7.5 m high above the proposed Boag Road grade.

The report includes minor revisions with references to lean concrete, pile verification procedure and the presence of cobbles and boulders in the subsoil, for consistency with the separate NBL Foundation Design Report.

6.2 Foundation Options

Various alternatives for the abutment foundations were considered and a summary of the advantages, disadvantages, relative costs and risks/consequences for each of the foundation options are summarised in Table 1.

Based on the subsurface and groundwater conditions, dewatering will be required to permit construction of shallow spread footings and pile caps if deep foundations are considered, in-the-dry. In general, it is recommended that the site be excavated to the proposed Boag Road level and the roadway profile and ditches be graded to allow sufficient time for the groundwater to drain (i.e. lower the groundwater level) prior to excavating for the abutment shallow foundations or for pile caps. This will reduce dewatering efforts and thereby reduce the potential for unstable base conditions within the foundation excavations.

Shallow foundations consisting of spread footings founded within the dense to very dense sand and silt till are considered suitable for support of the bridge structure, provided sufficient dewatering is carried out prior to excavation at the foundation footprint.

Steel H-piles are also appropriate, however the risks associated with penetrating through or the piles hanging up on cobbles / boulders within the till deposits, potential water seepage issues along the pile / soil interface, and potential for highly variable pile lengths within the footprint of each abutment location result in this alternative being less favourable and these issues would need to be considered in the design and construction of pile foundations.

Caissons are not considered to be a suitable option given the potential difficulties associated with the presence of cobbles and boulders within the upper sand and silt till layer, groundwater inflow / artesian conditions and potential for softening of the soils at the base of the caissons, and the highly variable subsurface conditions and inability to inspect the bottom of the caissons and confirm that "100-blow" material is present at the base of the caissons.

The following sections provide recommendations for both spread footing foundations and pile foundations to support the proposed structure. However, from a foundations perspective, the shallow foundation option is considered more practical for construction and is the preferred foundation alternative.

6.3 Spread Footings

Based on the General Arrangement drawing, the proposed founding elevation for the abutments is at about Elevation 237 m. It is our understanding that consideration is also being given to designing a continuous stepped (raised) footing to support the associated retaining (wing) walls at about Elevation 241 m.

Based on the subsurface conditions in the area of the proposed abutments, the abutments and associated retaining walls will be founded on the compact to very dense sand and silt till and very stiff to hard clayey silt till deposits. The south abutment, or sections of it, may also be founded on the dense to very dense silty sand to sand layers present within and between the till deposits.

For the retaining (wing) walls, subexcavation of the existing topsoil, clayey silt containing organics, silty sand to sandy silt, and sand and silt till up to 3 m below existing ground surface is required to reach the proposed founding soils at/below about Elevation 241 m. Any areas where the subgrade is located below the design founding elevation as a result of overexcavation to remove unsuitable materials can be raised with engineered fill comprised of OPSS 1010 Granular 'A' or Granular 'B' Type II placed in accordance with SP105S10 and SP902S01. In this case, the limits of engineered fill are defined by an area extending to at least 1 m beyond the outside edge of the founding level of any footing and then downward and outward at a slope of one horizontal to one vertical (1H:1V).

If the concrete for the footing on the native or engineered fill soil cannot be placed immediately after excavation, subgrade preparation and inspection, it is recommended that a working mat of lean concrete (mud mat) be placed to protect the integrity of the bearing stratum. In areas where the subgrade is located below the design founding elevation, lean concrete may be used to raise the subgrade instead of engineered fill as described above. A Non-Standard Special Provision should be included in the Contract Documents for use of lean concrete and an example is provided in Appendix C.

Subexcavation of the existing topsoil, clayey silt containing organics, silty sand to sandy silt, and sand and silt till soils to about 5 m to 6 m below the existing ground surface is required to reach the proposed founding level of Elevation 237 m. Cobbles and boulders are expected within the sand and silt till soils as discussed in Section 6.9.3.2. The groundwater table was measured in the piezometers at about 1.5 m below existing ground surface (Elevation 242 m), therefore, excavations will require dewatering to allow for construction of the foundations in the dry. For these reasons, it is recommended that conditions be included in the Contract requiring that the proposed Boag Road be excavated to the required grade and ditches be installed and sufficient time be allowed for the groundwater to be lowered in order to reduce the extent of staged excavation and dewatering efforts during foundation construction (refer to Section 6.9). A draft NSSP for dewatering has been provided in Appendix C.

If stepped spread footings are designed for the wing walls, the higher footing level should be located so that a 45 degree line from the bottom edge of the higher footing does not intersect any portion of the underside of the lower footing; otherwise, shoring may be required as per Section C6.7.1 of the *Canadian Highway Bridge Design*

Code (CHBDC) and its *Commentary*. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly.

6.3.1 Geotechnical Resistance

The following geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) may be used for the design of a 3 m to 4 m wide spread footing placed on the very dense sand and silt till, dense to very dense silty sand to sand, or on the very stiff to hard clayey silt till at or below Elevation 237 m for the abutment footings and at or below Elevation 241 m for the abutment wing walls.

Founding Element	Founding Soil	Founding Elevation	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm settlement)
North / South Abutment	Compact to Very Dense Sand and Silt Till / Very Stiff to Hard Clayey Silt Till / Dense to Very Dense Silty Sand to Sand	Below 241 m to 237 m	450 kPa	300 kPa

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the CHBDC and its Commentary.

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and the undisturbed subgrade soils should be calculated in accordance with Section 6.7.5 of the *Canadian Highway Bridge Design Code (CHBDC)*. The following summarises the coefficient of friction, tan δ, for the various interface materials.

Interface Materials	Coefficient of Friction (tan δ)
Cast-in-Place Concrete footing on Compact to very dense Sand and Silt Till / Dense to Very Dense Sand to Silty Sand	0.45
Cast-in-Place Concrete footing on very stiff to hard clayey silt till	0.40

These values represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

6.3.3 Frost Protection

All footings should be provided with a minimum 1.5 m of soil cover or equivalent thickness of insulation below the footings for frost protection. As a guide, 25 mm (1 inch) of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover.

6.4 Pile Foundations

Based on the subsurface conditions encountered in the area of the proposed abutments, steel H-piles should be driven to found within the hard ("100-blow") clayey silt till ranging from a depth of 10 m to 18 m below existing ground surface (Elevation 225 m to 232 m) at the north abutment location and ranging from 15 m to 25 m (Elevation 217 m to 227 m) at the south abutment location. The surface of the "100-blow" clayey silt till was typically encountered at a lower elevation on the west side of both abutments, and appears to slope downward from the north side to the south side of the proposed abutment locations. However, given the large variation in depth / elevation to the "100-blow" material within each foundation footprint, it is difficult to estimate the founding tip elevation for all of the piles. The following pile tip elevations may be used for design, assuming about 1 m penetration into the "100 blow" material:

Foundation Location	Design Pile Tip Elevation (m)	
	West Side	East Side
South Abutment	217 m	228 m
North Abutment	225 m	232 m

There should be provisions made in the contract for dealing with varying pile lengths. Given that the base of the pile cap will be at or below Elevation 237 m, pile driving through the sand and silt till containing cobbles / boulders is not anticipated to be an issue provided the area under the pile cap footprint is excavated to the underside of the pile cap level. However, removal of near surface cobbles / boulders may be required. For pile caps located higher than Elevation 237 m, pre-augering though the sand and silt till deposit containing cobbles/boulders may be required. Given the anticipated artesian groundwater conditions, specialized construction techniques will be required to mitigate the possible upward flow of water along the pile shaft. It is recommended that a sand filter, possibly in combination with a geotextile and drainage to the adjacent ditches, be placed beneath the pile caps to prevent the migration of fines that may be transported along the piles during and after construction.

6.4.1 Geotechnical Axial Resistance

For steel HP 310 x 110 piles driven to found within the hard "100-blow" clayey silt till at the elevations recommended above, the factored axial resistance at ULS may be taken as 1,500 kN and the geotechnical resistance at SLS for 25 mm of settlement may be taken as 1,000 kN.

The axial geotechnical resistance at this site will be obtained through a combination of the shaft friction along the pile length and resistance at the pile toe. However, considering that the piles are to be founded within the "100-blow" material, the majority of the resistance will be developed at the pile toe and therefore design axial resistance values are the same for the range of expected pile lengths.

Pile installation should be in accordance with SP903S01.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 3,000 kN. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.4 in accordance with Table 6.6.2.1 in the CHBDC (2006) to verify the factored ULS design value. Based on MTO experience with the Hiley formula in the Southern Ontario region, a resistance factor equal to 0.5 may be used for this project. The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation:

- "Piles to be driven in accordance with Standard SS-103-11 (Hiley Formula) using an ultimate capacity of 3,000 kN per pile, but should be driven to no higher than 1.5 m above the design pile tip elevations shown below for each abutment location:
    - South Abutment – West Side: Elevation 217 m
    - South Abutment – East Side: 228 m
    - North Abutment – West Side: Elevation 225 m
    - North Abutment – East Side: Elevation 232 m"

Assessment of ultimate pile resistance by the Hiley Formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley Formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation. An NSSP should be included in the Contract to address the pile capacity verification procedure and suggested wording is included in Appendix C.

Given the variable depth to the "100-blow" soils and corresponding founding elevations, it is recommended that the greater pile lengths be stipulated in the Contract Drawings for piles located between the west and east sides of the abutments to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley Formula on every pile installed. The pile termination or set criteria for the pile capacity selected 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.

Although not encountered within the clayey silt till during the current investigation, it is recommended that the H-piles be reinforced at the tip for driving piles through the sand and silt till which contains cobbles and boulders (i.e. the sand and silt till encountered above Elevation 236 m). The piles should be reinforced with flange plates as per OPSD 3000.100 or driving shoes such as Titus Standard "H" Bearing Pile Point design for protection during driving. An NSSP should be included in the Contract to address this requirement and suggested wording is included in Appendix C for reference.

6.4.2 Resistance to Lateral Loads

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

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$ where k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter / width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$ where k_h is the coefficient of horizontal subgrade reaction (MPa/m);
s_u is the undrained shear strength of the soil (MPa); and
B is the pile diameter / width (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the range in values reflects the variability in the subsurface conditions within each abutment footprint.

Structure	Soil Unit	n_h (MPa/m)	s_u (kPa)
South Abutment	Sand and Silt Till / Sand to Silty Sand El. 241 m to 232 m	10	-
	Clayey Silt Till Below El. 232 m	-	200
North Abutment	Sand and Silt Till El. 241 m to 237 m	5	-
	Clayey Silt Till Below El. 237 m	-	200

The maximum lateral resistance recommended for design for a single vertical HP310x110 pile driven to the design pile tip elevation is 120 kN at ULS and 50 kN at SLS for a lateral displacement of 10 mm at the pile head with reference to Section C6.8.7.1 of the Commentary to the CHBDC.

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

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

Pile Spacing in direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8D	1.0
6D	0.7
4D	0.4
3D	0.25

6.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.5 m of soil cover or equivalent thickness of insulation for frost protection. As a guide, 25 mm (1 inch) of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover.

6.5 Seismic Site Coefficient

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

6.6 Lateral Earth Pressures

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

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

- Select, free draining granular fill 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. Transverse 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 OPSD 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 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 the width equal to at least 1.5 m behind the back of the walls (Case I on Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case II on Figure C6.20(b) of the *Commentary* to the *CHBDC*).

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

	Earth Fill (Granular Material)
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
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 stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

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

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the *CHBDC*. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1. The site specific zonal acceleration ratio for Bradford is 0.05. For the thickness and competent overburden soils encountered at this site, an amplification factor of the ground motion is recommended for design (i.e. Site

Coefficient, S=1.2). As such, the recommended ground surface acceleration will increase to 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.03). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0.09). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = 2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design. These coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. 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 A	Case B	
	Earth Fill (Granular Material)	**Granular 'A'**	**Granular 'B' Type II**
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

Note : These *CHBDC* seismic K_{AE} values include the effect of wall friction (δ=Φ'/2) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

- 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:

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

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

6.7 Approach Embankment Design

The top of pavement at the proposed bridge approach embankments is about Elevation 246 m at the south approach and about Elevation 247 m at the north approach. The proposed road surface of Boag Road is about Elevation 239 m and the existing road surface at Boag Road is about 243 m, resulting in a permanent cut depth of about 4 m below existing grade. The configuration of the approach embankments involve lowering (cutting) the existing Boag Road and raising (filling) the approach embankment areas by up to 5 m above existing grade. The resulting north and south approach embankment slopes will be constructed in both cut and fill and will be up to 7.5 m high.

6.7.1 Stability

6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available software program "Slide", produced by Rocsciense Inc., employing the Morgenstern-Price method of analysis to check that a minimum Factor of Safety of 1.3 is achieved for global stability of the proposed approach embankment (i.e. up to 7.5 m high, comprised of both cut and fill such that the approach embankments will be up to 5 m above existing grade) and geometry under static conditions. This minimum Factor of Safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

6.7.1.2 Parameter Selection

The subsoils encountered in the area of the north and south approach embankments are generally composed of cohesionless soils underlain by cohesive clayey silt till. For the cohesionless soils, effective stress parameters were employed in the analyses assuming drained conditions. Undrained and drained conditions were modelled for the cohesive clayey silt till soils below the water table. The soil parameters were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT), visual classification and the results of laboratory testing. Granular earth fill embankments sloped at 2H:1V have been assumed. The piezometric conditions used in the analysis are based on the highest groundwater levels noted during drilling or measured in the piezometer installations.

The simplified soil stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach embankment areas is shown below.

Soil Layer	Bulk Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Cohesion (c') (kPa)	Effective Friction Angle (degrees)
Granular Fill	21	-	0	35
Compact to Very Dense Sand and Silt Till	21	-	0	35
Compact to Very Dense Silty Sand to Sand	20	-	0	30
Very Stiff to Hard Clayey Silt Till	21	200	0	32

6.7.1.3 Results of Analyses

The results of the analyses shown on Figures 1 and 2 indicate that a Factor of Safety greater than 1.3 was achieved for the proposed permanent slope configurations at the north and south approach embankment locations. The analyses assume that all topsoil or native soils containing organics has been removed from the proposed new embankment footprint and the new embankment fill is properly placed and compacted as per the following sections of this report.

6.7.1.4 Embankment Fill Types and Benching Requirements

For this project, locally available and/or imported, granular earth fill material that meets OPSS 1010 Select Subgrade Material (SSM) is considered suitable for use in construction of the approach embankments.

Given that the new north and south approach embankments are up to 7 m and 8 m high at the front slopes where the existing Boag Road will be cut, mid-height benches are not required.

6.7.1.5 Seismic Stability / Liquefaction Potential

Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of about 0.11g results in a Factor of Safety against side slope instability of 1.0. Based on the yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformation (i.e. less than about 5 mm) of the embankment would result under the design earthquake event.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio (CRR) of the soils with their normalised penetration resistance and fines content. Based on this assessment and assuming a ground surface acceleration of 0.06g, the subsoils are not considered liquefiable for an earthquake magnitude of 7.0.

6.7.2 Settlement

6.7.2.1 Methodology

Analyses were performed using the commercially available software program “Settle3D” produced by Rocscience Inc. and hand calculations to estimate the settlement of the foundation soils underlying the proposed approach embankments. The maximum fill thickness modelled at the south and north approach embankments was 5 m and 4 m respectively. A bulk unit weight of 21 kN/m^3 was employed for the granular fill in calculating the embankment loading on the subsoils.

6.7.2.2 Parameter Selection

Static settlement analyses were carried out for the foundation soils using the following parameters based on field and laboratory test data and accepted correlations as proposed by Kulhawy and Mayne (1990):

Approach Embankment	Soil Layer	Thickness (m)	Bulk Unit Weight (kN/m^3)	Estimated Deformation Properties
South	Compact to Very Dense Sand and Silt Till	6	21	E' = 50 MPa
	Compact to Very Dense Sand to Silty Sand Layers	2	20	E' = 40 MPa
	Very Stiff to Hard Clayey Silt Till	10	21	E' = 50 MPa C_r = 0.02
North	Compact to Very Dense Sand and Silt Till	3	21	E' = 50 MPa
	Very Stiff to Hard Clayey Silt Till	10	21	E' = 50 MPa C_r = 0.02

The analyses were carried out for both the north and south approach embankments and assume that all organic / very loose surficial soils have been removed prior to embankment fill placement.

6.7.2.3 Results of Analyses

The results of the analyses indicate a total settlement of up to 50 mm below the south approach embankment and up to 25 mm would occur below the north approach embankment for the 5 m high and 4 m high embankments, respectively. These settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to filling based on the cohesionless nature of the sand and silt till deposit and heavily overconsolidated nature of the very stiff to hard clayey silt tills which also contain silty sand seams / interlayers.

6.7.2.4 Settlement of New Embankment Fill

Provided that the embankment fill material consists of suitable fill meeting the requirements of and placed and compacted in accordance with SP206S03, the settlement of the new embankment fill itself is expected to be less than 25 mm, and the majority of settlement will occur during or shortly after construction.

6.8 Subgrade Preparation and Embankment Construction

6.8.1 Removal of Organics

Based on the borehole results, layers of topsoil or clayey silt / sandy silt containing organics up to about 1 m thick are present at the ground surface within the proposed embankment footprints. Prior to the placement of any fill for the new approach embankment construction, all topsoil and clayey silt / sandy silt containing organics should be stripped from below the proposed approach embankment areas in accordance with SP206S03.

6.8.2 Embankment Fill Placement

The exposed subgrade soils should be inspected prior to placement of embankment fill, proofrolled to identify soft / loosened areas, and any poorly performing areas should be subexcavated and replaced with suitable backfill.

Construction of the embankment or backfill for any poorly performing areas may be carried out using Select Subgrade Material (SSM) meeting the specifications of OPSS 1010.

Embankment fill should be placed in accordance with Special Provision SP206S03. The final lift prior to placement of the granular subbase and base courses should be compacted to at least 100 per cent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding or pegged sod should be placed as soon as possible in accordance with OPSS 572. If this protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting, is recommended to reduce the potential for remedial works being required on the side slopes in the Spring prior to topsoil and seeding.

6.9 Design and Construction Considerations

6.9.1 Excavation

The proposed Boag Road profile will be lowered up to about 4 m below existing grade at the location of the structure and will slope downward from west to east towards the Maskinonge River located about 500 m east of the structure location. The existing groundwater level at the site at the time of the subsurface investigation is about 1.3 m below ground surface. It is recommended that the proposed Boag Road profile be excavated from east to west, ditches and sub-drains be installed and groundwater allowed to drain during grade lowering and in advance of construction of the structure foundations to reduce dewatering efforts and reduce the risk for basal heave or disturbance / softening of subgrade soils during the excavation operations.

Permanent excavations for the grade lowering will be made through the surficial topsoil and road fills, typically compact to very dense sandy silts, silty sands and sands, compact to very dense sand and silt till, and very stiff to hard clayey silt till. Provided the road cut is allowed sufficient time to drain, permanent side-slopes no steeper than 2H:1V are considered adequate for the excavation provided a contingency/allowance for slope protection using gravel sheeting (minimum 0.5 m thick) is included in the Contract. Based on the subsurface information in the area of the proposed structure, localized zones of seepage and surficial instability may be encountered due to the high water table and presence of sandy layers within the till deposits.

Depending on the foundation option adopted, excavation for the bridge foundations (including wing wall foundations) will extend to depths of up to 4 m below proposed ground surface (up to 6 m below existing ground surface) to about Elevation 237 m. Temporary excavations will be made through the topsoil, typically compact to very dense sandy silts, silty sands and sands, compact to very dense sand and silt till, and very stiff to hard clayey silt till. The surficial soils (upper metre) are considered Type 3 soils and the remaining soils (above the water level) are considered Type 2 soil according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). As such, temporary excavations should be carried out with walls sloped to within 1.2 m of the bottom with a slope having a minimum gradient of 1 horizontal to 1 vertical (1H:1V) provided dewatering is achieved to below the base of the excavation. If dewatering is not implemented or if sufficient time is not allowed for the native soils to drain during the Boag Road grade cut, temporary side-slopes for the road cut and foundations excavations should be no steeper than 2.5H:1V.

All excavations must be carried out in accordance with the latest edition of the OHSA.

6.9.2 Basal Heave

Temporary subexcavations during foundation construction should be maintained as shallow as possible to reduce the risk of basal heave or unstable founding soil conditions within the clayey silt till / sand and silt till soils which contain interlayers of saturated silty sand to sand under artesian conditions. Given the artesian water conditions encountered within the silty sand to sand layers within the clayey silt tills, design calculations indicate that subexcavation to about Elevation 240 m can be carried out with an adequate factor of safety equal to 1.5 against base heave. However, subexcavation to about Elevation 237 m is required for the spread footings or pile cap and subexcavation to about Elevation 239 m is required for the proposed Boag Road profile cut in general. As a result, dewatering measures are required to prevent basal heave / softening of the founding soil conditions for both permanent and temporary excavations at the site. It is anticipated that grading for the Boag Road profile will begin prior to foundation excavation at the east side (near the Maskinonge River) and progress westward such that the excavated grade is sloped to drain groundwater towards the east. Consideration should be given to excavating a “pilot” trench along the proposed cut in order to drain the groundwater in sufficient time prior to excavation. This construction procedure should allow for confined / artesian groundwater conditions within the sandy interlayers ahead of the excavation to dissipate and drain as the existing Boag Road is lowered to the design profile.

Provided sufficient time is allowed for the groundwater to drain and stabilize, the anticipated groundwater level at the Boag Road structure site directly adjacent to the Boag Road cut will be at or below Elevation 239 m and will rise to the current water level of 242 m at a sufficient distance away from the cut grade. Temporary excavations to about Elevation 237 m could be carried out with a Factor of Safety equal to 1.2 against base heave directly adjacent to the roadway cut. However, excavations made within the foundation footprint where the water level is higher are susceptible to basal heave as the till deposit is less than 1 m thick in some areas and the present groundwater level is about 5 m above the proposed base of the excavation such that the groundwater pressures will need to be lowered to a level at least 0.5 m below the proposed founding elevation in order to protect the foundation subgrade. At locations where sandy layers are present (specifically at the south abutment) at or near the foundation subgrade level, it is expected that “boiling” (loosening of sandy soils due to unbalanced water pressures) as opposed to basal heave could also be an issue; as a result, groundwater control prior to the foundation subexcavations will be required as discussed in the next section.

6.9.3 Control of Groundwater and Surface Water

The groundwater level was measured to be as high as 1.3 m below existing ground surface (Elevation 242 m) at the foundation locations. As previously noted, provided the new Boag Road profile is excavated and allowed sufficient time to drain, local groundwater levels are expected to be lowered to below Elevation 239 m adjacent to the depressed roadway. It is anticipated the actual groundwater levels within the foundation footprint will range between about Elevation 239 m and 242 m, resulting in the base of temporary excavations being from 2 m to 5 m below groundwater level during construction.

Given the relative density and grain size distribution for the silt and sand till and clayey silt till in these areas, and the results of hydraulic conductivity testing performed within the sandy layer in BR-2 as part of the foundation hydrogeology investigation, it is considered that a combination of adequately sized pumped pressure relief wells and perimeter ditches / trenches (possibly built in combination with the proposed Boag Road permanent cut ditches / sub-drains) will be required to lower the groundwater level to at least 0.5 m below the founding level (i.e. Elevation 236.5 m) within the foundation footprint. The wells or the closest shoulder of trenches / ditches should be located at least 1 m beyond the outside limits of the foundation footprint so as not to disturb the founding subsoils or disrupt construction procedures at the foundation locations.

Alternatively, consideration could be given to the use of a sheetpile box configuration driven to sufficient depth to control groundwater pressures. However, such a method of groundwater control is less desirable as the dense nature of the sand and silt till deposit and the presence of cobbles / boulders within the till could lead to the sheetpiles to “hang up” on the obstructions and not provide a watertight seal.

Surface water should be directed away from the excavations at all times.

6.9.3.1 Permit to Take Water

A drawdown/seepage analysis has been carried out to estimate the volume(s) of groundwater flow that may have to be pumped at the north and south abutment locations in order to lower and maintain the groundwater level below the base of the excavations during spread footing or pile cap construction for the abutments. Based on upper bound estimates of hydraulic conductivity (k) for the soil strata at and below the base of the proposed excavated areas (from in-situ testing at the well locations), it is anticipated that groundwater pumping volumes greater than 50,000 litres/day at each abutment location will not be required. Therefore, a Permit to Take Water (PTTW) will not be required. The actual pumping volumes could increase depending on weather (i.e. precipitation), time of construction (i.e. snow melt) and construction methodology employed by the Contractor.

6.9.3.2 Obstructions

The native sand and silt till soils at the site contain cobbles and boulders as was observed on the ground surface and inferred from grinding of and refusal to advancement of the augers during borehole drilling.

Conventional excavation equipment should be suitable for the majority of the excavation through the subsoils on site. However, the presence of boulders may interfere with or slow the progress of stripping and excavation. The presence of such obstructions may also affect the excavation works and the installation of piles (depending on the pile cap level) if adopted for foundation design. It is recommended that a Non Standard Special Provision (NSSP), be included in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions. An example NSSP is included in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Ted Beadle, and reviewed by Mr. Kevin J. Bentley, P.Eng., a Senior Geotechnical Engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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TABLE 1
EVALUATION OF ABUTMENT FOUNDATION ALTERNATIVES
Boag Road Overpass – SBL Structure / Highway 404 Extension
W.P. 2005-07-00

Footing Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*[1]	*Risks/Consequences*
Spread Footings founded on compact to very dense sand and silt till / dense to very dense silty sand to sand / very stiff to hard clayey silt till	1	• Relative ease of construction with conventional equipment.	• Dewatering required; however the effort will be reduced if the final Boag Road grade is constructed first and allowed to drain prior to foundation excavation; • Does not allow for integral abutment design.	• Low relative costs ($170,000) compared to other options if dewatering can be performed as part of the proposed Boag Road grading.	• Potential difficulties in maintaining integrity of foundation subgrade if adequate dewatering is not performed.
Steel H-Piles driven to found within "100-blow" clayey silt till	2	• Allows for integral abutment design; • Greater axial resistance available for design.	• High water table will require dewatering for construction of pile caps in the dry; • Pile lengths anticipated to be highly variable as depth to "100-blow" material below proposed pile cap varies from 9 m to 20 m at the south abutment and from 4 m to 12 m at the north abutment; • Pile tip / flange will have to be reinforced to penetrate through cobbles/boulders; • Pile may "hang up" on boulders reducing the axial resistance and additional piles may be required.	• Higher costs ($210,000) compared to shallow footings • Additional costs for verifying individual pile capacities in the field given the variable depth to "100-blow" material.	• Difficulty predicting pile lengths and axial resistance given the highly variable depth to "100-blow" clayey silt till; piles will likely require independent field verification on a per pile basis; • Potential risk of water seepage along the pile due to artesian groundwater conditions; • Pre-augering through cobbles/ boulders may be required depending on pile cap elevation.
Caissons to found within "100-blow" material	NP	• Potential for greater axial resistance than steel H-piles.	• High water table, artesian water conditions and highly variable end-bearing stratum level within foundation footprint make this option less practical than shallow foundations or steel H-piles; • May not be possible to inspect founding level; • Will require temporary / permanent steel liner to be able to advance caissons.	• Higher cost ($370,000) per unit basis than piles but will require fewer units to carry the total design load.	• Higher risk of "heaving" or "softening" of end-bearing stratum; • Difficulty confirming caisson tip is founded on "100-blow" material given the highly variable consistency of the clayey silt till.

Notes:

1. The approximate costs are rough estimates and provided only for comparison purposes.

NP = not practical

MINISTRY OF TRANSPORTATION, ONTARIO

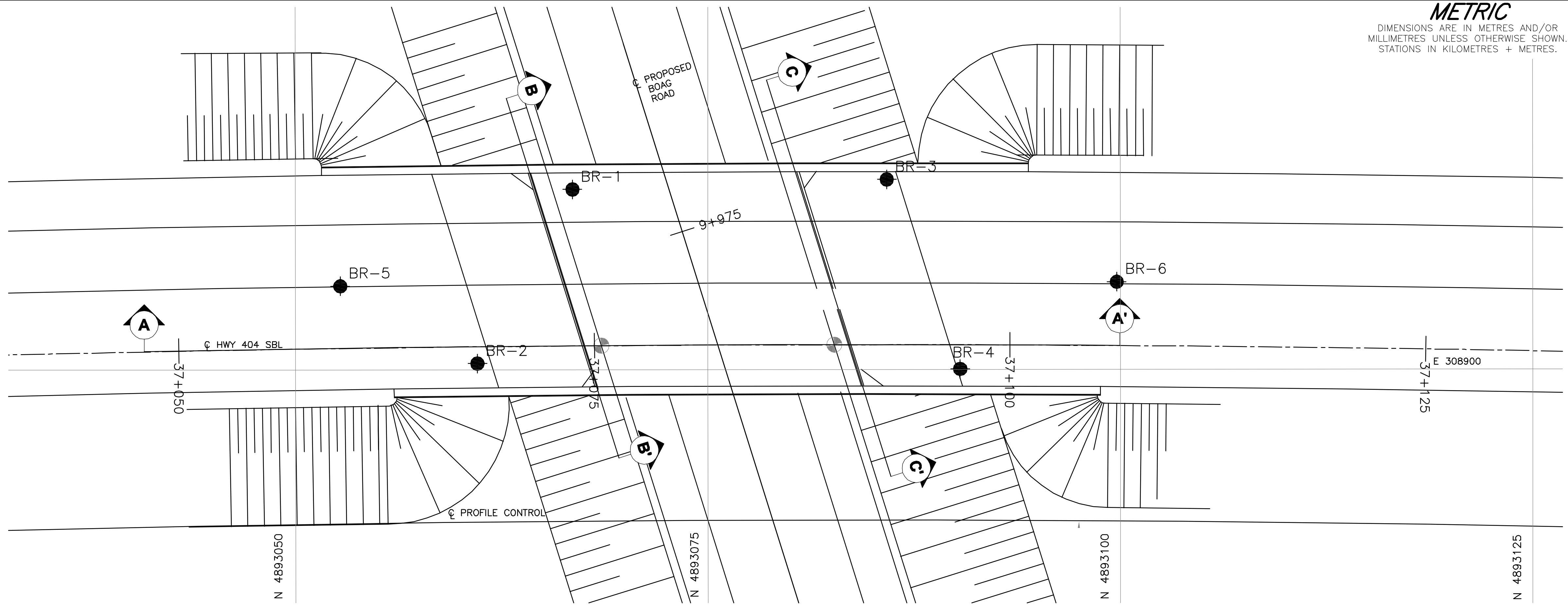
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2005–07–00

SHEET

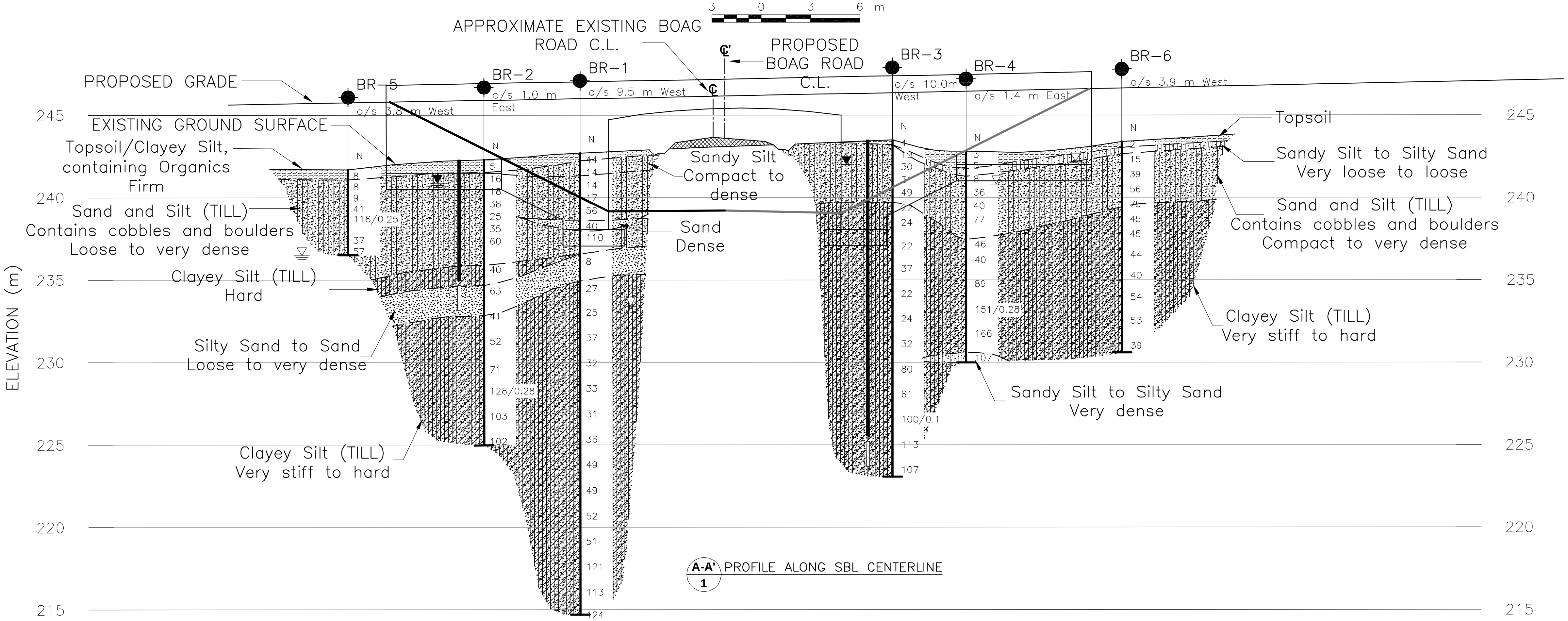
HIGHWAY 404
BOAG ROAD OVERPASS – SBL
BOREHOLE LOCATION AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



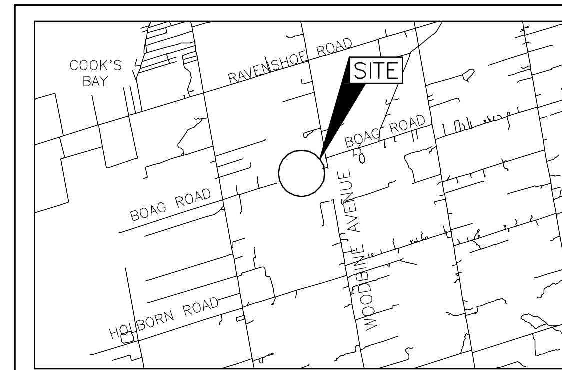
PLAN

SCALE
3 0 3 6 m



A-A' PROFILE ALONG SBL CENTERLINE 1

SCALE
3 0 3 6 m



KEY PLAN
SCALE
1.5 0 1.5 km

LEGEND

- Borehole – Current Investigation
- 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 20, 2009
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
BR–1	242.7	4893066.8	308889.1
BR–2	242.3	4893061.0	308899.6
BR–3	243.5	4893085.8	308888.5
BR–4	242.8	4893090.3	308900.0
BR–5	241.7	4893052.7	308894.9
BR–6	243.4	4893099.8	308894.7

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 AECOM, drawing file no. 2538–199–ST–0001–SBL–To Golder–090930.dwg, received Oct. 09, 2009.

NO.	DATE	BY	REVISION
1	Nov. 18, 2009	KJB	UPDATED BASE DRAWING

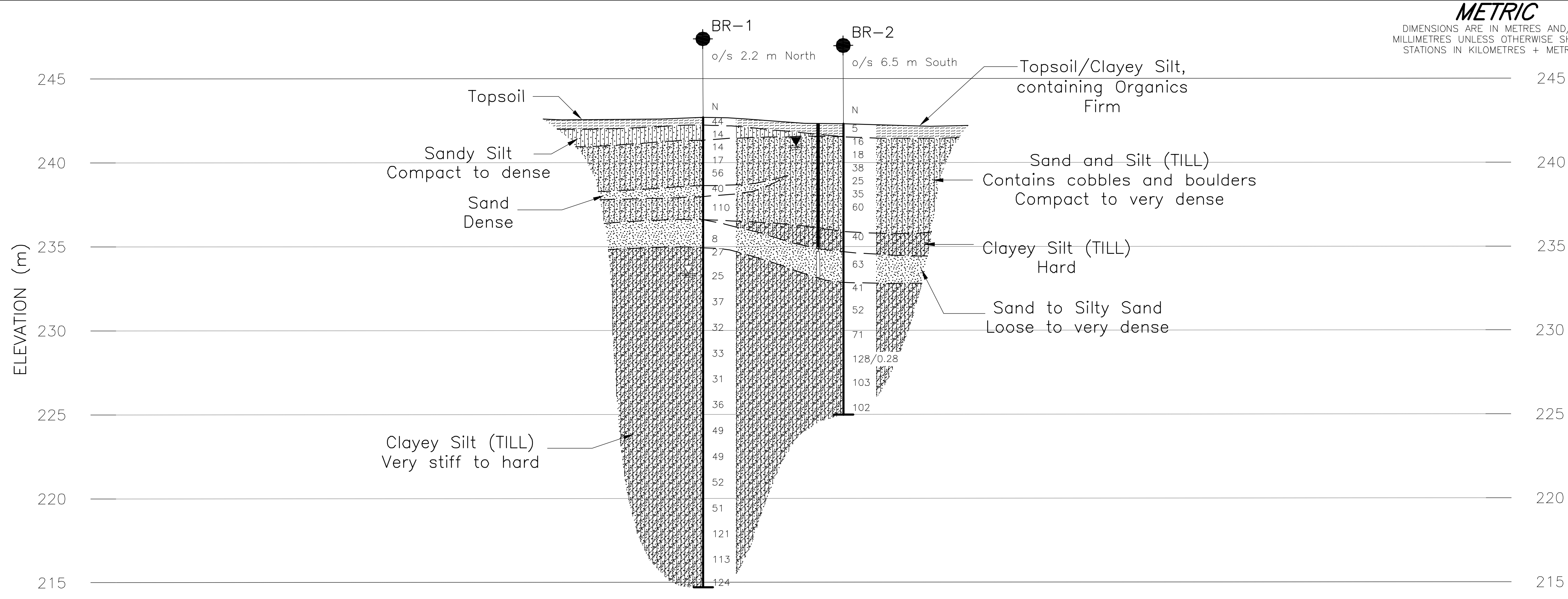
Geocres No. 31D–464

HWY. 404		PROJECT NO. 08–1111–0022	DIST.
SUBM'D.	CHKD. TB	DATE: Nov. 2009	SITE:
DRAWN: DD	CHKD. KJB	APPD. JMAC	DWG. 1

PLOT DATE: November 18, 2009
FILENAME: T:\Projects\2008\08-1111-0022 (UMA, York)\--DD-- SBL Bridge Report\08111100220001.dwg

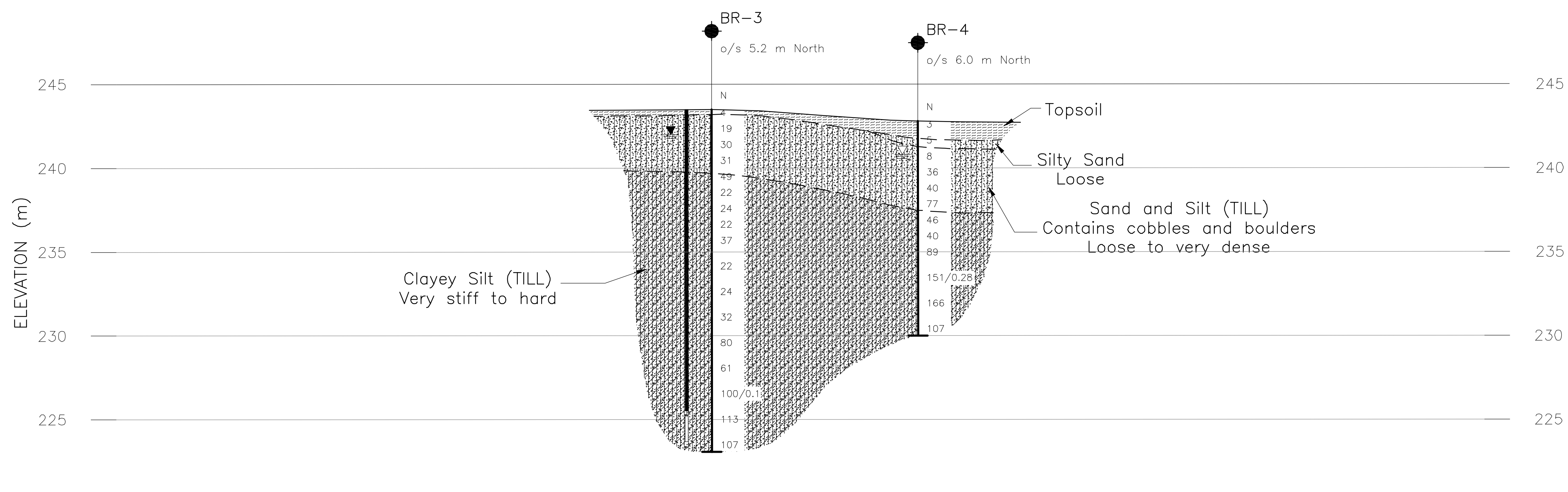
METRIC

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



B-B' 1 SECTION ALONG SOUTH ABUTMENT

SCALE 3 0 3 6 m



C-C' 1 SECTION ALONG NORTH ABUTMENT

SCALE 3 0 3 6 m

CONT No.
WP No.2005-07-00

HIGHWAY 404
BOAG ROAD OVERPASS – SBL
SOIL STRATA

SHEET

Golder Associates

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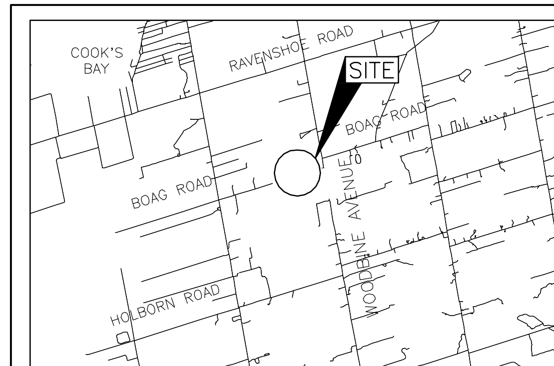
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE 1.5 0 1.5 km

LEGEND

- Borehole – Current Investigation
- 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 20, 2009
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BR-1	242.7	4893066.8	308889.1
BR-2	242.3	4893061.0	308899.6
BR-3	243.5	4893085.8	308888.5
BR-4	242.8	4893090.3	308900.0

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file no. 2538-199-ST-0001-SBL-To Golder-090930.dwg, received Oct. 09, 2009.

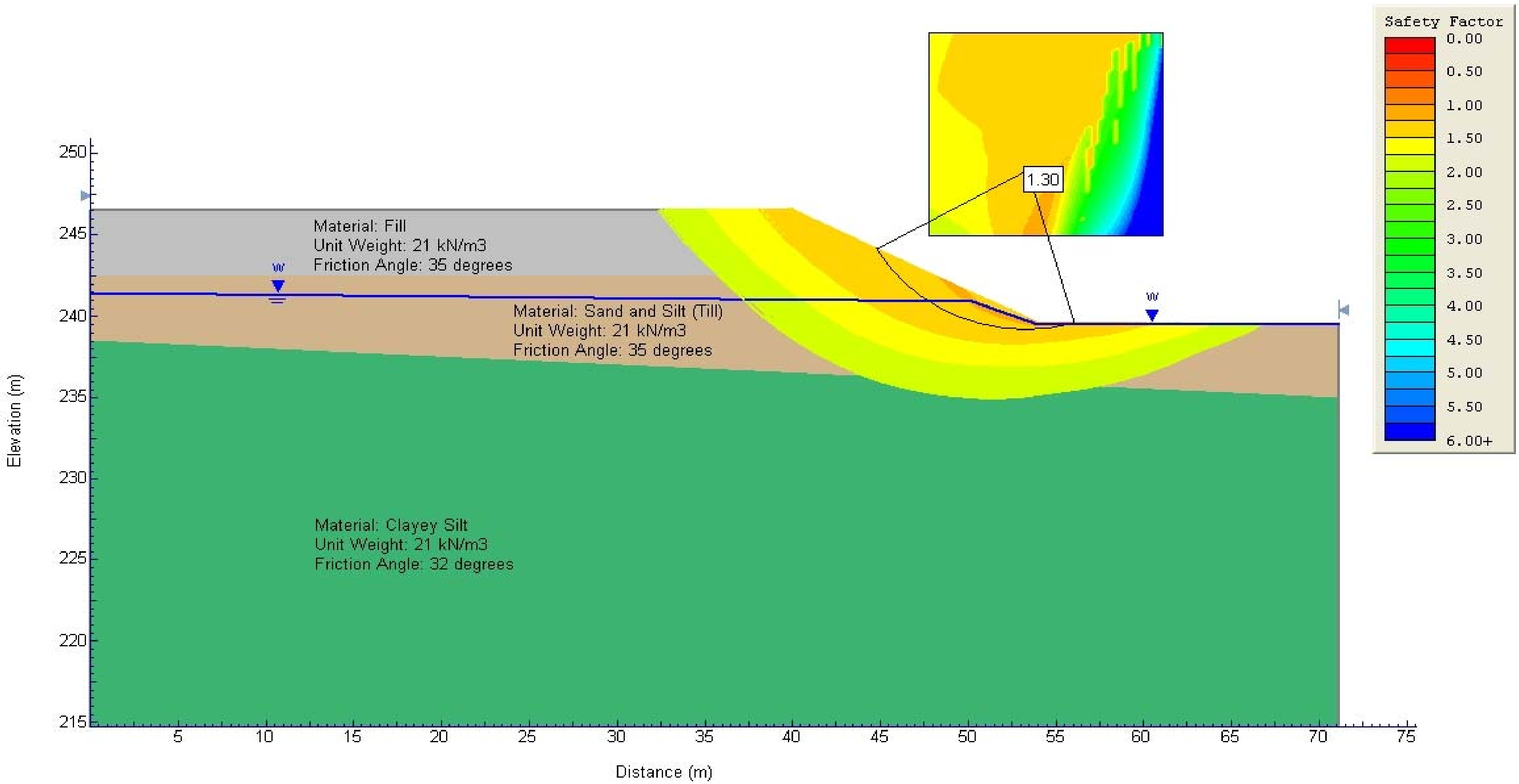
NO.	DATE	BY	REVISION
1	Nov. 18, 2009	KJB	UPDATED BASE DRAWING

Geocres No. 31D-464

HWY. 404		PROJECT NO. 08-1111-0022	DIST.
SUBM'D.	CHKD. TB	DATE: Nov. 2009	SITE:
DRAWN: DD	CHKD. KJB	APPD. JMAC	DWG. 2

Stability Analysis - North Approach
Hwy 404 SBL / Boag Road Overpass Structure

Figure 1



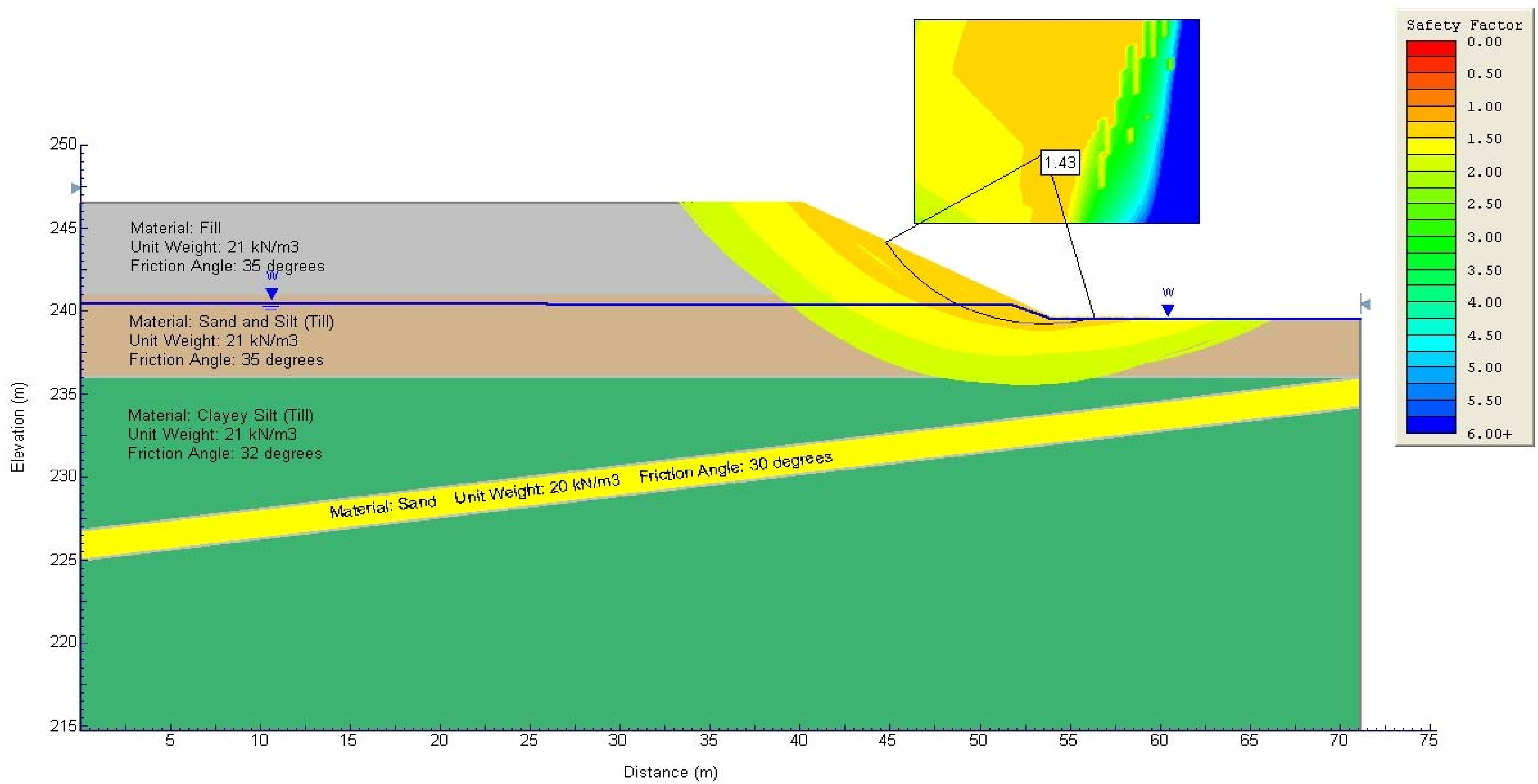
Date: Jul-09
Project: 08-1111-0022A

Golder Associates

Drawn: TB
Checked: KJB

Stability Analysis - South Approach
Hwy 404 SBL / Boag Road Overpass Structure

Figure 2



Date: Jul-09
Project: 08-1111-0022A

Golder Associates

Drawn: TB
Checked: KJB

APPENDIX A

Record of Boreholes

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
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

RECORD OF BOREHOLE No BR-1

 METRIC

PROJECT 08-1111-0022			
W.P. 2005-07-00	LOCATION N 4893066.8 ;E 308889.1		ORIGINATED BY TB
DIST	HWY 404	BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger	COMPILED BY SC
DATUM Geodetic	DATE February 19, 2009		CHECKED BY JB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
242.7	GROUND SURFACE				
0.0	TOPSOIL				
0.2	Sandy SILT, trace clay, trace gravel, containing organics and rootlets near surface Compact to dense Orange brown to brown Moist	1	SS	44	
		2	SS	14	
241.3 1.4	SAND and SILT, some gravel, trace to some clay, containing cobbles and boulders and oxidation staining (TILL) Compact to very dense Brown Moist	3	SS	14	
		4	SS	17	21 34 38 7
		5	SS	56	
238.6 4.1	SAND, trace to some silt Dense Brown Wet	6	SS	40	
238.0 4.7	SAND and SILT, trace clay, trace gravel, containing cobbles and boulders and oxidation staining (TILL) Very dense Brown Moist	7	SS	110	
236.6 6.1	Silty SAND, trace clay Loose Brown Wet	8	SS	8*	0 69 30 1
234.9 7.8	CLAYEY SILT, trace to some sand, trace to some gravel, containing sandy silt interlayers (TILL) Very stiff to hard Grey Moist	9	SS	27	
		10	SS	25	
	Grinding of augers on inferred cobbles and boulders to 15.2 m				
		11	SS	37	
		12	SS	32	1 6 70 23
		13	SS	33	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL).

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BR-1

 METRIC

PROJECT 08-1111-0022
W.P. 2005-07-00
DIST
HWY 404
DATUM Geodetic
LOCATION N 4893066.8 ;E 308889.1
BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger
DATE February 19, 2009
ORIGINATED BY TB
COMPILED BY SC
CHECKED BY JB

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

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION	GR	SA	SI	CL
	CLAYEY SILT, trace to some sand, trace to some gravel, containing sandy silt interlayers (TILL) Very stiff to hard Grey Moist	14	SS	31	227				
		15	SS	36	226				
		16	SS	49	225 / 224	11	11	58	20
		17	SS	49	223				
		18	SS	52	222 / 221				
		19	SS	51	220				
		20	SS	121	219 / 218				
		21	SS	113	217 / 216				
214.7 28.0		22	SS	124	215				

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BR-1

 METRIC

PROJECT 08-1111-0022			
W.P. 2005-07-00	LOCATION N 4893066.8 ;E 308889.1	ORIGINATED BY TB	
DIST	HWY 404	BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger	COMPILED BY SC
DATUM Geodetic	DATE February 19, 2009	CHECKED BY JB	

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 NOTE: 1. Water level inside augers at a depth of 9.3 m (Elev. 233.4 m) below ground surface upon completion of drilling. * The 'N' value of 8 may be the result of "blowing" sands during drilling operations and may not be representative of the in-situ relative density of the sand layer.										

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

Golder Associates

RECORD OF BOREHOLE No BR-2

 METRIC

PROJECT 08-1111-0022
W.P. 2005-07-00
LOCATION N 4893061.0 ;E 308899.6
ORIGINATED BY TB
DIST ____ HWY 404
BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger
COMPILED BY SC
DATUM Geodetic
DATE February 23, 2009
CHECKED BY JB

ELEV / DEPTH	SOIL PROFILE DESCRIPTION
242.3	GROUND SURFACE
0.0	CLAYEY SILT, some sand, trace gravel, containing organics and rootlets Firm Brown Moist
241.5 / 0.8	SAND and SILT, trace to some clay, trace to some gravel, containing cobbles and boulders and oxidation staining (TILL) Compact to very dense Brown Moist
235.9 / 6.4	CLAYEY SILT, some sand, trace gravel, contains sand seams (TILL) Hard Grey Moist to wet
234.7 / 7.6	Silty SAND, trace clay Very dense Brown Moist
232.9 / 9.5	CLAYEY SILT, trace sand, trace gravel, contains sandy silt seams (TILL) Hard Grey Moist

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	OC=2.6%
2	SS	16	
3	SS	18	
4	SS	38	
5	SS	25	10 33 45 12
6	SS	35	
7	SS	60	3 44 50 3
8	SS	40	
9	SS	63	
10A / 10B	SS	41	
11	SS	52	
12	SS	71	
13	SS	128/0.28	0 2 77 21

ELEVATION SCALE: 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228

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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BR-2

 METRIC

PROJECT 08-1111-0022

W.P. 2005-07-00

LOCATION N 4893061.0 ;E 308899.6

ORIGINATED BY TB

DIST HWY 404

BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger

COMPILED BY SC

DATUM Geodetic

DATE February 23, 2009

CHECKED BY JB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand, trace gravel, contains sandy silt seams (TILL) Hard Grey Moist	14	SS	103	227		
					226		
225.0		15	SS	102	225	~20	
17.3	END OF BOREHOLE						

NOTES:

1. Auger refusal on inferred boulder at 5.8 m depth (Elev. 236.5 m). Drill rig moved 1 m North and continued drilling.

2. Water level inside augers at a depth of 6.4 m (Elev. 235.9 m) during drilling operations.

3. Water level in piezometer at a depth of 1.4 m (Elev. 240.9 m) below ground surface on March 12, 2009.

4. Water level in piezometer at a depth of 1.3 m (Elev. 241.0 m) below ground suface on May 20, 2009.

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

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

RECORD OF BOREHOLE No BR-3

 METRIC

PROJECT 08-1111-0022
W.P. 2005-07-00
DIST
HWY 404
DATUM Geodetic
LOCATION N 4893085.8 ;E 308888.5
BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger
DATE February 10 and 11, 2009
ORIGINATED BY TB
COMPILED BY SC
CHECKED BY JB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
243.5	GROUND SURFACE						
0.0	TOPSOIL	1	SS	4	243		
243.2 / 0.3	SAND and SILT, trace clay, trace gravel, containing cobbles and boulders and oxidation staining (TILL) Compact to dense Brown Moist	2	SS	19		o	
		3	SS	30	242		
		4	SS	31	241	o	
		5	SS	49	240		
239.7 / 3.8	CLAYEY SILT, trace to some sand, trace gravel, containing sandy silt interlayers (TILL) Very stiff to hard Brown to grey Moist	6	SS	22		o	1 4 72 23
	Oxide staining to a depth of 6.1 m	7	SS	24	239	o	
					238		
	Becoming grey at a depth of 6.1 m	8	SS	22	237	o	
					236		
		9	SS	37	235		
		10	SS	22	234		
		11	SS	24	233	⊢o⊣	1 7 71 21
		12	SS	32	232, 231		
		13	SS	80	230, 229	o	

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

Continued Next Page

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

RECORD OF BOREHOLE No BR-3

 METRIC

PROJECT 08-1111-0022

W.P. 2005-07-00

LOCATION N 4893085.8 ;E 308888.5

ORIGINATED BY TB

DIST HWY 404

BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger

COMPILED BY SC

DATUM Geodetic

DATE February 10 and 11, 2009

CHECKED BY JB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace to some sand, trace gravel, containing sandy silt interlayers (TILL) Very stiff to hard Brown to grey Moist Oxide staining to a depth of 6.1 m	14	SS	61	228	
					227	
		15	SS	100/0.10		
					226	
		16	SS	113	225	
					224	
223.1		17	SS	107		
20.4	END OF BOREHOLE					

NOTES:

1. Water level inside augers measured at a depth of 2.9 m (Elev. 240.6 m) below ground surface upon completion of drilling.

2. Water level in piezometer at a depth of 4.0 m (Elev. 239.5 m) below ground surface on February 26, 2009.

3. Water level in piezometer at a depth of 2.1 m (Elev. 241.4 m) below ground surface on March 12, 2009.

4. Water level in piezometer at a depth of 1.5 m (Elev. 242.0 m) below ground surface on May 20, 2009.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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 BR-4

 METRIC

PROJECT 08-1111-0022
W.P. 2005-07-00
DIST ___ HWY 404
DATUM Geodetic
LOCATION N 4893090.3 ;E 308900.0
BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger
DATE February 10, 12 and 13, 2009
ORIGINATED BY TB
COMPILED BY SC
CHECKED BY JB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
242.8	GROUND SURFACE						
0.0	TOPSOIL	1	SS	3			
241.8 / 1.0	Silty SAND, trace clay, containing rootlets, Loose, Brown, Moist	2	SS	5		242	
241.3 / 1.5	SAND and SILT, trace clay, trace gravel, containing cobbles and boulders and oxidation staining (TILL), Loose to very dense, Moist, Brown	3	SS	8	∇	241	
		4	SS	36		240	2 50 43 5
		5	SS	40			
		6	SS	77		239	
						238	
237.5 / 5.3	CLAYEY SILT, trace to some sand, trace gravel, contains sandy silt interlayers (TILL), Hard, Grey, Moist to wet	7	SS	46		237	
		8	SS	40		236	
		9	SS	89		235	
						234	
		10	SS	151/0.28		233	5 15 64 16
	Becoming wet below a depth of 10.7 m	11	SS	166		232	0 1 74 25
						231	
230.6 / 12.2	Sandy SILT to Silty SAND, trace clay, Very dense, Grey, Wet	12	SS	107			
230.0 / 12.8	END OF BOREHOLE					230	

NOTES:

1. Water level in open borehole measured at a depth of 2.0 m (Elev. 240.8 m) below ground surface upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No BR-5

 METRIC

PROJECT 08-1111-0022

W.P. 2005-07-00 LOCATION N 4893052.7 ;E 308894.9 ORIGINATED BY TB

DIST HWY 404 BOREHOLE TYPE 108 mm O.D. Solid Stem Auger COMPILED BY SC

DATUM Geodetic DATE February 24, 2009 CHECKED BY JB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
241.7	GROUND SURFACE											
0.0	TOPSOIL	1	SS	8								
241.1 0.6	SAND and SILT, some gravel, trace to some clay, containing cobbles and boulders and oxidation staining (TILL) Loose to very dense Brown Moist	2	SS	8		241	o		14	33	44	9
		3	SS	9		240						
		4	SS	41		239	o		14	36	42	8
		5	SS	116/0.25								
		6	SS	37		238	o					
	Becoming grey/brown at a depth of 4.9 m	7	SS	57		237						
236.5 5.2	END OF BOREHOLE				▽							

NOTES:

1. Water level in open borehole measured at a depth of 5.2 m (Elev. 236.5 m)below ground surface upon completion of drilling.

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 (%)

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

Golder Associates

RECORD OF BOREHOLE No BR-6

 METRIC

PROJECT 08-1111-0022
W.P. 2005-07-00
DIST ___ HWY 404
DATUM Geodetic
LOCATION N 4893099.8 ;E 308894.7
BOREHOLE TYPE 108 mm I.D. Hollow Stem Auger
DATE February 13, 2009
ORIGINATED BY TB
COMPILED BY SC
CHECKED BY JB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
243.4	GROUND SURFACE					
0.0 243.0	TOPSOIL	1	SS	2		
242.6 0.8	Sandy SILT, trace clay, trace gravel, containing rootlets Very loose Brown Wet	2	SS	15	∇	
	SAND and SILT, trace to some clay, trace gravel, containing cobbles and boulders and oxidation staining (TILL) Compact to very dense Brown Moist	3	SS	39		
		4	SS	56		5 39 48 8
		5	SS	75		
239.4 4.0	CLAYEY SILT, trace to some sand, trace gravel, contains sandy silt interlayers (TILL) Hard Brown Moist	6	SS	45		
		7	SS	45		
	Becoming grey below a depth of 5.2 m					
		8	SS	44		2 10 64 24
		9	SS	40		
		10	SS	54		
		11	SS	53		
		12	SS	39		
230.6 12.8	END OF BOREHOLE					

NOTES:

1. Water level in open borehole measured at a depth of 1.2 m (Elev. 242.2 m) two hours after completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
ELEVATION SCALE: 243 242 241 240 239 238 237 236 235 234 233 232 231

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

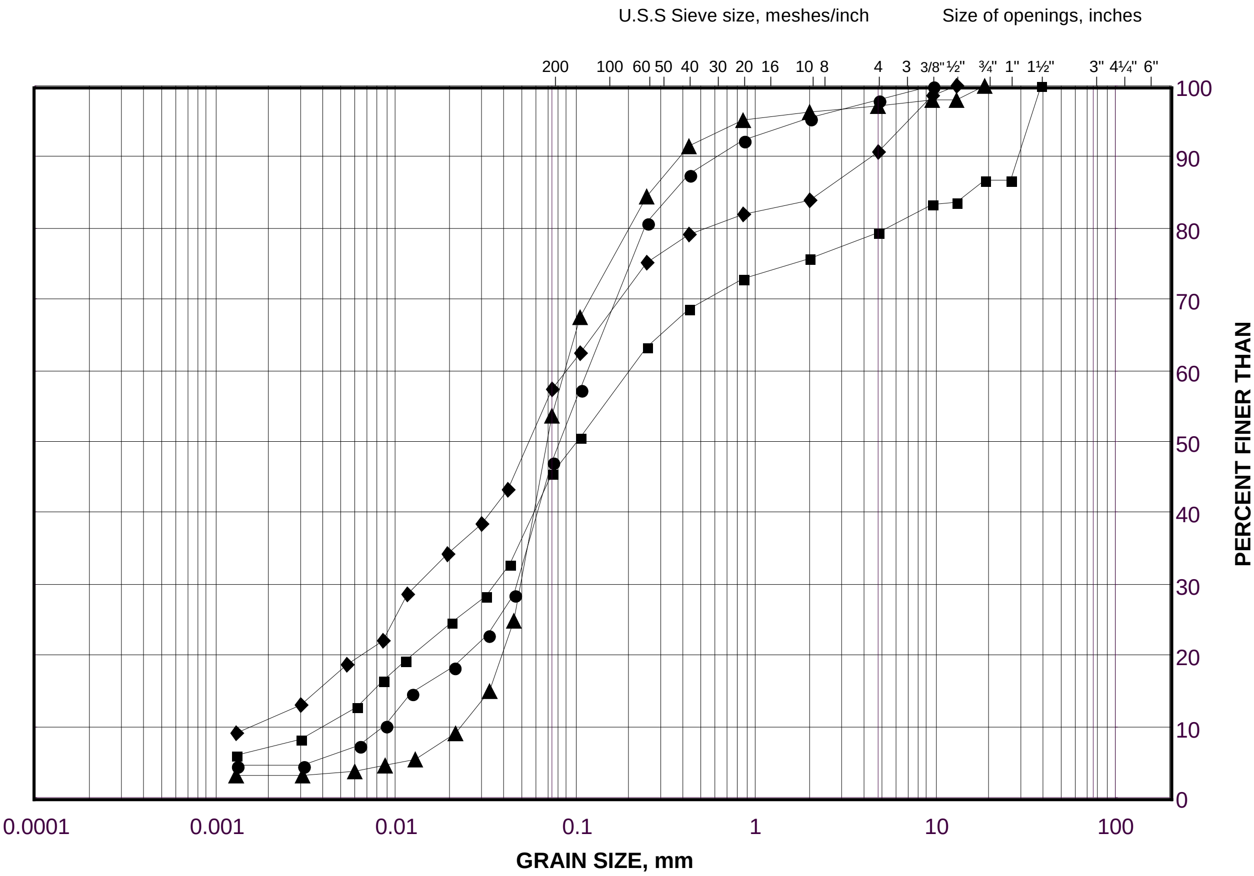
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand and Silt Till

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BR-4	4	240.2
■	BR-1	4	240.1
◆	BR-2	5	238.9
▲	BR-2	7	237.5

Project Number: 08-1111-0022

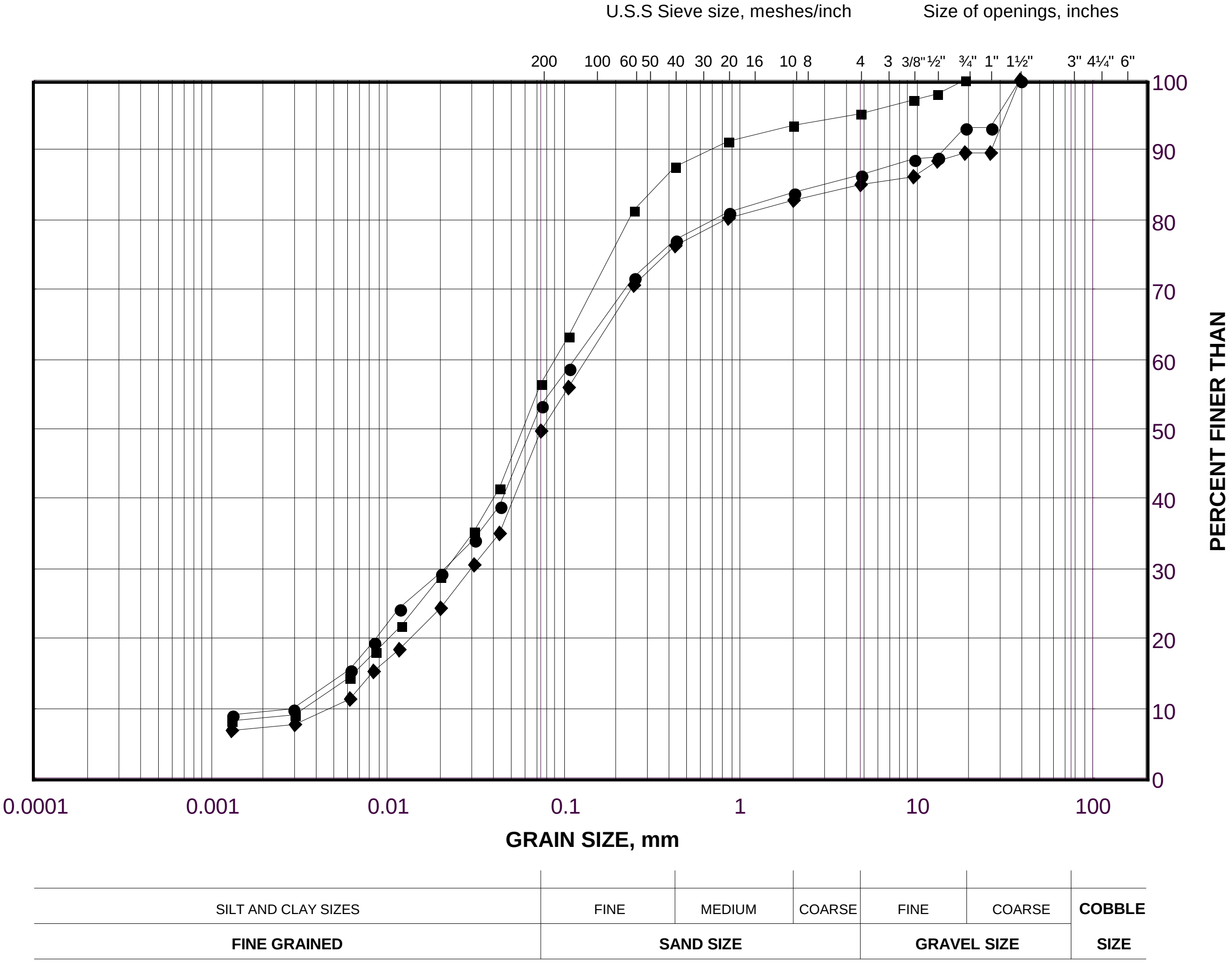
Checked By: KJB

Golder Associates

Date: 02-Jul-09

GRAIN SIZE DISTRIBUTION
Sand and Silt Till

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BR-5	2	240.7
■	BR-6	4	240.8
◆	BR-5	4	239.1

Project Number: 08-1111-0022

Checked By: KJB

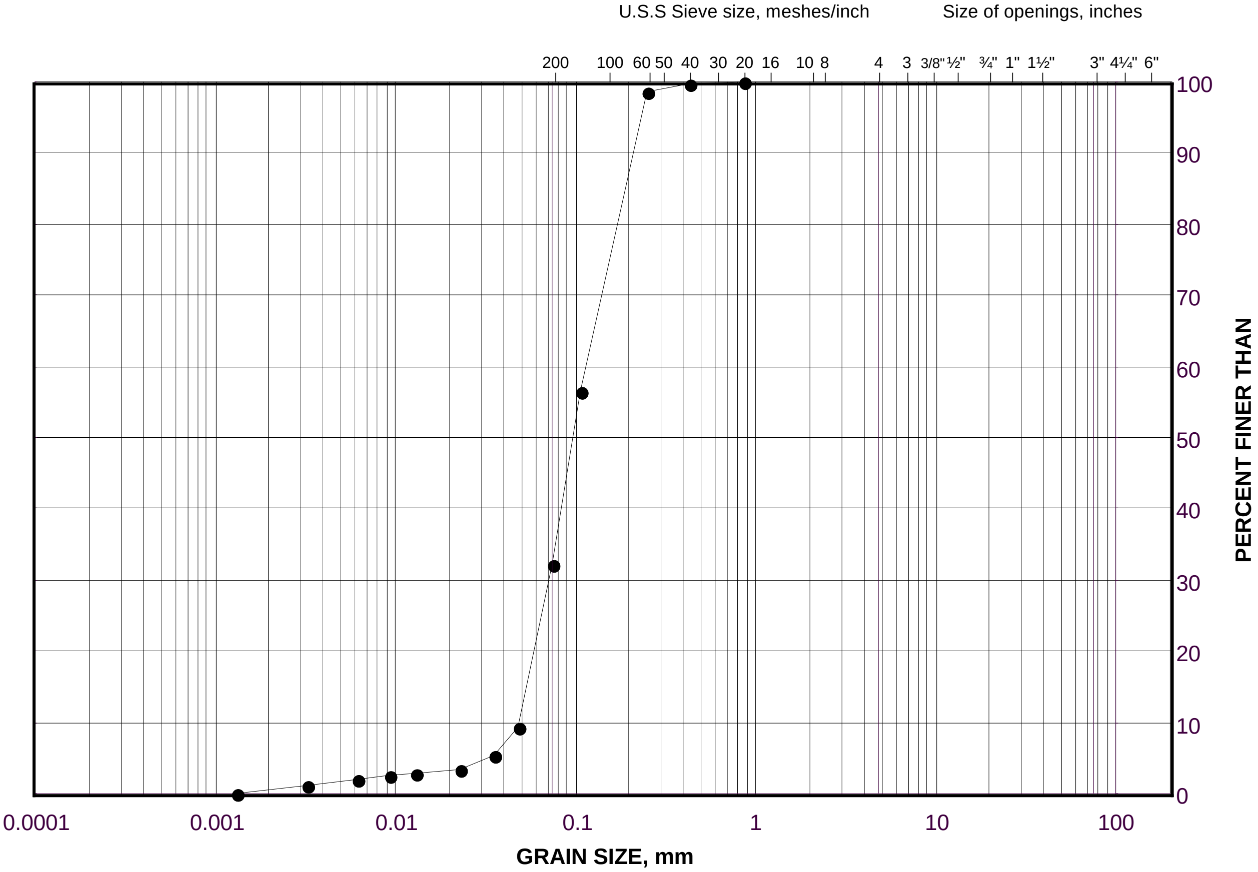
Golder Associates

Date: 02-Jul-09

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BR-1	8	236.2

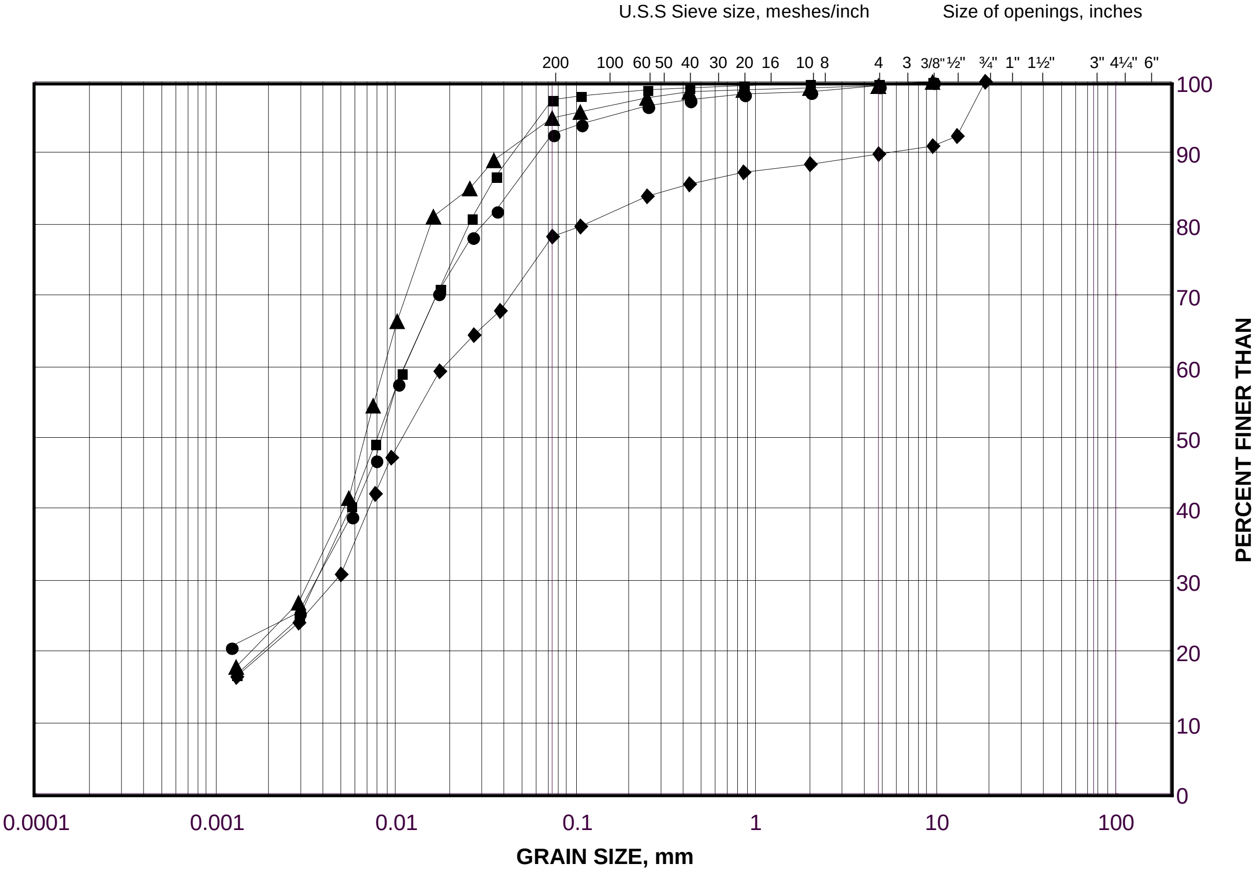
Project Number: 08-1111-0022

Checked By: KJB

Golder Associates

Date: 02-Jul-09

GRAIN SIZE DISTRIBUTION Clayey Silt Till	FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BR-1	12	230.2
■	BR-2	13	228.3
◆	BR-1	16	224.1
▲	BR-3	6	239.4

Project Number: 08-1111-0022

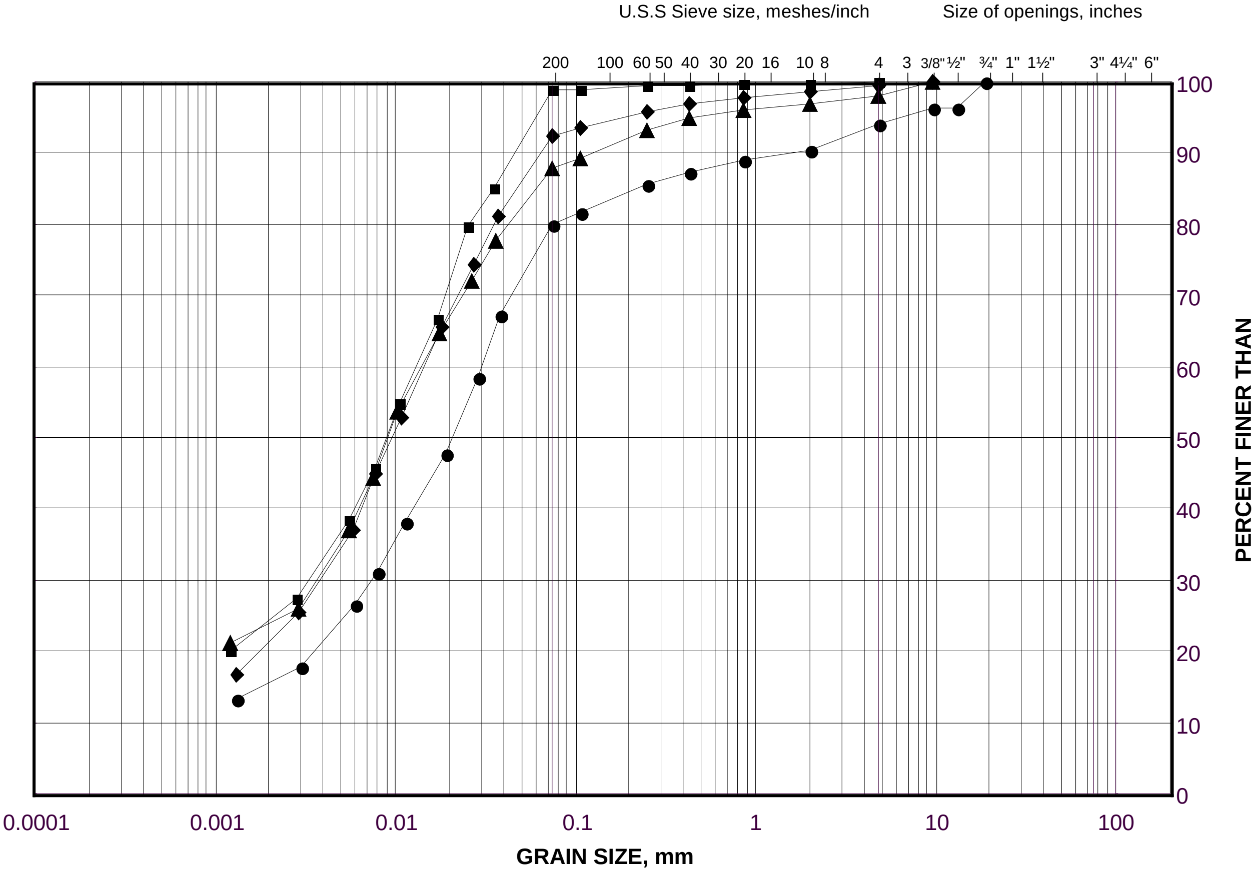
Checked By: KJB

Golder Associates

Date: 02-Jul-09

GRAIN SIZE DISTRIBUTION
Clayey Silt Till

FIGURE B5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BR-4	10	233.4
■	BR-4	11	231.8
◆	BR-3	11	232.5
▲	BR-6	8	237.0

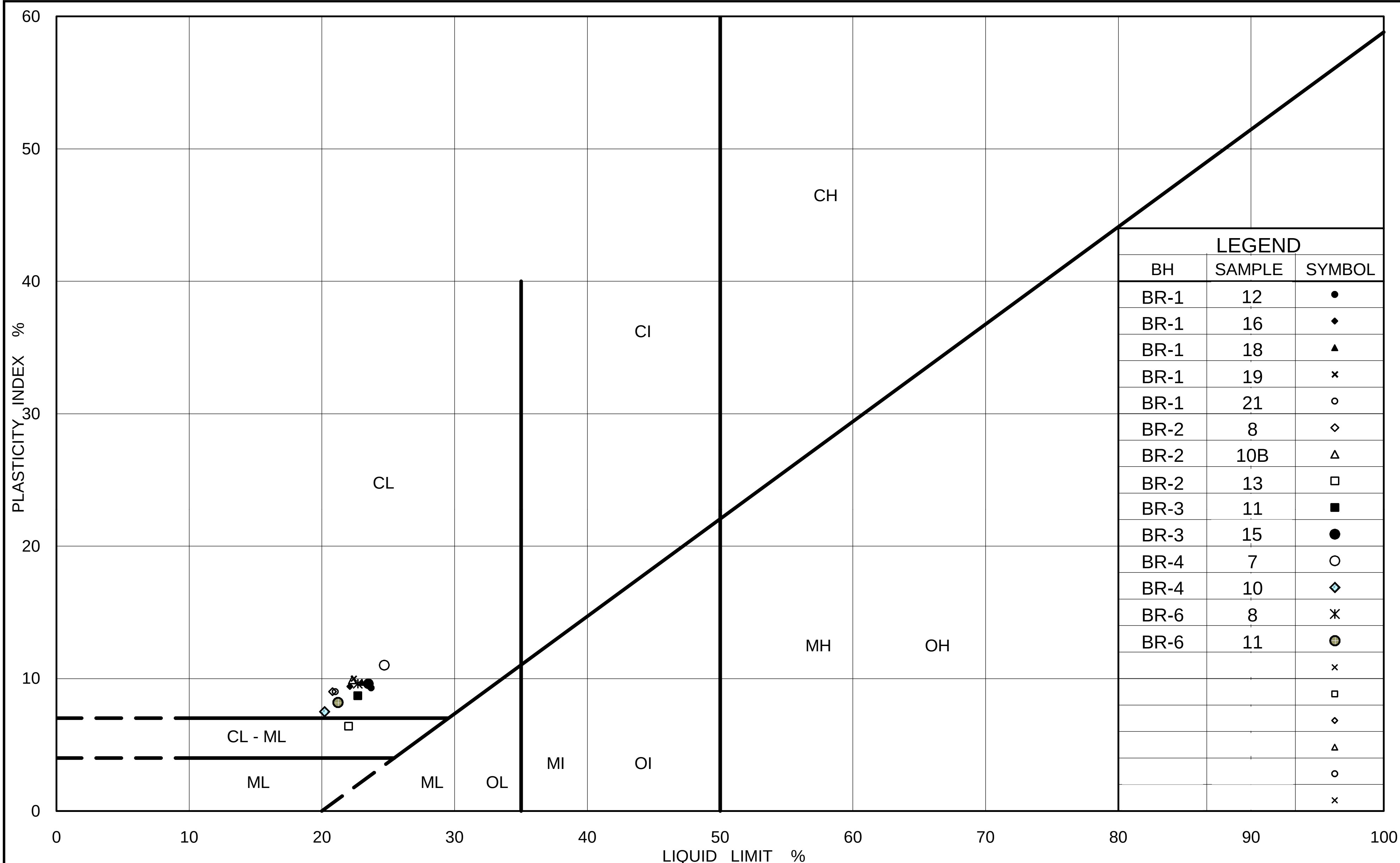
Project Number: 08-1111-0022

Checked By: KJB

Golder Associates

Date: 02-Jul-09

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Till

Figure No. B6

Project No. 08-1111-0022

Checked By: KJB

APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The approximately 3.2 m to 5.6 m thick sand and silt till deposit encountered at about 0.3 m to 1.5 m below ground surface and extending between about Elevation 235 m and Elevation 243 m contains cobbles and boulders as indicated in the Record of Borehole sheets. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation for spread footings and pre-augering for deep foundations.

Basis of Payment

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

END OF SECTION

DRIVING SHOES - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply Titus Standard "H" Bearing Pipe Point design driving shoes on HP 310 x 110 Piles for the Highway 404 – SBL Bridge over Boag Road.

References
OPSS 906 – Structural Steel
SP903S01

Materials

The driving shoes shall be of the following:

Product	***Manufacturer***
HPP-S-12	Titus Steel Company Ltd. 6767 Invader Cr. Mississauga, ON Tel (905) 564-2446

(Or approved equivalent)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

END OF SECTION

UNWATERING FOR FOUNDATION EXCAVATION - Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that artesian groundwater conditions (up to 1.3 m below existing ground surface) are present at the proposed Highway 404 –SBL Bridge site over Boag Road and along the proposed new Boag Road profile grade. It is estimated that the base of temporary excavations for the foundations will be up to 5 m below the groundwater level as measured in a piezometer installed in Borehole BR-3 on May 20, 2009. The subsoil conditions consist of sand and silt tills and clayey silt tills containing confined water-bearing silty sand to sand layers. Construction of shallow foundations / pile caps must be carried out in the dry. Dewatering within the foundation excavations will be required and the excavation shall be kept stable during the work. It is considered that a combination of adequately sized pumped pressure relief wells and perimeter ditches / trenches or sheetpile box configuration is required to lower the groundwater. If sheetpiles are used, installation procedures need to consider the presence of dense / hard till deposits containing cobbles and boulders encountered between about Elevation 235 m and 243 m, as indicated in the Record of Borehole sheets.

Basis of Payment

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

END OF SECTION

LEAN CONCRETE (MUD MAT) – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes supply and installation of the lean concrete (i.e. mud mat) to prevent erosion and/or disturbance to the foundation soils, if required. If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and inspection, a working mat of lean concrete should be placed in the excavation to protect the integrity of the bearing stratum.

Construction

Lean concrete shall have a compressive strength of at least 5 MPa, shall be placed in general accordance with OPSS 904, and the working mat shall have a minimum thickness of 75 mm. The working mat should extend to at least one metre beyond the foundation footprint or to the limits of the excavation.

Basis of Payment

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

PILE CAPACITY VERIFICATION PROCEDURE - Item No.

Non-Standard Special Provision

Scope

The Contractor shall commence assessment of the pile capacity by the Hiley Formula (Standard Structural Drawing SS-103-11) once the pile reaches a depth of 1.5 m above the design pile tip elevation shown in the Contract Drawings and assess the ultimate axial resistance of the pile using the Hiley Formula at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 1.5 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula should then be applied immediately upon re-striking of the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

References

SP903S01

Basis of Payment

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

END OF SECTION

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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