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REPORT ON
FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 400 SERVICE ROADS
FROM 3.7 KM NORTH OF MUSKOKA ROAD 48 NORTHERLY 3.6 KM
AND FROM 1.0 KM SOUTH OF MUSKOKA ROAD 33, NORTHERLY 2.1 KM
G.W.P. 5622-02-00
MINISTRY OF TRANSPORTATION, ONTARIO

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 400 SERVICE ROADS
FROM 3.7 KM NORTH OF MUSKOKA ROAD 48 NORTHERLY 3.6 KM
AND FROM 1.0 KM SOUTH OF MUSKOKA ROAD 33, NORTHERLY 2.1 KM
G.W.P. 5622-02-00
MINISTRY OF TRANSPORTATION, ONTARIO**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) to carry out a foundation investigation and provide design services as part of the detail design for the Highway 400 Service Roads north of Muskoka Road 48 and in the area of Muskoka Road 33.

This report addresses the foundation investigation for construction of new embankments over three swamps, the widening of an existing ramp embankment adjacent to a lake/swamp and the relocation of a highway on-ramp including the removal of bedrock adjacent to the existing bridge abutment. The plans detailing the proposed locations of the embankments were provided to Golder by URS in August 2006. Several foundation investigations have been carried out previously by others in the general area and are referenced in the following reports:

- Final Foundation Assessment Report, Highway 400 Service Roads from 3.7 km North of Muskoka Road 48 Northerly 3.6 km and from 1.0 km South of Muskoka Road 33 Northerly 2.1 km, G.W.P. 5622-02-00, Township of Georgian Bay, Ontario, Huntsville Area, District 42, GEOCRES No. 31D-405, by Jacques Whitford Limited, dated September 2004;
- Foundation Investigation Report for Joe King Road Overpass, Highway 69, Site No. 42-304, W.P. 204-90-05/06, Huntsville, GEOCRES No. 31D-339;
- Foundation Investigation Report for Muskoka Road 33 Underpass, Highway 69, Site 42-311, W.P. 689-93-04, District 52, Huntsville, GEOCRES, No. 31D-351, dated January 1995;
- High Fill Embankments at Georgian Bay Road/Hwy 69 Interchange, W.P. 215-89-00, Huntsville, District 52, GEOCRES No. 31D-352, dated February 1995;
- Foundation Investigation Report for Highway 69 Swamp Crossings from Muskoka Rd. 38 Northerly to the Musquash River; W.P. 689-93-00, Site No. N/A, Highway 69, District 52, Huntsville, GEOCRES No. 31D-368, dated February 1998; and
- In addition, in some swamp areas, the pavement probeholes carried out as part of the current design study have been utilized in this report.

We understand that the proposed service roads are to be gravel topped or surface treated to accommodate light volumes of traffic only.

2.0 SITE DESCRIPTION

Three new service roads are proposed along the west side of Highway 400, in the Township of Georgian Bay, between Joe King Service Road in the south and Global Tower Road in the north. In general, the overall site consists of rolling terrain including forested areas, swamp areas, lakes and numerous rock outcrops.

Swamps are present along the proposed alignment of each of the three service roads - Heather Path Trail, Hidden Glen Road, and Global Tower Road. The Muskoka Road 33 Interchange S-EW ramp is currently on a rock fill embankment which extends along the edge of Bear Lake.

The location of the foundation investigation sites are shown on the Key Plan on Drawings A-1 to D-1 in the appendices to this report. In total, four swamp crossing locations requiring foundation investigations have been identified for this project as outlined below:

<i>Site</i>	<i>Drawing No.</i>	<i>Station</i>	<i>Approximate Length (m)</i>
Heather Path Trail Swamp	A-1	9+510 to 9+650	140
Global Tower Road Swamp	B-1	10+065 to 10+175	110
Hidden Glen Road Swamp	C-1	11+100 to 11+223	123
Muskoka Road 33 S-EW Ramp (Bear Lake)	D-1	13+340 to 13+465	125

In addition, a new EW-N ramp is proposed at the Muskoka Road 33 Interchange, which will include widening of Highway 400 and removing some of the bedrock adjacent to the existing east bridge abutment. The location of the site is shown on the Key Plan on Drawing E-1.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

Field investigation work was carried out by Golder on March 16, 2006 at the Muskoka Road 33 Interchange site, on March 20 and 21, 2006 at the Hidden Glen Road site, and on May 18, 2006 at the Heather Path Trail site. The Global Tower Road site was investigated from May to June 2006. The following number of boreholes (BHs), Dynamic Cone Penetration Tests (DCPTs) and hand-augered probeholes (PHs) were carried out at the four sites.

<i>Site</i>	<i>Field Work Completed</i>
Heather Path Trail Sta 9+510 to 9+650	6 BHs 4 DCPTs 18 PHs
Global Tower Road 10+065 to 10+175	10 PHs
Hidden Glen Road 11+100 to 11+223	7 BHs 5 DCPTs 9 PHs
Muskoka Road 33 S-EW Ramp (Bear Lake) 13+340 to 13+465	4 BHs

Probeholes advanced for pavement design were utilized in accordance with the Terms of Reference (TOR) for the project to supplement the borehole and DCPT information, or where swamp access conditions prohibited foundation boreholes.

The fieldwork was carried out using a track-mounted CME 55 drill rig, a truck-mounted D-90 drill rig, a D-25 skid rig or motorized portable tripod equipment, supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. The boreholes were advanced using either 150 mm outside diameter (O.D.) continuous flight solid stem augers, 83 mm inside diameter (I.D.) continuous flight hollow stem augers, or NW and/or BW size casing with wash boring techniques. Soil samples were obtained, where possible, continuously or at intervals ranging from 0.75 m to 1.5 m intervals of depth, using 50 mm O.D. split-spoon samplers driven by either an automatic or manual hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). In situ vane testing (using an MTO 'N' vane) was carried out where possible within the soft clayey strata. Laboratory vanes were also conducted on Shelby tube samples obtained at selected locations within the soft stratum.

All boreholes and DCPTs were advanced to refusal, which was encountered at depths ranging from 0.3 m to 5.3 m. The boreholes were backfilled with bentonite “holeplug” in accordance with Ontario Regulation (O.Reg.) 128 (amendment to O.Reg. 903).

Piezometers were not installed in any of the boreholes. The water levels in the open boreholes were observed during the drilling operations. The results of the water level measurements are shown on the Record of Borehole and Geotechnical Probehole Sheets included in Appendices A to D.

The fieldwork was observed throughout by members of our engineering and technical staff, who located the boreholes and test holes, arranged for the clearance of underground service locations, directed the sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as relevant. Classification testing such as water content, grain size distribution and Atterberg limits was carried out on selected samples. The results of the laboratory testing are included in Appendices A to D.

On completion of the fieldwork, the location and ground surface elevation of each completed borehole and DCPT was obtained by members of our engineering staff. Locations were measured in reference to stationing and offsets from the proposed median centreline as staked by URS; elevations relative to the geodetic datum were surveyed in reference to ground surface at the median centre-line stakes. The northing and easting coordinates depicted on the Record of Borehole and Record of Penetration Test sheets were derived from these station and offset measurements and using the existing plans for the project provided by URS.

3.2 Rock Cut – Muskoka Road 33 Structure (East Abutment)

Field investigation work for the Muskoka Road 33 structure site was carried out on May 26, 2006, by geotechnical engineers from the Golder’s Rock Mechanics Division. The purpose of the inspection was to assess the rock conditions in the vicinity of the overpass and involved visual inspection and structural mapping of the existing rock.

The east side of the northbound lane was staked at 10 m intervals and photographs were taken of the rock face from the side and top of the bridge abutments (See Figures E-1 to E-3 in Appendix E). The geometry of the rock cut was measured in terms of the rock cut face height and clear zone width (horizontal distance from the edge of the road to the cut). Discontinuities (joints) were mapped in terms of their dip, dip direction and condition. Lower hemisphere equal area projections of the geological structure orientations were plotted to allow a kinematic assessment of potential failure modes (i.e. planar type sliding, wedge type sliding or toppling type failures).

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

From published geologic information, the site is located in the physiographic region known as the Georgian Bay Fringe, a broad belt bordering Georgian Bay. This area forms the southern part of the Canadian Precambrian Shield and part of the Grenville Province (Chapman and Putnam 1984, OGS Special Volume 2). The Georgian Bay Fringe is characterized by very shallow, narrow strips of fine sand, silt and clay loams in valley, and bare to thinly rock knobs (outcrops) and ridges of granite and other rocks of Precambrian age.

4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the Record of Borehole and Record of Penetration Test sheets for each swamp included in the attached Appendices A to D. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The soil stratigraphy as encountered in the boreholes is shown on Drawings A-1 to D-1 included in Appendices A to D. Detailed descriptions of the subsurface conditions at each investigated site are provided in the following sections.

4.3 Heather Path Trail Swamp – Sta. 9+510 to 9+650

The borehole locations advanced for the proposed Heather Path Trail swamp crossing between Stations 9+510 and 9+650 are shown in plan on Drawing A-1 in Appendix A. In addition, sections showing the interpreted stratigraphy at the west toe, centreline and east toe of the proposed Heather Path Trail swamp crossing are shown on Drawings A-1 and A-2. A total of 6 boreholes (Boreholes G-1 to G-6) and 4 DCPTs (DCPT G-1 to DCPT G-4) were completed to obtain information on the subsurface conditions within this area. A total of 5 geotechnical probeholes are referenced on the drawing to supplement the information for this swamp crossing. The Record of Borehole and Record of Penetration Test sheets, the complete probehole data and the results of laboratory testing are given in Appendix A.

In general, the subsurface soils encountered in the boreholes along the proposed alignment in this area consist of thin alternating surficial deposits of clay and sand/silty sand overlying bedrock.

The ground surface in the swamp area ranged between Elevations 193.7 m and 194.9 m. Bedrock outcrops were observed on the north, northwest and south sides of the swamp. Ponded water was observed at intermittent locations across the site and was between about 0.1 m and 0.4 m in depth. An existing snowmobile trail runs generally along the rock outcrops to the west of the site. The deepest boring at this site extended to 3.4 m below the existing ground surface.

4.3.1 Topsoil/Organics/Fill

A 0.2 m to 0.5 m thick surficial layer of topsoil/organics was encountered in Boreholes G-2 to G-6 at the ground surface.

A 0.6 m thick layer of sand and gravel fill, was encountered in Borehole G-1 at the ground surface. This borehole was advanced through the existing snowmobile trail.

4.3.2 Clay

A deposit of brown clay containing silt seams was encountered underlying the fill materials in Borehole G-1, the topsoil/organics in Boreholes G-3 and G-4, and the layer of sand and gravel in Borehole G-5. The surface of this deposit varied between Elevation 193.3 m and 193.9 m. The deposit ranged between 0.3 m and 1.7 m in thickness. The probeholes indicate a clay thickness of up to 2.3 m.

SPT measured 'N' values ranging from 6 blows to 32 blows per 0.3 m of penetration indicating a firm to hard consistency.

Atterberg limits testing carried out on two samples of the clay gave liquid limits of 56 and 60 percent, plastic limits of 21 and 24 percent, corresponding to plasticity indices of 32 and 39 percent. The results are shown on Figure A-1 in Appendix A and indicate that the material is classified as a clay of high plasticity. Grain size distribution of samples of the clay are shown on Figure A-2 in Appendix A.

The natural water content measured on selected samples of the clay deposit ranged between 25 percent and 48 percent.

4.3.3 Sand and Gravel

Underlying the clay in Boreholes G-1, G-3 and G-4, and underlying the topsoil/organics in Boreholes G-2 and G-6, a deposit of grey sand was encountered, ranging in thickness from 0.4 m to 1.8 m. In Borehole G-6, this deposit is described as silt sand. The surface of the sand and

gravel deposit was encountered between approximate Elevations 192.0 m and 197.8 m, typically below Elevation 192.5 m in the middle of the swamp area and higher at the edges of the swamp.

In Borehole G-5, two layers of sand and gravel, 0.4 m thick each, were encountered below the topsoil/organics at 0.2 m depth and below the clay at 1.5 m depth.

SPT 'N' values measured within this deposit ranged from 32 to greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density. Several instances of the split-spoon bouncing were encountered during sampling and have been noted on the Record of Borehole sheets.

Grain size distributions of samples of the sand and gravel are shown on Figure A-3 in Appendix A.

Natural water contents measured on selected samples of this deposit ranged between 9 percent and 15 percent.

4.3.4 Refusal

Refusal - defined by auger refusal, spoon refusal (i.e. spoon bouncing) and/or the augers sliding - indicating probable bedrock was encountered at depths of between 0.5 m and 3.4 m below the ground surface in the borings within the swamp. The shallower depths of refusal were encountered near the edges of the swamp, closest to the bedrock outcrops.

These refusal depths, while they do not necessarily confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock surface. No bedrock coring was undertaken as part of this work.

4.3.5 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be moist to wet and the groundwater level was generally found to be at or within 1 m of the ground surface during drilling. Borehole G-6, located on a thinly covered bedrock outcrop, was dry upon completion of drilling. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and the groundwater level will vary depending on precipitation and local soil permeability.

4.4 Global Tower Road Swamp – Sta. 10+065 to 10+175

The geotechnical probeholes locations advanced for the proposed Global Tower Road Swamp crossing between Stations 10+065 to 10+175 are shown in plan on Drawing B-1 in Appendix B.

In addition, profiles showing the interpreted stratigraphy along the west toe, centreline and east toe of the location of the proposed Global Tower Road Swamp are shown on Drawings B-1 and B-2. A total of 10 probeholes were utilized for foundation information in this swamp. The probehole data and the results of laboratory testing are given in Appendix B.

In general, the subsurface soils encountered in the probeholes along the proposed trail alignment in this area consist of thin alternating surficial deposits of silty clay to clay and sand overlying bedrock. The ground surface in the swamp area ranged between Elevations 198.8 m and 200.4 m at the probehole locations. Ponded water was observed at intermittent locations across the site and was between about 0.1 m and 0.3 m in depth. Bedrock outcrops were observed to the north and south of the site. The deepest probehole at this site extended to 5.0 m below ground surface.

4.4.1 Topsoil/Organics

A surficial layer of topsoil/organics was encountered in the probeholes at the existing ground surface or below the ponded water. This surficial layer was between 0.2 m and 1.2 m in thickness and is described as dark brown to black, saturated and soft.

4.4.2 Silty Clay to Clay

A deposit of grey silty clay to clay containing trace fine sand was encountered beneath the surficial organic deposit in the probeholes. The surface of this deposit varied between approximately Elevations 197.9 m and 198.7 m. The thickness of the deposit ranged from 0.8 m to 3.6 m.

Based on the hand-auger advance through this deposit, the consistency of the silty clay to clay was inferred to be soft to firm. Laboratory vane tests performed within two Shelby tube samples (obtained by manually pushing the tubes into the ground) measured undrained shear strengths of 6 kPa and 30 kPa, indicating a very soft to firm consistency. The clay deposit, as observed from the Shelby tube samples (taken at depths of less than 2.0 m), contained a fairly high proportion of organics and/or peat in the upper portion.

Atterberg limits testing carried out on two samples of the clay gave liquid limits of 45 and 51 percent, plastic limits of 19 and 22 percent, and plasticity indices of 26 and 29 percent. The results are shown on Figure B-1 in Appendix B and indicate that the deposit consists of a silty clay of intermediate plasticity to a clay of high plasticity.

The natural water content measured on selected samples of this deposit ranged between 28 percent and 85 percent, with an average of about 64 percent. This measured value was higher than the liquid limit resulting in a liquidity index of 1.5 indicating that the clay is a sensitive clay.

4.4.3 Sand

A thin sand layer containing trace to some silt and gravel was generally encountered below the silty clay to clay deposit and immediately above the inferred bedrock surface. In general, this deposit was less than 0.3 m in thickness and had a compact relative density.

4.4.4 Refusal

Refusal was encountered at depths of between 0.4 m and 5.0 m below ground surface at the probehole locations. These refusal depths, while they do not necessarily confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock surface. No bedrock coring was undertaken as part of this work.

4.4.5 Groundwater Conditions

In general, the samples taken in the probeholes were noted to be wet or saturated and the groundwater level was generally found to be at or near ground surface during drilling. Pondered water was observed at the surface of several of the probeholes. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and the groundwater level will vary depending on precipitation and local soil permeability.

4.5 Hidden Glen Road Swamp – Sta. 11+100 to 11+223

The borehole locations advanced for the Hidden Glen Road Swamp crossing between Stations 11+100 and 11+233 are shown in plan on Drawing C-1 in Appendix C. In addition, profiles showing the interpreted stratigraphy along the west toe, centreline and east toe of the alignment are shown on Drawing C-2. A total of 7 boreholes (Boreholes H-1 to H-7) and 5 DCPTs (DCPT H-1 to DCPT H-5) were completed to obtain information on the subsurface conditions within this area. A total of 9 probeholes were referenced to supplement the information for this swamp. The Record of Borehole and Record of Penetration Test sheets, the pavement probehole data and the results of laboratory testing are given in Appendix C.

In general, the subsurface soils encountered in the boreholes along the proposed alignment consist of peat/organics overlying deposits of silty clay, silty and gravelly sand and sand and gravel. The ground surface in the swamp area ranged between Elevations 202.0 m and 203.4 m at the borehole locations, but generally above Elevation 203 m. A thick layer of snow covered the area at the time of drilling and after the snow melted, pondered water was observed across the site and was between about 0.1 m and 0.5 m in depth. The deepest boring at this site extended to 3.4 m below ground/water surface.

4.5.1 Peat

A 0.1 m to 1.8 m thick surficial layer of brown to black, wet, fibrous peat was encountered in Boreholes H-1 to H-7. The thickness of the peat ranged from 0.2 m to 1.8 m and was typically greater than about 1 m.

SPT carried out within the peat measured 'N' values ranging from 0 blows (weight of hammer) to 1 blow per 0.3 m of penetration, indicating a very soft consistency.

Natural water contents of the samples of peat were between 75 percent and 640 percent.

A thin layer of alluvium, about 0.1 m thick, was encountered beneath the peat in Borehole H-3 at Elevation 202.0 m.

4.5.2 Silty Clay

Underlying the peat (and alluvium), a layer of wet, grey silty clay was encountered in Boreholes H-1, H-2, H-3 and H-5 and at some of the probeholes. The thickness of the silty clay ranged from 0.3 m to 0.9 m. The surface of the silty clay deposit was encountered at Elevation 202.6 m in Borehole H-1 and between Elevation 201.4 m and 201.6 m in Boreholes H-2, H-3 and H-5.

SPT 'N' values measured within the silty clay deposit were between 1 and 3 blows per 0.3 m of penetration. In situ vane testing carried out in Borehole H-1 measured an undrained shear strength of about 23 kPa, indicating a soft consistency.

Atterberg limits testing carried out on one sample of the silty clay gave a liquid limit of 39 percent, a plastic limit of 19 percent, and a plasticity index of about 20 percent. The results are shown on Figure C-1 in Appendix C and indicate that the material is a silty clay of intermediate plasticity. A grain size distribution of one sample of the silty clay is shown on Figure C-2 in Appendix C.

Natural water contents of samples of silty clay were between 43 percent and 72 percent.

4.5.3 Silty Sand to Gravelly Sand

A 0.1 m to 0.2 m thick deposit of wet, grey silty sand containing trace gravel was encountered below the silty clay in Boreholes H-1 and H-2. In Boreholes H-3, H-4, H-5 and H-6, a deposit of brown to grey gravelly sand containing some silt was encountered below the silty clay or peat. The gravelly sand was typically between 0.1 m and 0.2 m in thickness; however, in Borehole H-4,

this deposit was 1.4 m thick. A 0.2 m thick layer of sand and gravel was encountered beneath the peat in Borehole H-7.

SPT testing carried out in these deposits measure 'N' values between 28 and greater than 100 blows per 0.3 m of penetration indicating that these deposits have a compact to very dense relative density. It should be noted that the high 'N' values are likely attributed to the proximity to the inferred bedrock surface. Several instances of the sampler bouncing were encountered during sampling and have been noted on the Record of Borehole sheets.

Grain size distributions of one sample of the silty sand and one sample of the gravelly sand are shown on Figure C-3 in Appendix C.

Natural water contents of the samples of silty sand to gravelly sand were between 15 percent and 19 percent.

4.5.4 Refusal

Refusal - defined by auger refusal, spoon refusal (i.e. spoon bouncing) and/or the augers sliding - indicated probable bedrock was encountered at depths of between 0.3 m and 3.4 m below the ground surface in the borings within the swamp. The shallower depths of refusal were encountered near the edges of the swamp, closest to the bedrock outcrops.

These refusal depths, while they do not necessarily confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock surface. No bedrock coring was undertaken as part of this work.

4.5.5 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be wet and the groundwater level was generally found to be at the ground surface during drilling. This is further supported by the ponded water present at this site during the spring melt. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and the groundwater level will vary depending on precipitation and local soil permeability.

4.6 Muskoka Road 33 S-E/W Ramp (Bear Lake) – Sta. 13+340 to 13+465

The plan showing the borehole locations advanced for the Muskoka Road 33 S-E/W Ramp swamp crossing between Stations 13+340 to 13+465 are shown on Drawing D-1 in Appendix D. In addition, profiles showing the interpreted stratigraphy along the road grade and the east toe of the ramp slope are shown on Drawings D-1 and D-2. A total of 4 boreholes (Boreholes B-1 to B-4)

were completed to obtain information on the subsurface conditions within this area. The Record of Borehole sheets and the results of laboratory testing are given in Appendix D.

Boreholes B-1 and B-3 were advanced on the Highway 400 northbound paved east shoulder and Boreholes B-2 and B-4 were advanced in the swamp/lake adjacent to the toe of the existing ramp rock fill embankment. The ground surface of the roadway was between Elevation 196.1 m and 196.4 m and the ground surface in the swamp area was between Elevation 192.9 m and 193.0 m.

In general, the subsurface soils at the roadway borehole locations consist of the existing pavement structure over rock fill. The subsurface soils encountered in the swamp boreholes at the toe of the slope generally consist of peat overlying deposits of silty sand, clay and gravelly sand. Bedrock outcroppings were noted on north and south sides of the swamp/lake, as well as in the middle of the swamp/lake between about Stations 13+395 and 13+415. The deepest borehole at this site extended to 5.6 m below ground surface at the toe of the slope.

4.6.1 Pavement Structure and Fill Materials

Boreholes B-1 and B-3 encountered approximately 150 mm of asphalt underlain by 150 mm of Granular 'A' and 500 mm of Granular 'B' below the road surface.

Rock fill was encountered below the pavement structure in Boreholes B-1 and B-3. The rock fill consisted of blast rock containing sand and gravel and trace silt. The rock fill layer was 4.5 m and 2.9 m thick in Boreholes B-1 and B-3, respectively. Underlying the rock fill in Borehole B-3, a 0.9 m thick layer of fill comprised of sand and containing rock fragments was encountered.

Measured 'N' values from SPT testing within the rock and sand fill ranged from 5 blows to greater than 100 blows per 0.3 m of penetration indicating a loose to very dense relative density. The higher 'N' values are likely attributed to large pieces of gravel or rock fragments. The relative density of the rock fill is typically considered to be loose to compact. Voids were noted within the rock fill as the augers were advanced.

A grain size distribution of a sample of the blast rock fill and a sample of the sand fill is shown on Figure D-1 in Appendix D.

Natural water contents measured on samples of the fill materials were between 1 percent and 20 percent.

4.6.2 Peat

A 0.6 m to 2.1 m thick layer of peat was encountered in Boreholes B-2 and B-4 at the ground surface or below the ice surface.

SPT testing measured 'N' values that ranged from 1 blow to 9 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. The higher 'N' value is likely attributed to frozen material and therefore the 'N' values are typically less than 4 blows per 0.3 m of penetration indicating a very soft to firm consistency.

Natural water contents measured on samples of the peat were between 99 percent and 197 percent.

4.6.3 Sand (Alluvium)

A 1.8 m thick layer of wet, grey sand containing trace silt and trace organics was encountered below the peat in Borehole B-4. The surface of this deposit was encountered at Elevation 192.4 m.

SPT testing measured 'N' values that ranged from 2 blows to 9 blows per 0.3 m of penetration, indicating a very loose to loose consistency.

A grain size distribution of one sample of the sand (alluvium) is shown on Figure D-2 in Appendix D.

Natural water contents measured on three samples of sand (alluvium) were 22, 58 and 106 percent. The higher values were obtained on samples containing organic material.

4.6.4 Clay

A 1.1 m thick deposit of wet, grey clay was encountered in Borehole B-4. The surface of the clay deposit was encountered at Elevation 190.6 m.

One SPT 'N' value measured within the clay deposit was 2 blows per 0.3 m of penetration. In situ vane testing carried out in Borehole B-4 measured an undrained shear strength of about 37 kPa, indicating a firm consistency. Based on the in situ testing information and visual examination of the sample, the deposit is considered to have a soft to firm consistency.

Atterberg limits testing carried out on one sample of the clay gave a liquid limit of 59 percent, a plastic limit of 22 percent, and a plasticity index of 37 percent. The results indicate that the

material is a clay of high plasticity. The results of the testing are shown on Figure D-3 in Appendix D.

The natural water content measured on one sample of clay was 68 percent, which is higher than the liquid limit resulting in a liquidity index of 2, indicating that this deposit is a sensitive clay.

4.6.5 Silty Sand to Gravelly Sand

In Borehole B-2, a 0.5 m thick deposit of grey silty sand containing some gravel was encountered below the peat. In Borehole B-4, a 1.5 m thick deposit of gravelly sand was encountered beneath the clay. The surface of this deposit was encountered at Elevations 190.3 m and 189.5 m in these two boreholes, respectively.

Two SPT 'N' values measured within this deposit were 10 blows and greater than 100 blows per 0.3 m of penetration indicating a compact to very dense relative density. The higher 'N' value is likely attributed to the proximity to the inferred bedrock surface. Several instances of the split-spoon bouncing were encountered during sampling and have been noted on the Record of Borehole sheets.

Two grain size distribution test results on samples of the silty sand and gravelly sand deposits are shown on Figure D-4.

4.6.6 Refusal

Refusal was encountered between 3.1 m and 5.3 m of depth below the ground surface in the boreholes. These refusal depths (inferring bedrock surface by auger refusal and/or the augers sliding) indicate that there is a bedrock slope down towards Bear Lake and away from the central rock outcrop at approximately Station 13+400. In Boreholes B-1 and B-3, the boreholes were moved slightly to advance beyond the initial auger refusal depth.

These refusal depths, while they do not necessarily confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock surface. No bedrock coring was undertaken as part of this work.

4.6.7 Groundwater Conditions

In the boreholes advanced through the roadway, the groundwater level was measured at 3.7 m and 3.8 m depth below ground surface which corresponds to Elevations 192.3 m and 192.7 m. In the boreholes advanced at the toe of slope adjacent to Bear Lake, the groundwater level was found to be at about the ground surface during drilling which corresponds to Elevations 192.9 m

and 193.0 m. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and the groundwater level will vary depending on precipitation, local soil permeability and the lake water level.

4.7 Rock Cut – Muskoka Road 33 Structure (East Abutment)

A new EW-N ramp is proposed at the Muskoka Road 33 and Highway 400 interchange, which will involve widening of Highway 400 to accommodate this ramp. The location of the new ramp is shown in plan on Drawing E-1 in Appendix E.

4.7.1 Rock Mass Description

Based on field observations on May 25, 2006, the exposed existing rock cut consists of fresh, blocky, light orange-brown to grey, strong to extremely strong granite. Two or more sub-vertical joint sets of significant continuity are evident striking obliquely to the proposed access ramp excavation and Highway 400, and there is also a “foliation set” and a sub-horizontal joint surface associated with layering or folding in the rock. Mapping investigation results are presented on Figures E1 to E5 in Appendix E. There may be additional features which could not be observed as part of the rock face was obscured by debris and talus, in particular in the cut area directly under the bridge.

4.7.2 Structural Fabric

Figure E1 presents the rock-cut geometry measurements along the east rock face. The clear zone width between the edge of the road and the east abutment rock face was found to be consistently of the order of 6.9 m. Remnant half-barrels from the blast holes could be seen along the base of the rock cut. The height of the rock face varied between 1.7 m and 5.0 m.

Figure E2 presents additional views of the east side rock cut under Muskoka Road 33 and to the south of the overpass. These views illustrate the blocky nature of the rock mass and the jointing, in particular the characteristic saw tooth pattern in the rock face that tends to break back behind the original cut alignment. It is postulated that the blocks of rock that comprised these saw tooth wedge failures, formed by pre-existing joints, either fell out or were pulled out during the original excavation as they are generally too shallow to have been formed by sliding (i.e. shallower than the angle of friction for this rock mass).

Figure E3 presents views looking down onto the bedrock surface along the crest of the rock cut, highlighting the continuous joints that control the saw tooth pattern of the rock face. The two main joint sets have near vertical dips and intersect to form wedge shaped blocks. A third joint set associated with the rock foliation dips sub-vertically. A sub-horizontal joint set dipping

towards the road (shown best on Figure E1), acted as the basal release feature for these three-plane wedges.

Figure E4 presents the stereographical plots of the twenty-two (22) joint surfaces mapped. There are three main joint set: two sets (Joint 1 and Joint 2) dipping sub-vertically and a third set (Foliation 1) dipping sub-horizontally. A fourth minor joint set is seen as a dyke or vein (Vein 1) which occasional cuts steeply across the rock mass and could form an additional potential release plane. The joint orientations are as follows:

<i>Set #</i>	<i>Dip</i>	<i>Dip Dir</i>
Joint 1	90°	067°
Joint 2	87°	324°
Foliation 1 (Flat set)	13°	360°
Vein/dyke	67°	342°

Figure E5 presents a simple kinematic assessment plot using the peak orientation listed above and provides the trends and plunges of the potential wedges, such as those illustrated on Figures E-1 to E3. Note that these steep wedges require a third plane (the Flat Set) in order to daylight.

4.7.3 Groundwater Seepage

No groundwater seepage was observed along the existing east abutment rock cut on the day of the site inspection.

4.8 Closure

The field technician supervising the land portion of this drilling program was Mr. Indulis Dumpis of Golder's Sudbury office. The field engineer for the rock cut inspection of this project was Mr. Marc Rougier, P.Eng., a senior rock mechanics engineer and Associate with Golder's Mississauga office. This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Miss Sarah Poot, P.Eng., a senior geotechnical engineer from Golder's Sudbury office. Mr. Jorge Costa, P.Eng., a Principal and Designated MTO Contact for Golder, conducted a quality control review of the report.

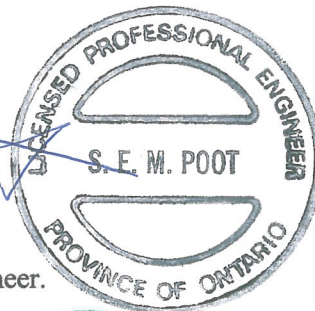
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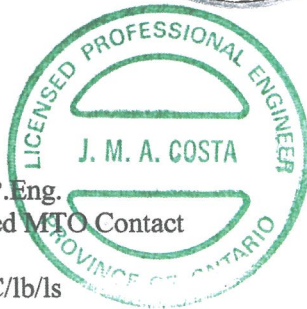
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Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Contact



AB/SEP/MR/JMAC/lb/lb

PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 400 SERVICE ROADS
FROM 3.7 KM NORTH OF MUSKOKA ROAD 48 NORTHERLY 3.6 KM
AND FROM 1.0 KM SOUTH OF MUSKOKA ROAD 33, NORTHERLY 2.1 KM
G.W.P. 5622-02-00
MINISTRY OF TRANSPORTATION, ONTARIO**

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of design of the proposed swamp crossings and rock cut based on interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations made are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

5.1 General

Golder Associates Ltd. (Golder) was retained by URS Canada Inc. (URS) to provide recommendations on foundation aspects of design and construction of the high fill embankments/swamp crossings at the areas noted below.

- Heather Path Trail Swamp/High Fill, Station 9+550 to 9+650;
- Global Tower Road Swamp/High Fill, Station 10+065 to 10+175;
- Hidden Glen Road Swamp, Station 11+100 to 11+223; and
- Hwy 400/Muskoka Road 33 S-E/W Ramp (Bear Lake), Station 13+340 to 13+465.

High fills are considered to be embankments or embankment widening greater than 4.5 m in height; whereas swamp crossings generally refer to areas where the topography is low-lying and has poor drainage, regardless of the embankment height. Based on profiles of the proposed new alignments provided to us by URS in June 2006, we understand that the new service roads will require fill embankments up to about 6.5 m in height.

We understand that with the exception of the S-E/W Ramp, the service roads will be either gravel topped or surface treated, and are expected to have low traffic volumes.

The scope of work included carrying out stability and settlement analyses, providing recommendations for stable embankment geometry and for fill materials, and providing recommendations for ground improvement techniques that may be required as a means to reduce settlements and to improve stability and addressing geotechnical related construction concerns, including sub-excavating soft/organic materials and placement of fill materials.

In addition, the scope of work required that recommendations be made for stability measures to the rock cut under the east abutment of the Muskoka Road 33 bridge structure, as part of the proposed EW-N Ramp accessing Highway 400.

Section 5.2 of this report summarizes the methods used for the analysis of stability and settlement for critical areas of embankment construction for the new service roads. Section 5.3 provides general discussions and recommendations related to techniques for minimizing potential stability and settlement-related design and construction problems. The results of the analyses and recommendations on mitigating stability and time-dependent settlements in relevant high fill embankment and swamp crossing areas are presented for each individual area in Section 5.4.

5.2 Method of Analysis

The following sections outline the methodology and parameters used to evaluate embankment stability and settlement at the various sites. The results of the analyses are presented in Section 5.4 for each high fill/swamp crossing areas where recommendations regarding possible design and construction alternatives are also given.

5.2.1 Stability Analyses

Stability analyses were performed for the critical sections of the proposed high fill embankments. For this report, critical sections are assumed to correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive materials. In all sites where cohesive strata were encountered in the subsoils, the stability of the proposed new embankment sections was analyzed using limit equilibrium methods. In sites where the subsoils consisted of cohesionless soils only, the stability of the proposed embankment section was assessed based on precedent experience in similar soil conditions.

5.2.1.1 Methodology

All slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at these sites considering the design requirements and the field data available and is based on a deep-seated, global trial failure surface that would affect the performance of the roadway. The stability analyses were performed to check that the target minimum factor of safety was achieved for the various embankment heights and geometries. In general, circular slip surfaces were analyzed in the design. Non-circular slip surfaces were not analyzed since there are no obvious thin/weaker zones within the clay deposits.

At all sites, the analyses assume that the organic soils (encountered at or below the ground surface during drilling operations) will be removed prior to construction of the new embankments (see Sections 4.0 and 5.4 for organic deposit thicknesses). For design purposes, the groundwater level was based on conditions observed during drilling and accounting for seasonal variations. Typically, the groundwater level used in the analysis was at the ground surface in each of the swamp areas.

5.2.1.2 Parameter Selection

The subsoils encountered in the various sites are composed of granular soils (silt, sand and gravel) or a combination of cohesive (clay and varved clay) and granular soils. For granular soils, effective stress parameters were employed in the analyses assuming drained conditions for the soils. The effective stress parameters (effective friction angle and cohesion) for the granular soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT), in conjunction with engineering judgement considering experience in similar soil conditions.

For cohesive deposits, total-stress parameters were employed in the analyses assuming undrained conditions for these soils. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of field vane shear tests (where applicable) and estimated from correlations with the SPT results and other laboratory test data (natural water content and Atterberg limits). For the intermediate to high plasticity or varved clays encountered, a correction factor of 0.90 was applied to the undrained shear strength estimated from the above methods in order to account for the effect of the weak clay layers interspersed between the stronger silt or clayey silt layers as well as the relative plasticity of the samples.

In some analyses, effective drained parameters were required for the cohesive deposits. In this case, the effective stress angle of internal friction (ϕ') for the cohesive soils was estimated using the following equation based on Kulhawy and Mayne (1990) for illitic soils:

$$\sin \phi' = 0.7 - 0.094 \ln PI$$

where: $PI \approx w_n - 20$

5.2.2 Settlement Analysis

The subsoils encountered in the high fill/swamp areas are composed of either granular or cohesive soil deposits over bedrock at shallow depth. Thin, surficial deposits of soft organic soils (i.e. topsoil and/or fibrous peat) were encountered in most of the boreholes. At the Muskoka Road

33 S-E/W Ramp (Bear Lake) site, the existing highway embankment consists rock fill over sand and gravel fill at some locations. Granular soils will typically settle rapidly under the embankment loading (i.e. during construction) while the settlement of cohesive soils is typically time-dependent and will occur over the long term.

5.2.2.1 Methodology

Settlement analyses were performed on the critical sections of the proposed fill embankments. The settlement analysis was performed using standard equations from literature. The sources of settlement were considered to include:

- primary time-dependent consolidation of the cohesive deposits;
- secondary time-dependent (creep) consolidation of the cohesive deposits (long term);
- immediate settlement of the native granular soils and existing fill material; and
- self-weight compression of the new embankment fill materials.

The thicknesses of the compressible strata are variable in each swamp area which implies that the consolidation settlement along the length of the swamp will also be variable. In addition, the proposed embankment height is typically variable along the length of each swamp/high fill area.

At all areas, the settlement analyses assume that organic soils will be removed prior to construction of the proposed embankments. For design purposes, the groundwater level was based on conditions observed during drilling and accounting for seasonal variations. Typically, the groundwater level used in the analysis was at the ground surface in each of the swamp areas.

It is known that some consolidation settlement occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the partially over-consolidated clays but will generally constitute a smaller fraction of the total settlement of the consolidating layer. Secondary settlement analyses has been included in the analyses. The following equations for secondary (creep) settlement from Holtz and Kovacs (1981) were employed in the analyses.

$$S_c = C_{\alpha\epsilon} \times L_o$$

$$C_{\alpha\epsilon} = \frac{w_n}{100}$$

where :

S_c	=	secondary (creep) settlement (mm)
$C_{\alpha\epsilon}$	=	modified secondary compression index (%)
L_o	=	initial thickness of compressible clay deposit (mm)
w_n	=	natural water content (%)

5.2.2.2 Parameter Selection

The immediate compression of the very loose to very dense silt, sandy silt to silty sand, sand, and gravel layers was modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

Settlement analyses were carried out using the results of borehole information, in situ field test data and laboratory results of water content and Atterberg limits determinations.

The over-consolidation ratio (OCR) profile required in the settlement analyses was established using the results of correlations with the results of the in situ vane shear tests. The following correlation relating in situ undrained shear strength to preconsolidation pressure (Mesri, 1975) was employed:

$$s_u = 0.22\sigma_p'$$

where :

$$\begin{aligned} s_u &= \text{average mobilized undrained shear strength (kPa)} \\ \sigma_p' &= \text{preconsolidation pressure (kPa)} \end{aligned}$$

The compression index and recompression index profiles required in the analysis were established using correlation with laboratory test data. The following published correlation (Kulhawy and Mayne, 1990) relating the Plasticity Index to the compression and recompression indices was used to determine the preliminary correlation:

$$\begin{aligned} C_c &= I_p/74 \\ C_r &= C_c/10 \end{aligned}$$

where :

$$\begin{aligned} C_c &= \text{compression index} \\ C_r &= \text{recompression index} \\ I_p &= \text{plastic limit (\%)} \end{aligned}$$

The coefficient of consolidation, c_v , required in the analysis was established using the results of the correlation with liquid limit (U.S. Navy NAVFAC 1971).

5.2.3 Fill Selection

Consideration could be given to the use of earth fill or rock fill in construction of the new and widened embankments.

The advantages of using rock fill as embankment fill is that there may be surplus rock fill from other parts of the project (including the existing stockpile, as noted in the RFP) thereby making

use of rock fill more economical than using imported earth fill. In addition, relatively steep embankment side slopes (1.25H:1V) are possible in rock fill thus reducing the overall quantity of material required for the project. The main disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the rock fill itself will occur, as discussed below. From a constructability perspective, there is typically better control over placement of rock fill below the water table, if required, in comparison to earth fill. It should be noted that it may be difficult to use rock fill for the S-EW Ramp widening due to the benching required into the existing embankment.

The main advantage of using earth fill is that settlement would occur during construction and a suitable local source may be available. The disadvantages of using earth fill are that placing earth fill below the water table is difficult and additional settlement could occur as a result of uncontrolled placement and compaction techniques. Also, flatter side slopes are required (2H:1V), even flatter below the water level, compared to rock fill. A geotextile separator may also be required where earth fill is placed over rock fill (i.e. widening of S-E/W Ramp).

The stability and settlement analyses have been carried out for both fill types. Where earth fill is used in the analyses, we assume that it will be granular material (i.e. non-cohesive).

5.2.3.1 Earth Fill

Where earth fill is used for embankment construction, a unit weight of 21 kN/m^3 and a friction angle of 30 degrees have been employed together with side slopes of 2H:1V.

Settlement of earth fill is dependent on the percentage of granular material within the fill. If the earth fill is comprised solely of granular fill (i.e. Granular B Type II), then the settlement will typically occur during construction, assuming proper placement and compaction procedures are followed. If the embankment fill is comprised primarily of cohesive soils (not recommended) then additional long-term settlement may result. Where earth fill is placed below the water level, additional settlement may occur as a result of uncontrolled placement.

5.2.3.2 Rock Fill

Where rock fill is used for the construction of the embankments, a unit weight of 19 kN/m^3 and a friction angle of 40 degrees have been employed, together with side slopes of 1.25H:1V.

Settlement of the rock fill depends on the method and sequence of placement and compaction of the rock fill, as well as whether the fill is being placed below or above the groundwater level (i.e. where sub-excavation has been carried out or infilling into water). Settlement of the rock fill will

occur both above the water level where it can be properly placed and compacted and below the water level where end-dumping procedures must be used.

The data contained in the document entitled “Rockfill in the Foundation Design of Highway Structures” by the Ministry of Transportation and Communications, Research and Development Branch, dated 1982, was used to establish the relative percentages for varying rock fill embankment heights. The post-construction settlement depends on the method of placement; the two methods are discussed as follows:

1. **Compacted Rock Fill:** Compacted rock fill is placed in regular lifts (i.e. 1.5 m maximum lift thickness) and in accordance with Special Provision SP206S03, dated July 2006. This would be the type of method used to construct rock fill embankments above the existing ground surface.
2. **Dumped Rock Fill:** This is rock fill that is end-dumped into place assuming little or no control over the compaction. This method would be used for backfilling the sub-excavated area below the water table.

Long-term post-construction settlement may occur as a result of time-dependent creep due to rearrangement of rock particles under load and breakage of rock particles (i.e. local crushing and degradation). The majority of this settlement (approximately 60% - MTO, 1982) will occur in the first year following construction.

Presented below are estimated long-term post-construction settlements as a percentage of total embankment height for a range of embankment heights/rock fill thickness and the two methods of placement (taken from Figure 20 of the 1982 MTO report). For intermediate embankment heights, the percentage can be determined by interpolation. It should also be noted that additional platform width would be required to accommodate the expected settlements.

<i>Embankment Height/Sub-excavation Depth (m)</i>	<i>Rock Fill Consolidation Settlement (%)</i>	
	<i>Compacted (embankment construction)</i>	<i>Dumped (sub-excavation backfill)</i>
5	0.4	0.9
10	0.8	1.8
15	1.2	2.7
20	1.6	3.6

5.2.4 Seismic Analysis and Liquefaction Potential

The liquefaction potential of the granular soils below the immediate approach embankments and other high fill embankment areas under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *Canadian Highway Bridge Design Code* (CHBDC)

Commentary, which correlates the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) of the soils with their normalized penetration resistance and fines content for granular soils. The CRR has been determined using the empirical method suggested by the CHBDC based on papers by Seed et al (1984) using SPT 'N' values and accounting for fines content. The method used to determine the CSR will be the simplified procedure suggested by Seed and Idriss (1971) relating to the peak ground acceleration and effective overburden stress.

In general, geologically young, loose deposits of sand and non-plastic silt sands with low fines content (less than 5 percent passing No. 200 sieve) which are below the water table are potentially susceptible to liquefaction.

The liquefaction potential for cohesive soils is determined using the Chinese criteria proposed by Seed et al (1973) and Wang (1979) in the CHBDC correlating liquid limit, percent fines and water content. Clayey soils that have percent fines less than 15 percent, liquid limit less than 35 percent and natural water content greater than 0.9 times the liquid limit, could be potentially susceptible to liquefaction.

5.2.4.1 Liquefaction Induced Settlements

Where liquefaction is identified to be a problem either in clayey soils or in granular soils using the methods described above, vertical deformation of the soil under the earthquake loading may occur due to the contraction of the sand deposit using the method developed by Tokimatsu and Seed (1987) as noted in the CHBDC. This deformation can be estimated using relationships proposed by Makdisi and Seed (1978) as noted in the CHBDC. If deformation is anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

5.2.4.2 Stability under Seismic Conditions

The susceptibility of granular deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions has been assessed. The peak zonal acceleration for this site (Parry Sound) is 0.065g, which is based on a zonal acceleration of 0.05g multiplied by an amplification factor of 30 percent for the types of soils found at the high fill and swamp sites in this area. Typically, for the swamp areas with deposits of soft clay, it is assumed that the seismic loading will be applied to the long-term (drained) conditions, where consolidation of the clay has taken place under the embankment loading. However, although long-term conditions are assumed, undrained parameters are used due to the short-term nature of seismic events where it is assumed that drainage of the clay and dissipation of pore pressures will not take place.

If liquefaction of the subsoils under the embankment loading is not anticipated, a factor of safety of 1.0 is typically used to assess the stability under magnitude 7.0 earthquake events.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual strength parameters in the liquefied layers using the correlation as noted in the CHBDC proposed by Seed and Harder (1990) which is correlated to SPT 'N' values. If, under these conditions, the embankment is estimated to have a factor of safety less than 1.0 under static conditions, the embankment is considered to be susceptible to a flow slide. Flow slides are characterized by very large lateral and vertical displacements of the embankment. If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and are estimated using the Newmark method, which compares the design ground acceleration to that necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration). If the yield acceleration is greater than the maximum acceleration for this site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration, soil improvement methods may be necessary to improve soil conditions.

5.3 Stability and Settlement Control Methods

At each swamp location, settlement and/or stability could be a concern. A brief discussion on the various mitigation alternatives is given below. A detailed description of the feasible mitigation alternatives (if required) for each swamp area is given in Section 5.4.

5.3.1 Full Sub-excavation

In order to control the stability and settlement of the proposed embankments, sub-excavation of the organics and soft, compressible, cohesive soils may be carried out. Since most of the swamps consist of thin soil deposits over bedrock at shallow depth, this alternative will likely be the most practical and economical and will typically provide the best technical solution in terms of the stability and long-term performance of the roadway. If full sub-excavation is considered for the widening area, there may be concerns with respect to stability of the existing roadway.

The bedrock or refusal stratum is typically located less than about 6 m below the ground surface in the swamp areas. The groundwater level is typically located at the existing ground surface and the sub-excavation would be carried out below the water table. In the widening area, the bedrock is located about 5 m below the lake water surface.

Side slope profiles will be as flat as 4H:1V below the water table in order to maintain the stability of the sub-excavation and that may affect property requirements; the actual slope profiles are given for each swamp area. In the case where sub-excavation is carried out adjacent to an

existing road embankment (i.e. Bear Lake), temporary roadway protection and un-watering may be required. Protection of this embankment may also prohibit full sub-excavation.

Adopting the sub-excavation alternative for minimization of settlement will result in increasing the effective thickness of the new embankment because of the additional backfill required below the existing ground surface. The increase in fill thickness will result in additional post-construction settlement of the embankment rock fill (see Section 5.2.2).

For the embankment widening, the additional below-grade rock fill should be constructed with the same side slope profile as that used for the above grade embankment (i.e. 1.25H:1V). We do not recommend the use of earth fill for “below water” filling.

5.3.2 Toe Berms and Preloading

Preloading involves constructing the embankment and allowing it to consolidate the subsurface soils below the embankment for a specified period of time - the preload period - prior to final subgrade construction and paving. The actual preload period required will depend on the compressibility parameters of the subsoils. Typically, the preload period continues until 90 to 95 percent of the estimated primary settlement is completed; however, the preload period could be extended depending on the results of monitoring (see Section 5.3.7). It should be noted that some long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option.

If the construction schedule can accommodate the specified preload period, preloading the foundation soils by building the embankment as early as possible can be considered as a feasible alternative for a given swamp crossing.

In order to maintain stability of the embankments at some swamp crossing locations, or if surcharge loads are added, toe berms may have to be constructed. Toe berms consist of additional embankment materials placed near the bottom of the slope. Such berms typically have a cross-section with a relatively flat slope near the new high embankment and will be on the order of one third to one half of the main embankment height. The lateral extent of toe berms varies depending on stability analyses but can range from about equal to the embankment height up to about twice this value. Thus, the use of toe berms results in a stepped embankment cross-section geometry. This stepped configuration produces a similar effect on stability as using flatter embankment slopes, but often requires less fill material. Depending on the results of analyses and subsoil conditions, the toe berms may be removed after sufficient time has passed if their necessity is for temporary stability only and property restrictions require their removal. Alternatively, consideration could be given to constructing the embankment in stages as discussed further in Section 5.3.4.

5.3.3 Surcharging

Surcharging an embankment involves placement of additional load onto the embankment above the final grade. The surcharge material has to be removed after the end of the preload period and prior to construction of the pavement substructure.

Since this process temporarily increases the embankment height, toe berms may be necessary to maintain stability of the embankments constructed to the top-of-surcharge grade.

The advantage of adding a surcharge is to reduce the preload period (i.e. time for 90 percent consolidation settlement). It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option.

5.3.4 Lightweight Fill

Another alternative for reducing the magnitude of long-term settlement and improving stability at the swamp crossings would be to use lightweight fill in a portion of, or all of, the embankment. Lightweight fill could consist of either blast furnace slag fill or expanded polystyrene (EPS) fill.

Lightweight fill materials are typically an order of magnitude more costly than standard rock or earth fill. Part of the reason for the high cost is that lightweight fill materials are not locally available and will have to be shipped to the site. For this reason, lightweight fill is typically not considered an economically suitable option, however, could be considered if other options such as sub-excavation are not considered practical. Further discussion will be given for each swamp if this is considered to be a practical alternative.

5.3.5 Staged Construction

As cohesive soils consolidate (or settle) beneath imposed loads, the additional stress on the ground causes the ground strength to increase over time. Initially, when new loads are imposed on soft cohesive soils, the water pressure within the soil increases to match the new loads. As the pressures dissipate, the soils then consolidate. The time required for strength gain is dependent upon the time required for the induced pore water pressures to partially or fully dissipate. Constructing the embankment in stages often allows the subsoils to gain strength between filling stages, thereby improving stability and limiting the requirement for toe berms. Each stage of embankment construction must be left to rest for a specified period of time to allow for sufficient strength gain before construction of the next stage can begin. Typically, staged construction is used in conjunction with preloading and surcharging methods, where the preloading and surcharging are used to reduce long-term settlement and staged construction is used to maintain stability.

It should be noted that monitoring of the settlement and dissipation of the excess pore water pressures would be required to check that adequate consolidation had occurred prior to proceeding with the subsequent construction stages (see Section 5.3.7). It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option.

5.3.6 Wick Drains

In addition to the preloading/surcharging/staged construction alternatives discussed above, wick drains may be installed to reduce the preload period. Wick drains are prefabricated geotextile strips installed through the clay deposit at a 1.5 m (typical) triangular grid spacing across the embankment area. The installation of these strips increases the drainage area through the clay deposit thus speeding up the consolidation process. The advantage of wick drains, in addition to reduction of the preload period, is that the number of construction stages required could also be reduced. Monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation had occurred prior to proceeding with the subsequent construction stages. It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option.

A granular drainage blanket is required on the subgrade (at the base of the embankment) to promote drainage away from the wick drain area and, depending on the thickness required, could be constructed using Granular 'B' Type I or sand to also facilitate the installation of the wick drains. It should be noted that, depending on the consistency of the surficial soils and the thickness of the drainage blanket at each swamp crossing site, installation of the wick drains may be difficult and pre-drilling may become necessary in some locations if wick drains are used. In all cases, stripping of the peat/organics at the ground surface will be required (see Section 5.5).

Typically, wick drains are considered feasible where the total length of the drainage path can be reduced significantly in order to reduce the time required for consolidation. At the swamp sites for this project, the maximum thickness of clay is less than 3.5 m and, therefore, the use of wick drains is impractical as it would not significantly reduce the drainage path.

5.3.7 Instrumentation and Monitoring

In order to assess the magnitude and time rate of settlement during and after construction of all of the high fill embankments and embankments over swamps, settlement and pore pressure monitoring will be required, except in the case of full sub-excavation.

The monitoring program should consist of a series of settlement plates and settlement cells within the embankment which would be surveyed at regular intervals during and after construction, for

the duration of the preloading and/or surcharge period. In addition, vibrating wire piezometers should be installed at specified locations along the embankments to monitor the dissipation of excess pore pressures during staged construction or where wick drains are used. Standpipe piezometers are required to calibrate the vibrating wire piezometers.

The frequency of reading of the monitoring instruments will depend on the type of mitigating alternative chosen for a given site and the height of the proposed embankment. Detailed requirements regarding the number and layout as well as specifications for the supply, installation, protection and monitoring of the instruments should be included as a special provision (SP) in the contract; and SP can be provided if this option is considered in the final design.

The monitoring should be carried out by a monitoring specialist retained by MTO who would be responsible for obtaining the survey and piezometer readings and interpretation and reporting of the data. The installation of the instruments will be the responsibility of the contractor.

5.4 Results of Analysis

5.4.1 Summary of Alternatives

Details of the settlement and stability analyses results, as well as the mitigation alternatives, are given in the sections that follow. A summary of the recommended foundation treatment for each swamp/high fill area is given in Table 1 and summarized below.

At the Heather Path Trail swamp/high fill area, we recommend preloading the embankment with a 300 mm surcharge for a 3 month period. The post-construction settlement (after surcharge removal) is estimated to be less than 50 mm. Given that the road will be gravel topped, this amount of post-construction settlement is considered acceptable.

At the Global Tower Road swamp area, due to stability concerns, we recommend sub-excavation of the soft clay deposit prior to construction of the embankment.

At the Hidden Glen Road and swamp area there are no special settlement or stability mitigation requirements. Since the road will be surface treated, the estimated settlement of less than 25 mm is considered acceptable.

For the Muskoka Road 33 S-E/W Ramp (Bear Lake), we recommend sub-excavation of the organics/soft clay prior to construction of the embankments in all the swamps to enhance stability and long-term performance of the roadway as well as to reduce anticipated construction schedule and overall costs.

5.4.2 Heather Path Trail Station 9+510 to 9+650

The swamp in this area extends from Station 9+510 to 9+650 along the new alignment of Heather Path Trail. This swamp area is surrounded by rock outcrops at the north and south ends of the swamp as well as an outcrop on the northwest side of the swamp. A snowmobile trail intersects the swamp at the north end. Open water was present to the east of the alignment. The ground surface within the swamp is at about Elevation 194 m. The average overall length of this swamp is about 140 m. The proposed embankment ranges from less than 2 m in height at the south end of the swamp up to about 6.2 m in height towards the north end of the swamp.

The subsoils in this low-lying swamp area consist of up to 0.6 m of peat and/or topsoil underlain by up to about 2.3 m of soft to stiff compressible clay soil. A thin layer of dense sand and gravel was present below the clay to depths up to 3.4 m below the ground surface. Closer to the rock outcrops, the sand and gravel deposit was located directly below the organic deposits. Based on SPT 'N' values and visual observations, the undrained shear strength of the clay deposit is estimated to range between 25 kPa and 75 kPa. Groundwater was encountered at levels between ground surface and about 1.0 m depth. For details of the subsurface conditions, refer to Section 4.3. A summary of the engineering parameters used in the analysis is given in Table 2.

5.4.2.1 Stability

Constructing the proposed 6.2 m high earth fill or rock fill embankment on the existing ground conditions is estimated to result in an estimated factor of safety against instability greater than 1.3 during construction without implementation of ground improvement or settlement mitigation measures. The results of the limit equilibrium stability analysis for the existing soil conditions in this area are presented on Figures 1 and 2 for earth fill and rock fill embankment construction, respectively.

Using the methods outlined in Section 5.2.4, the soils at this site are not considered to be liquefiable. A factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

5.4.2.2 Settlement

The total estimated settlement of the foundation soils and embankment fill is summarized below for the most critical embankment section at Station 9+630, where the embankment loading is highest (6.2 m). At this section, the soft to firm clay deposit is 1.7 m thick. The clay deposit is thickest (2.3 m) at Station 9+550; however, the proposed embankment at this location is only 2 m in height and therefore does not represent the critical section. It should be noted, however, that

the values of settlement indicated below must be considered approximate because of the natural variability of the soil and limitations of such geotechnical analyses.

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	1.8 m	10 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	1.7 m	130 mm	About 95% occurs within 6 months
Long-term Creep ⁽²⁾	1.7 m	10 mm	About 50% occurs within 50 years.
Earth Fill Embankment	6.2 m	20 mm	Occurs rapidly (during construction).
Rock Fill Embankment	6.2 m	30 mm	About 60% occurs within the first year.
Total Estimated Settlement		170 mm - 180 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			110 to 180 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.2.3 Alternatives for Mitigation of Settlement/Stability

The presence of the approximately 2 m thick soft clay layer influences both the stability and magnitude of post-construction settlement of the proposed up to 6.2 m high embankment in this area as discussed above. Alternatives for minimizing post-construction settlements as discussed below should be considered. The alternatives described below have been evaluated on the basis of advantages, disadvantages, relative costs and risks/consequences and are summarized in Table 3.

5.4.2.3.1 Sub-excavation

In order to minimize the post-construction settlement of the embankment, full sub-excavation of the soft clay and replacement with rock fill may be carried out in this swamp area. The excavation would have to extend to depths up to about 3.1 m. Given that the depth at which refusal was encountered in the boreholes/probeholes within the swamp area is up to 3.4 m, consideration could be given to full sub-excavation of the overburden down to the bedrock surface. The excavation will be carried out below the water table and side slopes within the overburden materials should be constructed at no steeper than 3H:1V.

Although sub-excavation will eliminate the post-construction settlement of the soft clay, settlement of the rock fill used as backfill will occur (see discussion in Section 5.2.3.2). The total estimated settlement of the foundation soils and rock backfill are given in the table below:

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	1.8 m	10 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	0 m	0 mm	n/a
Long-term Creep ⁽²⁾	0 m	0 mm	n/a
Earth Fill Embankment	6.2 m	20 mm	Occurs rapidly (during construction).
Rock Fill Embankment	6.2 m	30 mm	About 60% expected to occur within the first year.
Rock Fill Backfill	3.4 m	20 mm	
Total Settlement		50 mm - 60 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			10 mm to 20 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.2.3.2 Preloading/Surcharging/Staged Construction

Another alternative to reduce the magnitude and rate of post-construction settlements is to preload the embankment in conjunction with adding a surcharge. Placing a surcharge load on the embankment will also assist with further reducing long-term creep settlements.

Preloading

Preloading the embankment would be used to reduce the magnitude of post-construction settlement under the embankment. About 95 percent of the primary consolidation settlement is expected to occur within about 6 months. If the construction schedule can accommodate this time period, preloading the foundation soils by building the embankment as early as possible can be considered as an alternative to sub-excavation. The magnitude of post-construction settlement at the end of the preload period (including creep) is estimated to be less than 15 mm to 25 mm depending on the embankment construction material. Monitoring would be required to assess the magnitude and time rate of settlement during and after construction as discussed in Section 5.3.

Surcharging/Toe Berms

The surcharge load would be placed to reduce long-term post-construction creep settlements. The addition of surcharge load would cause instability in the embankment and, as such, the use of toe berms is recommended. For a 2 m high surcharge on the 6 m high earth fill embankment, construction of toe berms 5 m wide and 3 m high (as shown on Figure 3) would result in a factor of safety of greater than 1.3. For a rock fill embankment with a 2 m high surcharge, the width of the toe berm can be reduced to 3 m. The toe berms should be constructed to full width at the

north end of the swamp then can be tapered to 0 where the final embankment is less than 6 m in overall height. The toe berms could be removed after the surcharge is removed, if desired.

The addition of the 2 m surcharge load would reduce the preload period from 6 months to 3 months to achieve 95% primary consolidation as shown on Figure 4. The magnitude of post-construction settlement at the end of the preload period is estimated to be less than 25 mm.

Given the type of road (i.e. gravel) for Heather Path Trail, a greater amount (greater than 25 mm) of post-construction settlement should be tolerable. In this regard, consideration could be given to the addition of a 300 mm surcharge left in place for 3 months. With this amount of surcharge, there is no requirement for stability berms. Over this duration of surcharge, about 80% of the primary consolidation would be achieved (as shown on Figure 4) and up to 50 mm of post-construction settlement would occur which should be tolerable for this road. Settlement monitoring is not required for this case.

Staged Construction

If material is not available for toe berm construction, then consideration could be given to staged construction of the surcharged embankment; i.e. constructed in stages and the excess porewater pressure is allowed to dissipate before continuing with the subsequent lifts of fill. The embankment could be constructed to 6 m height and then left for a minimum of 2 months prior to adding the final 2 m surcharge fill. In this case, a minimum of 4 months would be required for preloading/surcharging. The wait times are subject to monitoring as discussed in Section 5.3.

5.4.2.3.3 Wick Drains

Another alternative to reduce the magnitude and rate of post-construction settlements as well as to enhance stability to reduce the time required for settlements to occur is to preload the embankment in conjunction with adding a surcharge, and installing wick drains beneath the embankment. However, due to relatively thin clay deposit (less than about 2.3 m) and depth to refusal (less than 3.4 m), wick drains are considered impractical at this site as they will not significantly reduce the drainage path.

5.4.2.3.4 Lightweight Fill

If the settlement discussed above cannot be tolerated by the embankment, then consideration could be given to the use of lightweight fill. Lightweight aggregate such as slag would reduce settlements by about 60%; lightweight EPS fill would reduce settlements to less than 25 mm. However, the cost of embankment construction using lightweight fill material is typically on an order of magnitude higher than other options.

5.4.3 Global Tower Road – Sta. 10+065 to 10+175

The swamp in this area extends from Station 10+065 to 10+175 along the new alignment of Global Tower Road. This swamp area consists of low-lying flooded terrain with rock outcrops at the north and south ends of the swamp. The ground surface within the swamp is at about Elevation 199 m. The average overall length of this swamp is about 110 m. The proposed embankment ranges between about 4 m and 5 m in height along the length of the swamp.

The subsoils in this low-lying swamp area consist of up to 1.2 m of peat/organics underlain by up to about 3.6 m of very soft to firm compressible silty clay to clay soil. A thin layer of dense sand and gravel was present below the clay to depths up to 5.0 m below the ground surface. Laboratory vane testing in two Shelby tube samples of the silty clay to clay deposit gave undrained shear strength values of 6 kPa and 30 kPa. Groundwater was generally encountered at about the ground surface. For details of the subsurface conditions, refer to Section 4.4. A summary of the engineering parameters used in the analysis is given in Table 4.

5.4.3.1 Stability

The embankment height at the south end of the swamp is a maximum of 5 m. At this end of the swamp, the very soft to firm clay deposit is less than 1.5 m in thickness. Towards the central and north end of the swamp (i.e. north of Station 10+100), the embankment height is about 4 m and the clay deposit is as thick as 3.6 m. Embankments constructed out of rock fill or earth fill will have an estimated factor of safety less than 1.3 m. The maximum stable embankment height for a factor of safety greater than unity is 1.5 m. In order to obtain a factor of safety of about 1.3 at the conclusion of consolidation (settlement) for the 4 m to 5 m high embankment, ground improvement or settlement mitigation measures, as discussed in Section 5.4.3.3, will be required to avoid inducing instability when the embankment is constructed above 1.5 m in height.

Using the methods outlined in Section 5.2.4, the soils at this site are not considered to be liquefiable. A factor of safety of less than 1.0 is obtained for magnitude 7.0 earthquake events and as a result, stability mitigation measures as discussed in Section 5.4.3.3 will be required.

5.4.3.2 Settlement

The total estimated settlement of the foundation soils and embankment fill is summarized below for the most critical embankment section at Station 10+125, where the clay deposit is thickest at 3.6 m. At this location, the embankment height is 4.0 m. Although the embankment loading is higher (up to 5.0 m) at other locations, the thickness of clay is much less and therefore not considered to be the critical section. It should be noted, however, that the values of settlement

indicated below must be considered approximate because of the natural variability of the soil and limitations of such geotechnical analyses.

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	0.3 m	0 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	3.6 m	500 mm	About 95% occurs within 6 months
Long-term Creep ⁽²⁾	3.6 m	25 mm	About 50% occurs within 50 years.
Earth Fill Embankment	4.0 m	15 mm	Occurs rapidly (during construction).
Rock Fill Embankment	4.0 m	15 mm	About 60% occurs within the first year.
Total Estimated Settlement		540 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			410 to 625 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.3.3 Alternatives for Mitigation of Settlement/Stability

The presence of the approximately 3.6 m thick soft clay layer influences both the stability and magnitude of post-construction settlement of the proposed 4 m to 5 m high embankment in this area as discussed above. Due to stability concerns of the embankment constructed over the very soft to firm clay, the only feasible mitigation alternative is sub-excavation of the clay as discussed below.

5.4.3.3.1 Sub-excavation

In order to minimize the post-construction settlement of the embankment, full sub-excavation of the soft clay replacement with rock fill would need to be carried out in this swamp area. The excavation would have to extend to depths up to about 4.7 m. It should be noted that, in general, the depth to refusal was less than 2.5 m across the majority of the swamp area and only isolated areas were as deep as the maximum 5 m and therefore consideration could be given to full sub-excavation of the overburden down to the bedrock surface. The excavation would be carried out below the water table and side slopes within the overburden materials should be constructed at no steeper than 3H:1V.

Sub-excavation of the clay will increase the factor of safety to greater than 1.3 for global stability of the embankment. A factor of safety of greater than 1.0 is also obtained for magnitude 7.0 earthquake events.

Although sub-excavation will eliminate the post-construction settlement of the soft clay, settlement of the replacement rock fill settlement will occur (see discussion in Section 5.2.3.2). The total estimated settlement of the foundation soils and rock backfill are given below:

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	0.3 m	0 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	0 m	0 mm	n/a
Long-term Creep ⁽²⁾	0 m	0 mm	n/a
Earth Fill Embankment	4.0 m	15 mm	Occurs rapidly (during construction).
Rock Fill Embankment	4.0 m	15 mm	About 60% expected to occur within the first year.
Rock Fill Backfill	5.0 m	45 mm	
Total Settlement		60 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			< 25 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.3.3.2 Other Alternatives

The sub-excavation alternative is considered to be the only feasible alternative that will lead to a stable embankment. Staged construction would require lengthy wait times to allow the embankment to be constructed to the final height; construction could take between one and two years, depending on the length of time for dissipation of excess pore pressure for each stage as determined from monitoring data. The use of wick drains may reduce these wait times slightly, however, due to relatively thin clay deposit and depth to refusal, wick drains are considered impractical at this site as they will not significantly reduce the drainage path. Lightweight fill could be considered but would increase the costs by at least an order of magnitude over sub-excavation.

5.4.4 Hidden Glen Road – Sta. 11+100 to 11+223

The swamp in this area extends from Station 11+100 to 11+223 along the new alignment of Hidden Glen Road. This swamp area is a low lying area of ponded water with mixed forest to the north and bedrock outcrops to the south of the site. The ground surface within the swamp is at about Elevation 203.2 m. The average overall length of this swamp is about 115 m. The proposed embankment ranges from less than 1 m at the south end of the swamp up to about 2 m (above the water surface) towards the north end of the swamp.

The subsoils in this low-lying swamp area consist of up to 1.8 m of peat and/or organic deposits underlain by less than 1 m of soft compressible clay soil. A thin layer of very dense silty sand and gravel was present below the clay to depths up to about 3 m below the ground surface. Closer to the rock outcrops, the sand and gravel deposit was located directly below the organic deposits. The undrained shear strength of the clay deposit is estimated to be between 20 kPa and 50 kPa based on in situ vane testing and empirical correlations. Groundwater was encountered at about the ground surface and in the areas with ponded water, the depth of the water was between 0.3 m and 1.2 m. For details of the subsurface conditions refer to Section 4.5. A summary of the engineering parameters used in the analysis is given in Table 5.

5.4.4.1 Stability

Constructing the proposed 2 m high rock fill or earth fill embankment on the existing ground conditions is estimated to result in a factor of safety against instability of greater than 1.3 during construction without implementation of ground improvement or settlement mitigation measures. The results of the limit equilibrium stability analysis for the existing soil conditions in this area are presented on Figure 5 for earth fill.

Using the methods outlined in Section 5.2.4, the soils at this site are not considered to be liquefiable. A factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

5.4.4.2 Settlement

As a consequence of the low embankment height and the relatively thin clay layer, short-term and long-term consolidation settlements of the clay deposit are expected to be minimal.

The total estimated settlement of the foundation soils and embankment fill is summarized below for the most critical embankment section at Station 11+205, where the embankment loading is highest and where the soft clay deposit is thickest. It should be noted, however, that the values of settlement indicated below must be considered approximate because of the natural variability of the soil and limitations of such geotechnical analyses.

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	1.4 m	10 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	0.9 m	15 mm	Occurs rapidly (during construction).
Long-term Creep ⁽²⁾	0.9 m	0 mm	n/a
Earth Fill Embankment	2.0 m	5 mm	Occurs rapidly (during construction).
Rock Fill Embankment	2.0 m	5 mm	About 60% occurs within the first year.
Total Estimated Settlement		35 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			< 25 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.4.3 Alternatives for Mitigation of Settlement/Stability

Based on the soil conditions at the site and due to the relatively low proposed embankment, it is not anticipated that there will be embankment settlement or stability issues at this site. The total estimated post-construction settlement of the subsoils is less than about 25 mm which is within the tolerance for settlement for this type of road (i.e. surface treated).

Using the methods outlined in Section 5.2.4, the soils at this site are not considered to be liquefiable. A factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

5.4.5 Muskoka Road 33 S-E/W Ramp – Sta. 13+340 to 13+465 (Bear Lake)

The swamp/lake in this area extends from Station 13+340 to 13+465 along the re-aligned S-E/W Ramp to Muskoka Road 33. The existing highway ramp embankment is up to about 4 m above the water level in Bear Lake, which is present immediately east of the ramp slope. It is proposed that the ramp alignment be shifted, resulting in embankment widening of up to 10 m. The shoreline of Bear Lake, where it intersects the ramp, is interspersed with rock outcrops to the north, south and centre of this section. The ramp road grade ranges from Elevation 196 m to 197 m from south to north, while the water level/ice surface in Bear Lake (at the time of drilling) was at about Elevation 193 m. The overall length of the ramp where it extends along Bear Lake is about 125 m.

The existing embankment fill is comprised of rock fill mixed with sand and gravel fill. The subsoils at the toe of the slope at Bear Lake consisted of up to 2.1 m of peat and/or topsoil underlain by up to about 2.1 m of very loose silty sand to sand. The sandy deposit was underlain by about 1 m of soft to stiff compressible clay soil. The undrained shear strength of the clay

deposit is estimated to be between 25 kPa and 50 kPa, based on in situ vane testing and empirical correlations. An up to 1.4 m thick layer of dense gravely sand was present below the clay to depths up to 5.0 m below the ground surface at the toe of the slope. Groundwater was encountered at the ground surface at the toe of slope and within the existing rock fill embankment corresponding to about the water elevation in Bear Lake. For details of the subsurface conditions, refer to Section 4.6. A summary of the engineering parameters used in the analysis is given in Table 6.

5.4.5.1 Stability

Constructing the proposed 4 m high widened rock fill or earth fill embankment on the existing ground conditions is estimated to result in a factor of safety against instability of greater than 1.3 during construction without implementation of ground improvement or settlement mitigation measures. The results of the limit equilibrium stability analysis for the existing soil conditions in this area are presented on Figure 6 for earth fill embankment construction.

Using the methods outlined in Section 5.2.4, the very loose sands under the organic material are subject to liquefaction. Using liquefied or reduced parameters within the sand, a factor of safety of greater than 1.0 is obtained and, therefore, the liquefied soils would not pose a significant problem during a magnitude 7.0 earthquake events as shown on Figure 7. However, sub-excavation of these surficial loose sands should be considered as discussed in Section 5.4.5.3.1 below.

5.4.5.2 Settlement

The total estimated settlement of the foundation soils and embankment fill is summarized below for the most critical embankment section at Station 13+370, where the embankment loading is highest and/or where the soft clay deposit is thickest. It should be noted, however, that the values of settlement indicated in the tables below must be considered approximate because of the natural variability of the soil and limitations of such geotechnical analyses. It should be noted that this settlement will be differential with respect to the existing roadway.

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	3.4 m	20 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	1.1 m	35 mm	About 95% occurs within 3 months
Long-term Creep ⁽²⁾	1.1 m	10 mm	About 50% occurs within 50 years.
Earth Fill Embankment	4.0 m	15 mm	Occurs rapidly (during construction).
Rock Fill Embankment	4.0 m	15 mm	About 60% occurs within the first year.
Total Estimated Settlement		80 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			35 mm to 60 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

5.4.5.3 Alternatives for Mitigation of Settlement/Stability

The presence of the approximately 1.1 m thick soft clay layer influences both the stability and magnitude of post-construction settlement of the proposed 4.0 m high widened embankment in this area as discussed above. In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the alternatives as discussed below should be considered. The alternatives described below have been evaluated on the basis of advantages, disadvantages, relative costs and risks/consequences and are summarized in Table 7.

5.4.5.3.1 Partial Sub-excavation

In order to minimize the post-construction settlement of the embankment and differential settlement, consideration should be given to sub-excavation of the surficial organics and soft clay within the limits of the embankment widening in this swamp area. The excavation would extend to a maximum depth of about 3.5 m (below the water surface) based on the boreholes encountered within this swamp area. The excavation will be carried out below the water level to about Elevation 189.5 m south of the middle rock outcrop and to about Elevation 190.3 m north of the middle rock outcrop. The other benefit to sub-excavation of the soft clay is that, as a result, the near surface very loose sands will also be removed thereby removing the potential for liquefaction concerns.

Sub-excavation of the clay adjacent to the existing embankment will be subject to restrictions placed on the construction procedures as outlined in Section 5.6.1. Temporary Roadway Protection will not be required for this alternative.

Sub-excavation of the organics and soft clay will reduce the post-construction settlement. In addition, settlement of the replacement rock fill settlement will also occur (see discussion in Section 5.2.3.2). These estimated settlements assume that all the clay has been subexcavated below the existing embankment. The total estimated settlement of the foundation soils and rock fill for the partial sub-excavation alternative are given below:

<i>Contribution to Total Settlement from:</i>	<i>Maximum Total Thickness⁽³⁾</i>	<i>Maximum Estimated Settlement</i>	<i>Time Dependency</i>
Granular Soils	1.5 m	10 mm	Occurs rapidly (during construction).
Cohesive Soils ⁽¹⁾	0 m	0 mm	n/a
Long-term Creep ⁽²⁾	0 m	0 mm	n/a
Earth Fill Embankment (EF)	4.0 m	15 mm	Occurs rapidly (during construction).
Rock Fill Embankment (RF)	4.0 m	15 mm	About 60% occurs within the first year.
Rock Fill Backfill	3.5 m	25 mm	About 60% occurs within the first year.
Total Estimated Settlement		50 mm	
ESTIMATED POST-CONSTRUCTION SETTLEMENT			< 25 mm

1. Primary consolidation of cohesive deposit.
2. Secondary (creep) consolidation settlement expected over two log cycles of time after the end of primary consolidation.
3. At critical section.

The estimated post construction settlement applies to the end construction, about 1 year following completion. The total estimated settlement expected to occur during construction could be up to 50 mm and will be differential relative to the existing highway.

5.4.5.3.2 Preloading

Another alternative to reduce the magnitude and rate of post-construction settlements is to preload the embankment in conjunction with adding a surcharge. Placing a surcharge load on the embankment will also assist with further reducing long-term creep settlement.

Preloading

Preloading the embankment could be used to reduce the magnitude of post-construction settlement under the embankment as an alternative to sub-excavation. About 95 percent of the primary consolidation settlement is expected to occur within about 3 months as shown on Figure 8. If the construction schedule can accommodate this time period, preloading the foundation soils by building the embankment as early as possible can be considered as an alternative to sub-excavation. The magnitude of post-construction settlement at the end of the preload period (including creep) is estimated to be less than 25 mm.

Surcharging

Placement of a surcharge load adjacent to the existing highway may be difficult and space may not be available. Since preloading for 3 months will reduce post-construction settlements to less than 25 mm, the addition of a surcharge load is not considered warranted.

5.4.5.3.3 Lightweight Fill

If the differential settlements discussed above (between 50 mm and 80 mm) cannot be tolerated by the widened embankment, then consideration could be given to the use of lightweight fill. Lightweight aggregate such as slag would reduce differential settlements by about 60%, i.e. to between 30 mm and 50 mm; lightweight EPS fill would reduce settlements to less than 25 mm. However, the cost of supplying the lightweight fill material is typically on an order of magnitude higher than other options.

5.4.5.3.4 Wick Drains

Another alternative to reduce the magnitude and rate of post-construction settlements as well as to address stability concerns is to preload the embankment in conjunction with adding a surcharge and installing wick drains. To reduce the time required for the settlement to occur, wick drains could be installed beneath the embankment. However, due to relatively thin clay deposit (less than 1.1 m) and depth to the base of clay deposit (less than 3.5 m), wick drains are considered impractical at this site as they will not significantly reduce the drainage path.

5.5 Subgrade Preparation and Embankment Construction

As discussed in Section 5.1, the construction of the new service roads will require the construction of up to 6.2 m high embankments over swamps. Prior to the placement of any fill for the new embankment construction, all surface and near surface layers of topsoil and organic deposits should be stripped from the plan limits of the proposed works. The depth of organics is given for each individual swamp/high fill area in Section 5.4 and is typically between 0.5 m and 2 m. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

It should be noted that since all the embankments are less than 8 m in height, there is no requirement for mid-height benches.

5.5.1 Removal of Organics/Sub-excavation

In areas where new fill embankments will be constructed immediately adjacent to, or on top of, the existing Highway 400 embankments (i.e. Muskoka Road 33 S-EW Ramp adjacent to Bear Lake), construction procedures should implement the guidelines of OPSD 203.020. These guidelines require that the slopes of the existing embankment be temporarily excavated to a 1H:1V profile to allow for removal of a larger extent of organic material. In this area, the depth of sub-excavation should encompass not only the peat/organics but the soft clay as well, to an approximate depth of up to 3.5 m below the water surface. Further detail regarding the staged excavation in this area is provided below and will be incorporated into a operational constraint to the contract (a sample is included in Appendix F).

- removal of the organics/clay for the entire section of the widening should be carried out in short sections perpendicular to the highway alignment with the base of the excavation/trench not wider than 3 m at any time;
- the crest of the excavation should start no closer than 2 m from the existing edge of pavement;
- temporary excavation side slopes through the organics/clay should be no steeper than 3H:1V;
- excavation and backfilling operations should be carried out simultaneously in a manner that the excavation is not left open for more than 3 m in length at any given time; and
- maintain operation of the highway during excavation and backfilling operations including and not limited to traffic control, regrading and asphalt padding.

Since some distress to the existing roadway may occur during the staged excavation, provision for traffic control measures must be included in the Contract to maintain the safe operation of Highway 400 during the excavation and backfilling operations.

It should be noted that the rock is sloping in the sub-excavation both away from the embankment (towards Bear Lake) as well as away from the rock outcrops at the edges of the lake and the centre of this roadway section. Therefore, the exact excavation profile shown on OPSD203.02 may not be achievable and the quantities may vary slightly.

5.5.2 Platform widening

For the gravel and surface treated roads in the swamp/high fill embankment areas (Heather Path Trail, Global Tower Road and Hidden Glen Road) platform widening is not required.

For highway embankment construction, typically a 2 m platform widening is required for future grade raises. If settlements during construction are anticipated to be large, additional platform width could be required. Since the organics and soft clay will be sub-excavated in the area of the

S-E/W ramp (Bear Lake), the post-construction settlement is expected to be less than 25 mm and therefore no additional platform width is required for this purpose.

5.5.3 Slope Flattening

It is understood that consideration is being given to using sub-excavated material, if any, for flattening of the proposed embankment fill slopes. In addition, as per the Northeastern Region Guidelines, slope flattening is typically required for all embankments under 4 m in height. However, depending on the type of material used, and the timing of placement of the additional material, slope flattening may adversely affect the long-term performance of the roadway. Considerations with respect to settlement and stability are discussed below. It is assumed that the rock fill side slopes will be made at 1.25H:1V, earth fill side slopes at 2H:1V and that the flattened side slopes will be at 4H:1V. It is also assumed that the material used for the slope flattening will consist of earth material, rather than rock fill.

5.5.3.1 Stability

In general, global stability is enhanced when side slopes are flattened (i.e. where slope flattening was applied). A factor of safety against deep-seated failure of 1.3 is considered appropriate. Where toe berms are required for stability (i.e. Global Tower Road), they should still be constructed such that the overall slope with the slope flattening is at least equal to or flatter than the case with just toe berms. In these areas, the slope flattening should consist of granular earth fill.

5.5.3.2 Settlement

If sub-excavation of the subsoils is considered as the preferred settlement or stability mitigation alternative, then the timing of placement of slope flattening material is not a concern. However, if preloading is chosen as the preferred mitigation alternative, then there will be concerns with respect to the timing of placement of slope flattening material, as settlement of the embankment will occur as a result of placement of the slope flattening material. Three scenarios are presented below for placement of the additional slope flattening material as well as the settlement implications.

1. Slope flattening is carried out simultaneously with construction of the embankment (in stages where required): It is assumed that construction of this nature would be difficult; however, this method of embankment construction would produce the least amount of post-construction settlement of the roadway. This construction method is recommended where the sub-excavation alternative is used. It should be noted that some settlement of the slope flattening areas should be anticipated since it is assumed that organics/soft clay would not be removed below the flattening areas (i.e. beyond the toe of the final embankment slope).

2. Slope flattening is carried out after construction of the preload embankment and prior to placement of the final surcharge and/or prior to the full preload period: This construction method is recommended since the settlement of the embankment caused by the addition of the slope flattening material would take place during the preload period, and any adjustments to the road grade could be made prior to final paving.
- Slope flattening is carried out after the preload period is complete: This construction method is not recommended, since the additional load from the slope flattening material will cause immediate and long-term settlement beneath both the embankment side slopes and the roadway. The magnitude of the settlement could be significant, depending on the subsoil conditions in the area.

5.5.4 Embankment Fill Placement

Where existing embankments are composed of sand and gravel or rock fill (i.e. Muskoka Road 33 S-EW Ramp), benching into the existing side slopes should be carried out as per OPSD 208.010 as part of the embankment widening construction.

Placement of all fill (rock or earth) material should be carried out in accordance with the requirements as outlined in the Special Provision SP206S03, dated July 2006. Rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass.

Final rock fill side slopes should be no steeper than 1.25H:1V. Final earth fill side slopes should be no steeper than 2H:1V.

5.5.5 Backfilling

Where sub-excavation of soft clay is being carried out as a construction alternative, rock fill should be used as backfill below the water table. Provisions for placing rock fill under water are given in SP206S03. It is expected that the rock fill will be end dumped below the water table as the excavation advances. Recommendations for temporary cut side slopes are given for each swamp crossing in Section 5.4.

5.6 Excavations and Groundwater Control

As noted in Section 5.4, excavation within the plan limits of the proposed works will be required in order to remove organic deposits and soft compressible clay deposits prior to embankment fill placement. Groundwater flow into the excavations can be expected to occur due to the high groundwater levels observed at the sites, the relatively permeable subsoils as well as high water levels during seasonal times and poor drainage.

The maximum depth of excavation at the various sites is anticipated to be less than about 5.5 m. Groundwater inflow will occur and it is anticipated that the excavations will fill with water. Conventional excavation equipment should be suitable for the majority of excavation work through the on-site soils.

Where cutting is required through the rock outcrops, rock excavation and blasting will be required to accommodate the road widening and new alignment. All rock excavation and grading should be carried out in accordance with the requirements as outlined in Special Provision SP206S03, dated July 2006.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects as well as good construction practice. The excavated soils at this site are considered to be Type 4 soils according to OHSA.

5.7 Rock Cut – Muskoka Road 33 Structure (East Abutment)

Recommendations for rock excavation of the proposed Muskoka Road 33 EW-N Ramp are provided below. The location of the existing and proposed ramps are shown on Drawing E-1 in Appendix E.

Following completion of the east abutment rock cut investigation and review of the structural mapping, the feasibility of the proposed rock cut excavation (based on a cross-sections provided by URS) was assessed. While the existing cut is comprised of a strong to very strong rock mass, there is potential for three-planed wedges to form and daylight during excavation of the new EW-N Ramp. This has already been observed to have occurred south of the overpass on the same rock cut (see Figures E-2 and E-3).

The proposed ramp realignment indicates that there is a limited amount of space under the overpass, where rock cuts are required. Should over-break occur during rock excavation adjacent to the abutment, the extent of the wedge failure break-back would encroach within the typically recommended 0.5H:1V imaginary set-back line drawn extending back from the toe of the rock cut to the base of the existing bridge foundation. However, given the high quality of the bedrock, the potential encroachment is considered acceptable at this site provided that rock dowels are installed prior to excavation along the crest of the cut (to pre-support the rock face and control over-break). The dowels may be used in conjunction with careful and controlled blasting techniques (to minimize over-break) or alternatively, consideration could be given to the use of careful non-explosive excavation techniques.

Due to proximity of the proposed rock excavation to the abutment, non-explosive rock excavation techniques are recommended in order to minimize potential structural damage to the existing bridge foundation as well as minimize potential for over-break. One option would be line drilling (to define the cut-face) and rock removal by hoe-ramming. A second option could be the use of line drilling (to define the cut-face) and chemical expansive agents to break the rock. Should this chemical expansive agents alternative be considered, care would be required to ensure that the expansion-induced rock breakage does not propagate past the line drilling into the pre-supported (doweled) rock abutment through existing joints; this can be accomplished through the use of plastic socks in the drillholes prior to placement of the expansion agents. A sample NSSP for the installation of rock dowels is included in Appendix F for reference. A sample NSSP for rock excavation using non-explosive methods at the east abutment is given in Appendix F.

Rock blasting can be used for the remainder of the rock cut outside of the east abutment area. The blasting should be in accordance with the standard special provision and the most up-to-date version of the associated operational constraint (being used in current MTO construction specification packages). The blasting should also follow the Guidelines for Safe Blasting in Ontario Highway Construction Operations, ORBA October 2001.

The plan limits where explosive and non-explosive blasting are recommended are shown on Drawing 1.


Inspection of all new rock cut faces by qualified geotechnical personnel immediately after blasting should be carried out in order to assess where scaling/loosened rock removal should be carried out or where potential rock wedges should be supported by additional rock dowels.

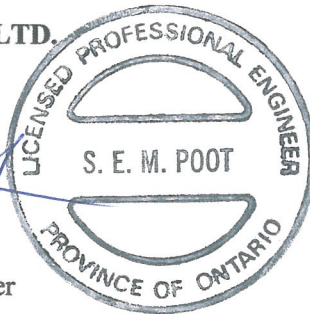
Additional words should be added to the existing (standard) blasting operational constraint for vibration monitoring close to the existing bridge. A sample of the additional monitoring is included in Appendix F. We recommend limiting ground vibration levels (peak particle velocity) to 50 mm/s for the abutment footing. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the bridge abutment.


5.8 Closure


This report was prepared by Miss Sarah Poot, P.Eng., a senior geotechnical engineer and the technical aspects were reviewed by Ms. Anne Poschmann, P.Eng., a Principal with Golder Associates Ltd. Mr. Jorge Costa, P.Eng., a Principal and Designated MTO Contact for Golder, conducted a quality control review of the report.

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SEP/ASP/JMAC/lb/lb

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TABLE 1
SUMMARY OF RECOMMENDED FOUNDATION TREATMENTS
HIGHWAY 400 SESERVICE ROADS, G.W.P 5622-02-00

<i>Station</i>	<i>Proposed Maximum Embankment Height</i>	<i>Recommended Foundation Treatment</i>	<i>Estimated Post-Construction Settlement ⁽¹⁾</i>
Heather Path Trail 9+510 to 9+650	6.2 m	Preloading with 300 mm surcharge for 3 months (no monitoring)	< 50 mm
Global Tower Road 10+065 to 10+175	5.0 m	Sub-excavation of overburden (soft clay) to approximately 5 m depth required for stability.	< 25 mm
Hidden Glen Road 11+100 to 11+223	2.0 m	No special mitigation requirements.	< 25 mm
Muskoka Road 33 S-EW Ramp (Bear Lake) 13+340 to 13+465	4.0 m	Sub-excavation of organics/soft clay: <ul style="list-style-type: none"> • to Elevation 189.5 m south of Stn 13+400 • to Elevation 190.3 m north of Stn 13+410. 	< 25 mm (differential with respect to the existing embankment)

Notes: 1. To be read in conjunction with Section 5.4 of the Foundation Design Report.

TABLE 2
SUMMARY OF ENGINEERING PARAMETERS
HEATHER PATH TRAIL STA. 9+510 TO 9+650
G.W.P. 5622-02-00

<i>Deposit</i>	<i>Depth (m)</i>	<i>Elevation (m)</i>	<i>Groundwater Elevation (m)</i>	γ (<i>kN/m³</i>)	ϕ' (<i>°</i>)	s_u (<i>kPa</i>)	c_v (<i>cm²/s</i>)	C_c	C_r	e_o	$C_{\alpha\varepsilon}$
Firm to Stiff Clay	0 – 1.7	193.7 – 192.0	193.7	17	26	25	2.3×10^{-3}	0.425	0.089	0.986	3.6×10^{-3}
Compact to Very Dense Sand and Gravel	1.7 – 2.8	192.0 – 190.9		20	30	n/a	n/a	n/a	n/a	n/a	n/a

TABLE 3
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
HEATHER PATH TRAIL STA. 9+510 TO 9+650
HIGHWAY 400, G.W.P 5622-02-00

<i>Stability/ Settlement Mitigation Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Total Post-Construction Settlement ⁽¹⁾</i>	<i>Risks/Consequences</i>
Pre-loading/ Surcharging <ul style="list-style-type: none"> • Base of soft clay up to 3.1 m depth. • Embankment up to 6.2 m in height. • 300 mm surcharge 	<ul style="list-style-type: none"> • Eliminates sub-excavation below the water table. • Reduces long term settlement. • Monitoring program not required. • Toe berms not required. 	<ul style="list-style-type: none"> • Minimum preload period of 3 months required with small surcharge load. 	Less expensive than sub-excavation. Less surcharge material required.	Less than 50 mm.	<ul style="list-style-type: none"> • Settlement of clay will occur during preload period. • Some post-construction settlement will occur after the preload period.
Pre-loading/ Surcharging/Staged Excavation <ul style="list-style-type: none"> • Base of soft clay up to 3.1 m depth. • Embankment up to 6.2 m in height. • 2 m surcharge. 	<ul style="list-style-type: none"> • Eliminates sub-excavation below the water table • Compression of rock fill minimized. • Reduces long term settlements. • Reduce preload time with surcharge. 	<ul style="list-style-type: none"> • Minimum preload period of 6 months required. • Monitoring program for settlements and pore-pressures required to confirm duration of settlement. • Preload period reduced to 3 months with surcharging depending on results of monitoring program. • Toe berms required for stability. • Staged construction may be used instead of toe berms but preload period increased to 4 months. 	Less expensive than sub-excavation.	Less than 25 mm.	<ul style="list-style-type: none"> • Settlement of clay will occur during preload period. • Construction duration dependant on the results of the monitoring program.
Sub-excavation of weak, soft and compressible material <ul style="list-style-type: none"> • Base of soft clay up to 3.1 m depth. • Embankment up to 6.2 m in height. 	<ul style="list-style-type: none"> • Long-term settlement of clay minimized. • Stability not a concern. • Limited construction period. 	<ul style="list-style-type: none"> • Long-term settlement of rock fill increased. • High groundwater table. • Cut slopes at 3H:1V required. • Disposal of excavated soil; has to be stockpiled and dried for re-use as slope flattening. 	Cost of sub-excavation and backfill material.	Less than 25 mm.	<ul style="list-style-type: none"> • Rock fill settlement.
Wick Drains (with preloading and surcharging)	<ul style="list-style-type: none"> • Reduction in preload time. 	<ul style="list-style-type: none"> • Not practical at this site due to limited thickness of clay. 	n/a	n/a	n/a

Notes: 1. Settlement estimates should be read in conjunction with Section 5.4 of this report.

TABLE 4
SUMMARY OF ENGINEERING PARAMETERS
GLOBAL TOWER ROAD STA. 10+065 TO 10+175
G.W.P. 5622-02-00

<i>Deposit</i>	<i>Depth (m)</i>	<i>Elevation (m)</i>	<i>Groundwater Elevation (m)</i>	γ (<i>kN/m³</i>)	ϕ' (<i>°</i>)	s_u (<i>kPa</i>)	c_v (<i>cm²/s</i>)	C_c	C_r	e_o	$C_{\alpha\varepsilon}$
Peat/Organics	0.3 – 1.1	199.0 – 198.2	199.3	13	26	20	n/a	n/a	n/a	n/a	n/a
Very Soft to Firm Silty Clay to Clay	1.1 – 4.7	198.2 – 194.6		17	26	6 - 30	1.1×10^{-2}	0.494	0.085	1.526	6.4×10^{-3}
Compact Sand	4.7 – 0.3	194.6 – 194.3		20	30	n/a	n/a	n/a	n/a	n/a	n/a

TABLE 5
SUMMARY OF ENGINEERING PARAMETERS
HIDDEN GLEN ROAD STATION 11+100 TO 11+223
G.W.P. 5622-02-00

<i>Deposit</i>	<i>Depth (m)</i>	<i>Elevation (m)</i>	<i>Groundwater Elevation (m)</i>	γ (<i>kN/m³</i>)	ϕ' (<i>°</i>)	s_u (<i>kPa</i>)	c_v (<i>cm²/s</i>)	C_c	C_r	e_o	$C_{\alpha\varepsilon}$
Peat	0 – 1.8	203.2 – 201.4	203.2	13	23	20	n/a	n/a	n/a	n/a	n/a
Soft Clay	1.8 – 2.7	201.4 – 200.5		17	26	20	3.2×10^{-3}	0.440	0.070	1.553	5.4×10^{-3}
Very Dense Gravelly Sand	2.7 – 3.0	200.5 – 200.2		20	30	n/a	n/a	n/a	n/a	n/a	n/a

TABLE 6
SUMMARY OF ENGINEERING PARAMETERS
MUSKOKA ROAD 33 S-EW RAMP STA. 13+340 to 13+465 (BEAR LAKE)
G.W.P. 5622-02-00

<i>Deposit</i>	<i>Depth (m)</i>	<i>Elevation (m)</i>	<i>Groundwater Elevation (m)</i>	γ (<i>kN/m³</i>)	ϕ' (<i>°</i>)	s_u (<i>kPa</i>)	c_v (<i>cm²/s</i>)	C_c	C_r	e_o	$C_{\alpha\varepsilon}$
Peat/Organics	0.5 – 2.4	192.5 – 190.6	193.0	13	23	20	n/a	n/a	n/a	n/a	n/a
Loose Sand	2.4 – 3.1	190.6 – 189.9		20	29	n/a	n/a	n/a	n/a	n/a	n/a
Firm Clay	3.1 – 3.5	189.9 – 189.5		17	26	25	3.5×10^{-3}	0.466	0.093	1.148	7.6×10^{-3}
Compact Gravelly Sand	3.5 – 5.0	189.5 – 188.0		21	33	n/a	n/a	n/a	n/a	n/a	n/a

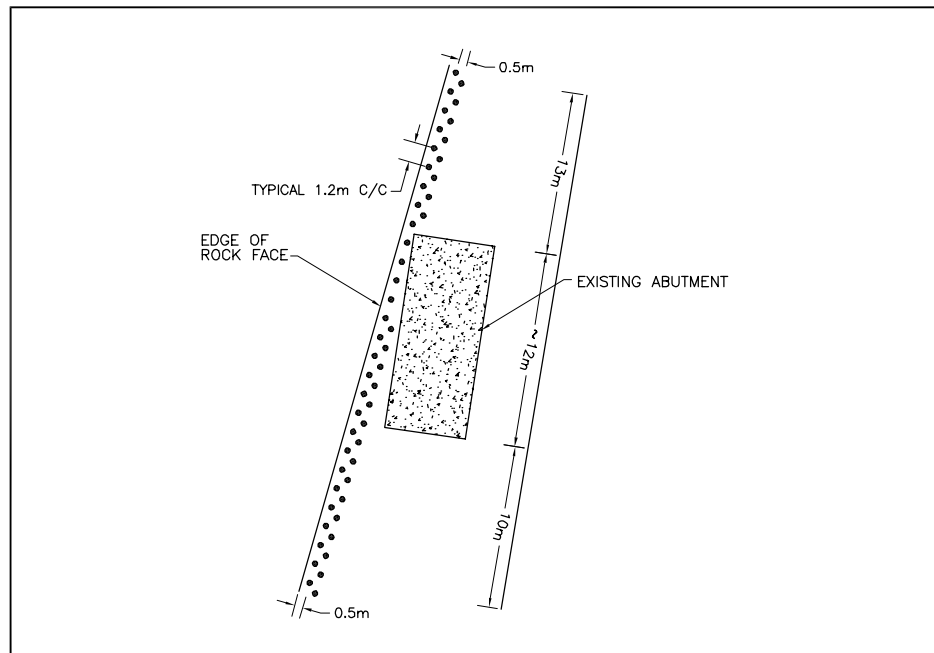
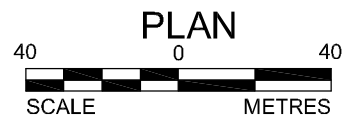
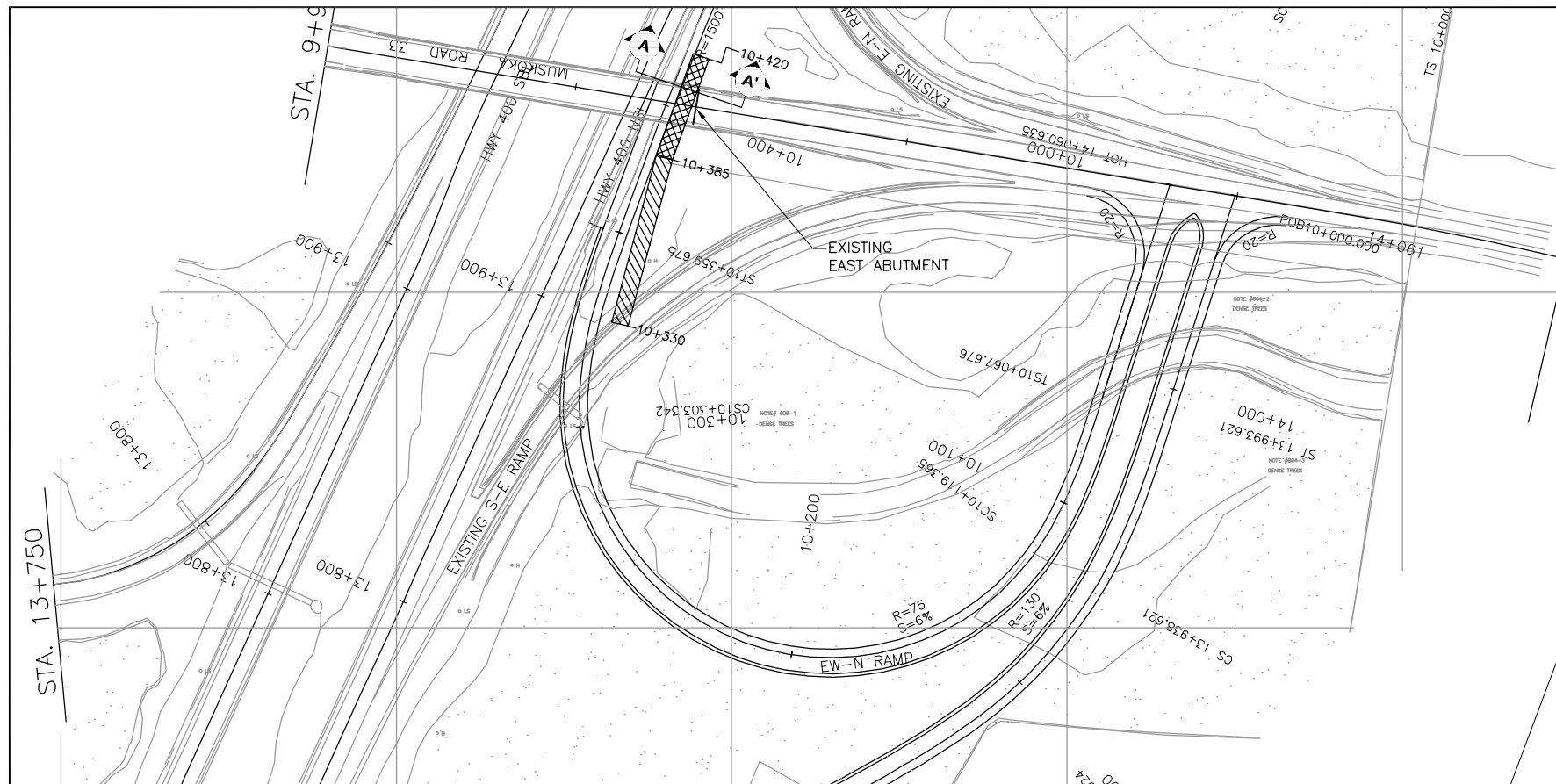
TABLE 7
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
MUSKOKA ROAD 33 S-EW RAMP STA. 13+340 TO 13+465 (BEAR LAKE)
HIGHWAY 400, G.W.P 5622-02-00

<i>Stability/ Settlement Mitigation Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Estimated Costs</i>	<i>Total Post- Construction Settlement (See Note 1)</i>	<i>Risks/Consequences</i>
Sub-excavation of weak, soft and compressible material <ul style="list-style-type: none"> • Base of soft clay up to 3.5 m depth. • Embankment up to 4.0 m in height. 	<ul style="list-style-type: none"> • Long-term settlement of clay minimized. • Stability not a concern. • Limited construction period. • Liquefaction potential eliminated as very loose sands also removed. 	<ul style="list-style-type: none"> • Long-term settlement of rock fill increased. • High groundwater table and difficult excavation adjacent to existing highway. • Cut slopes at 3H:1V required. • Disposal of excavated soil; has to be stockpiled and dried for re-use as slope flattening. 	Cost of sub-excavation and replacement with rock fill.	Less than 25 mm.	<ul style="list-style-type: none"> • Rock fill settlement. • Differential settlement.
Preloading <ul style="list-style-type: none"> • Base of soft clay up to 3.5 m depth. • Embankment up to 4.0 m in height. 	<ul style="list-style-type: none"> • Eliminates sub-excavation below the water table • Compression of rock fill minimized. • Reduce long term settlements. 	<ul style="list-style-type: none"> • Minimum preload period of 3 months required. • Monitoring program for settlements and pore-pressures .required to confirm magnitude and duration of settlement. • Liquefaction potential not addressed. 	Less expensive than sub-excavation.	Less than 25 mm.	<ul style="list-style-type: none"> • Differential settlement. • Settlement of soft clay will occur during preload period. • Construction duration dependant on the results of the monitoring program.
Lightweight Fill	<ul style="list-style-type: none"> • Reduces differential settlement. 	<ul style="list-style-type: none"> • Significant cost. • Issues regarding placement of LWF adjacent to/into Bear Lake. 	Typically an order of magnitude higher.	Depends on type of lightweight fill.	n/a
Wick Drains (with preloading and surcharging)	<ul style="list-style-type: none"> • Reduction in preload time. 	<ul style="list-style-type: none"> • Not practical at this site due to limited thickness of clay. 	n/a	n/a	n/a

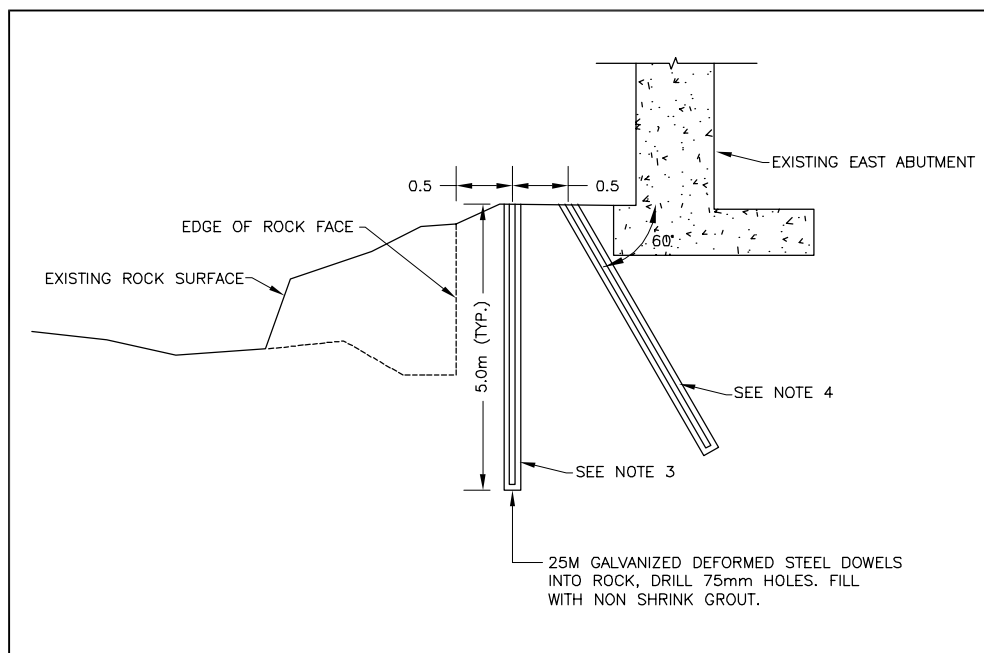
Notes: 1. Settlement estimates should be read in conjunction with Section 5.4 of this report.

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NTS



NTS

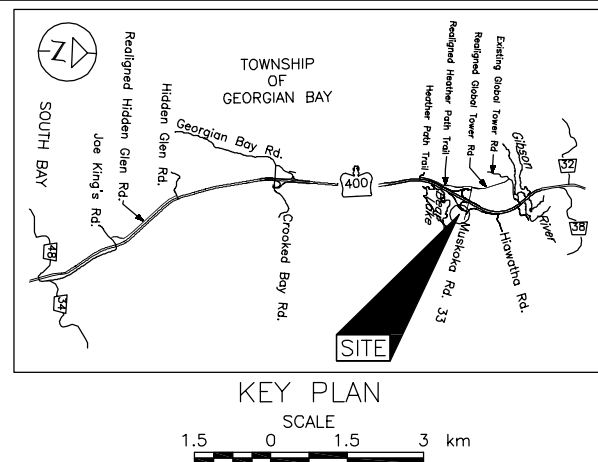
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5622-02-00




MUSKOKA ROAD 33
EAST ABUTMENT ROCK EXCAVATION
AND REINFORCEMENT



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND:

-  ROCK DOWEL
 AREA REQUIRING ROCK DOWEL REINFORCEMENT AND NON-EXPLOSIVE ROCK EXCAVATION
 AREA REQUIRING ROCK EXCAVATION BY CONTROLLED BLASTING

NOTES:

1. INSTALL FULLY CEMENT GROUTED, HOT DIP GALVANIZED DEFORMED ROCK DOWELS AT LOCATIONS SHOWN ON THE CONTRACT DRAWINGS AND AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
2. THE ROCK DOWELS ARE TO CONSIST OF 5.0M LONG 25MM DIAMETER (MINIMUM) DEFORMED REBAR GRADE (MINIMUM YIELD STRENGTH 400 MPA) STEEL.
3. FIRST ROW OF DOWELS PLACED 0.5M FROM EDGE OF ROCK-CUT, AND SPACED 1.2M C/C ACROSS THE AREA REQUIRING REINFORCEMENT.
4. SECOND ROW OF DOWELS PLACED 1.0M FROM EDGE OF CUT (WHERE SPACE ALLOWS), STAGGERED 0.6M FROM FIRST ROW AND SPACED 1.2 C/C ACROSS THE AREA REQUIRING REINFORCEMENT.
5. DOWEL LOCATION TO BE FIELD VERIFIED AND ADJUSTED BY THE QVE ONCE THE OUTCROP IS CLEANED AND INSPECTED.
6. 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.

6. 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.

3. FIRST ROW OF DOWELS PLACED 0.5M FROM EDGE OF ROCK-CUT, AND SPACED 1.2M C/C ACROSS THE AREA REQUIRING REINFORCEMENT.

4. SECOND ROW OF DOWELS PLACED 1.0M FROM EDGE OF CUT (WHERE SPACE ALLOWS), STAGGERED 0.6M FROM FIRST ROW AND SPACED 1.2 C/C ACROSS THE AREA REQUIRING REINFORCEMENT.

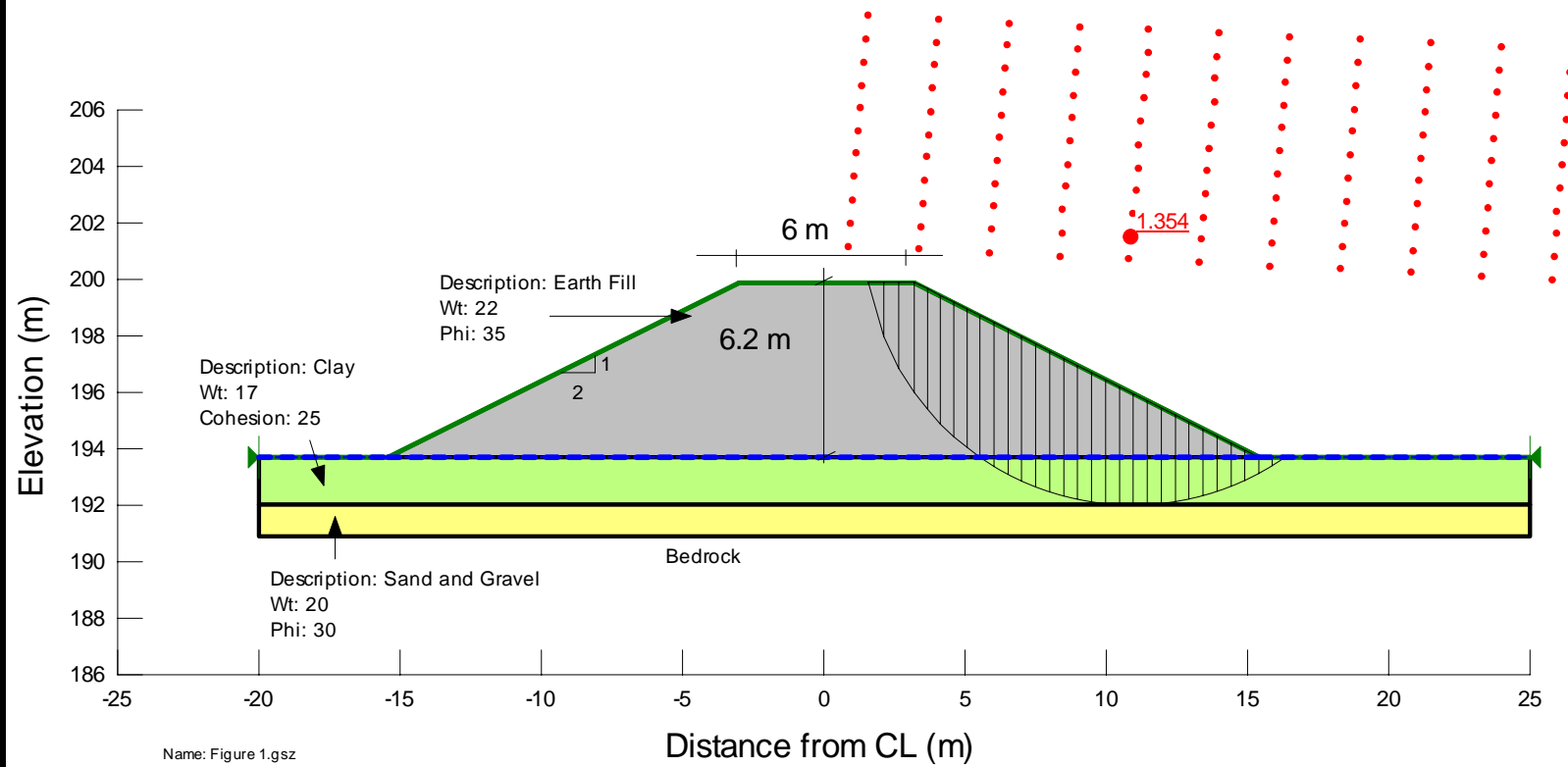
5. DOWEL LOCATION TO BE FIELD VERIFIED AND ADJUSTED BY THE QVE ONCE THE OUTCROP IS CLEANED AND INSPECTED.

6. 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.

NO.	DATE	BY	REVISION	
Geocres No.				
HWY. 400		PROJECT NO. 05-1191-029		DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006		SITE:
DRAWN: RN	CHKD. SEP	APPD. JMAC		DWG. 1

STABILITY ANALYSIS
Heather Path Trail (Earth Fill)

FIGURE 1



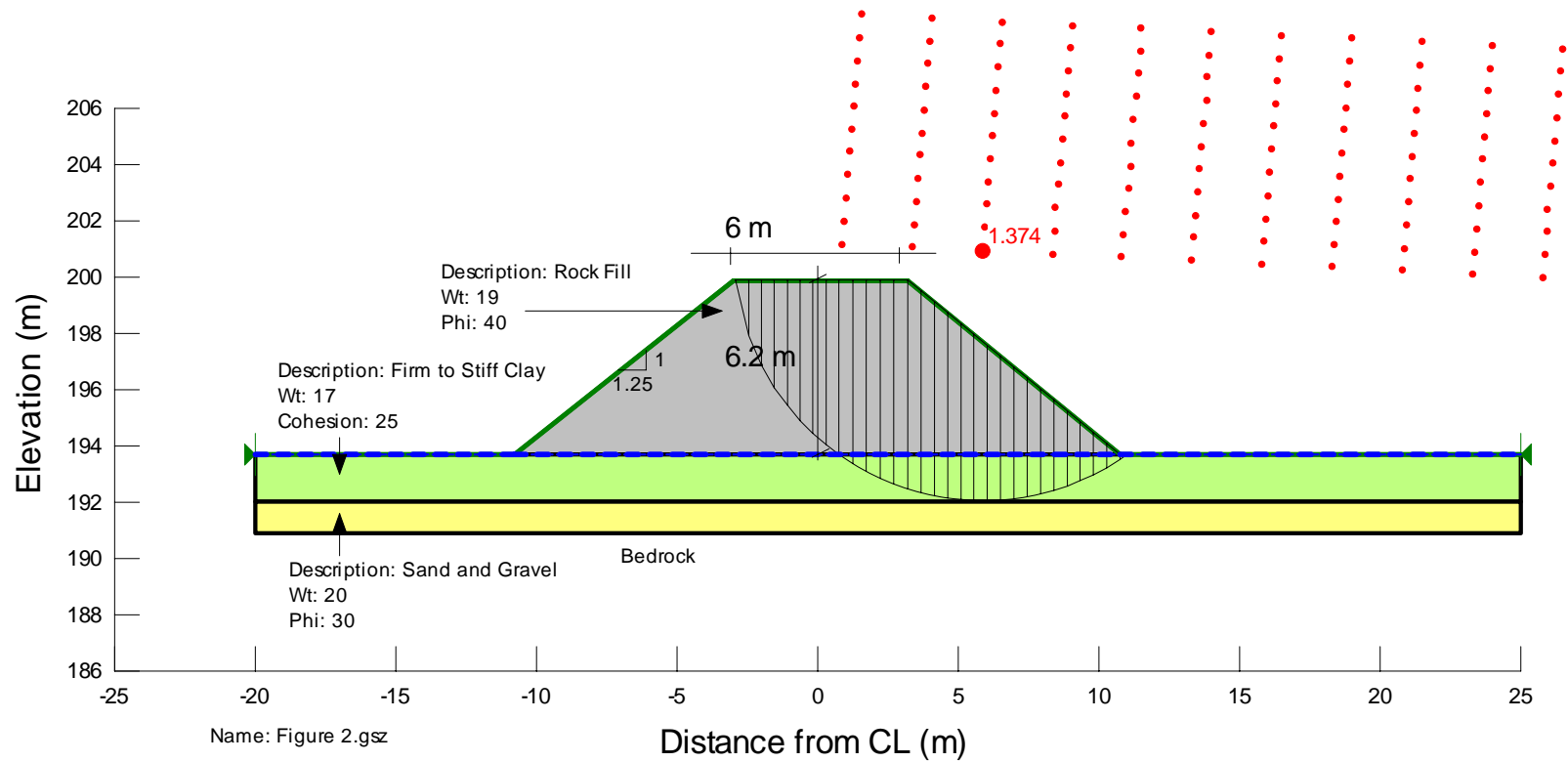
Date: October 2006
Project: 05-1191-029

DRAFT
Golder Associates

Drawn: AB
Checked: SEP

STABILITY ANALYSIS
Heather Path Trail (Rock Fill)

FIGURE 2



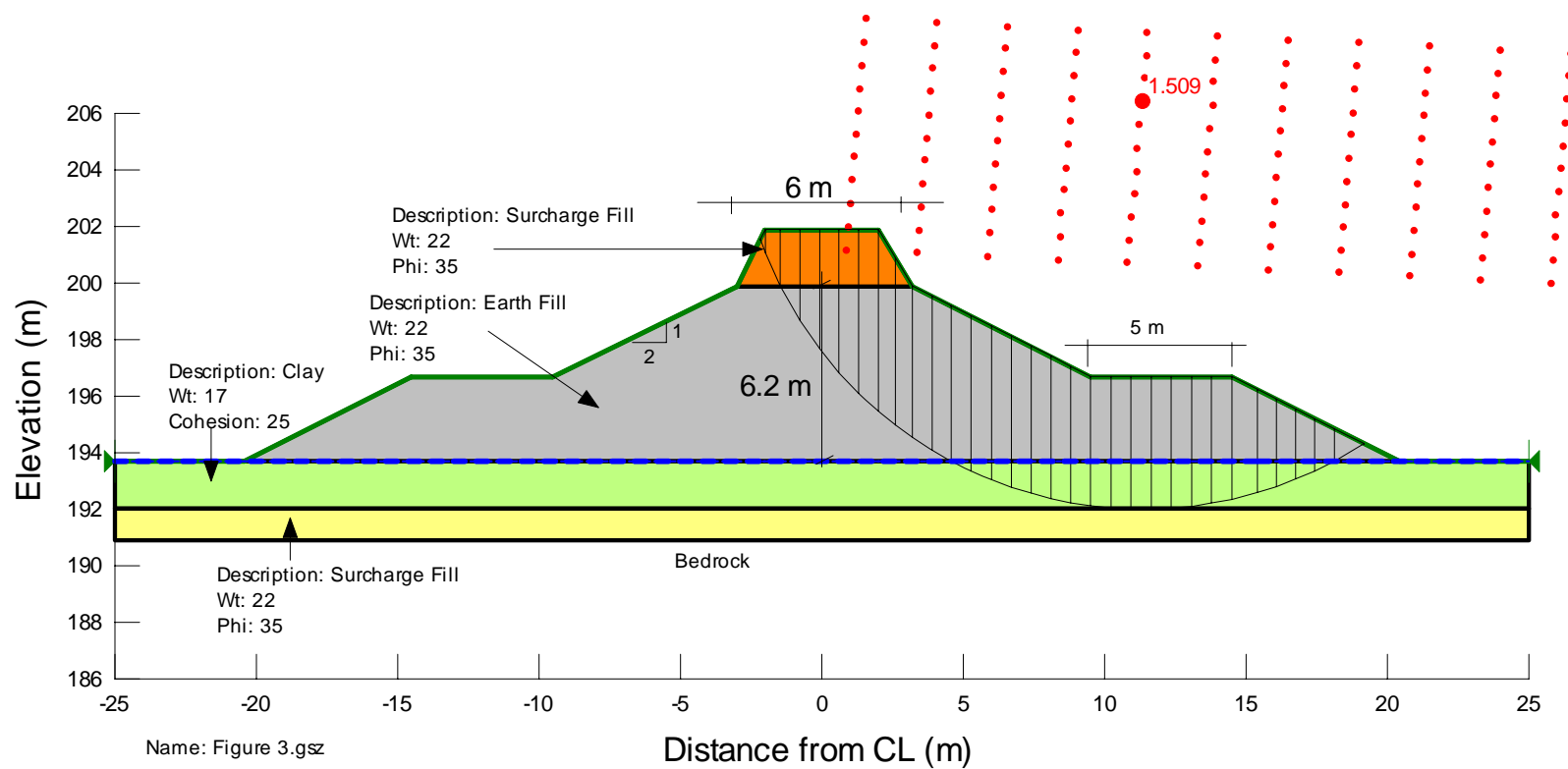
Date: October 2006
Project: 05-1191-029

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Checked: SEP

STABILITY ANALYSIS
Heather Path Trail (Surcharge Fill)

FIGURE 3



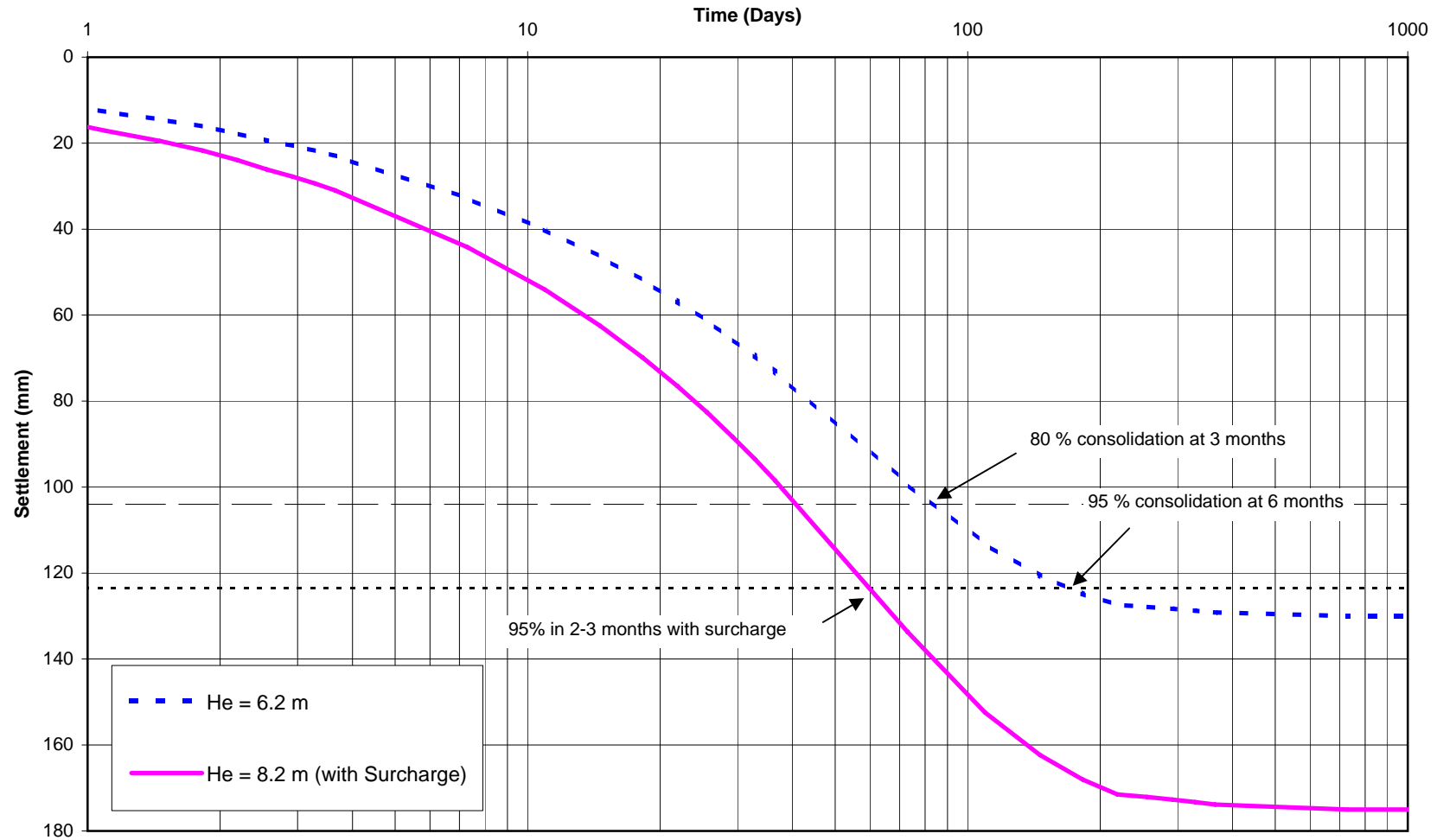
Date: October 2006
Project: 05-1191-029

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Drawn: AB
Checked: SEP

TIME RATE OF SETTLEMENT
Heather Path Trail

FIGURE 4

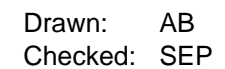


Date: October 2006
Project: 05-1191-029

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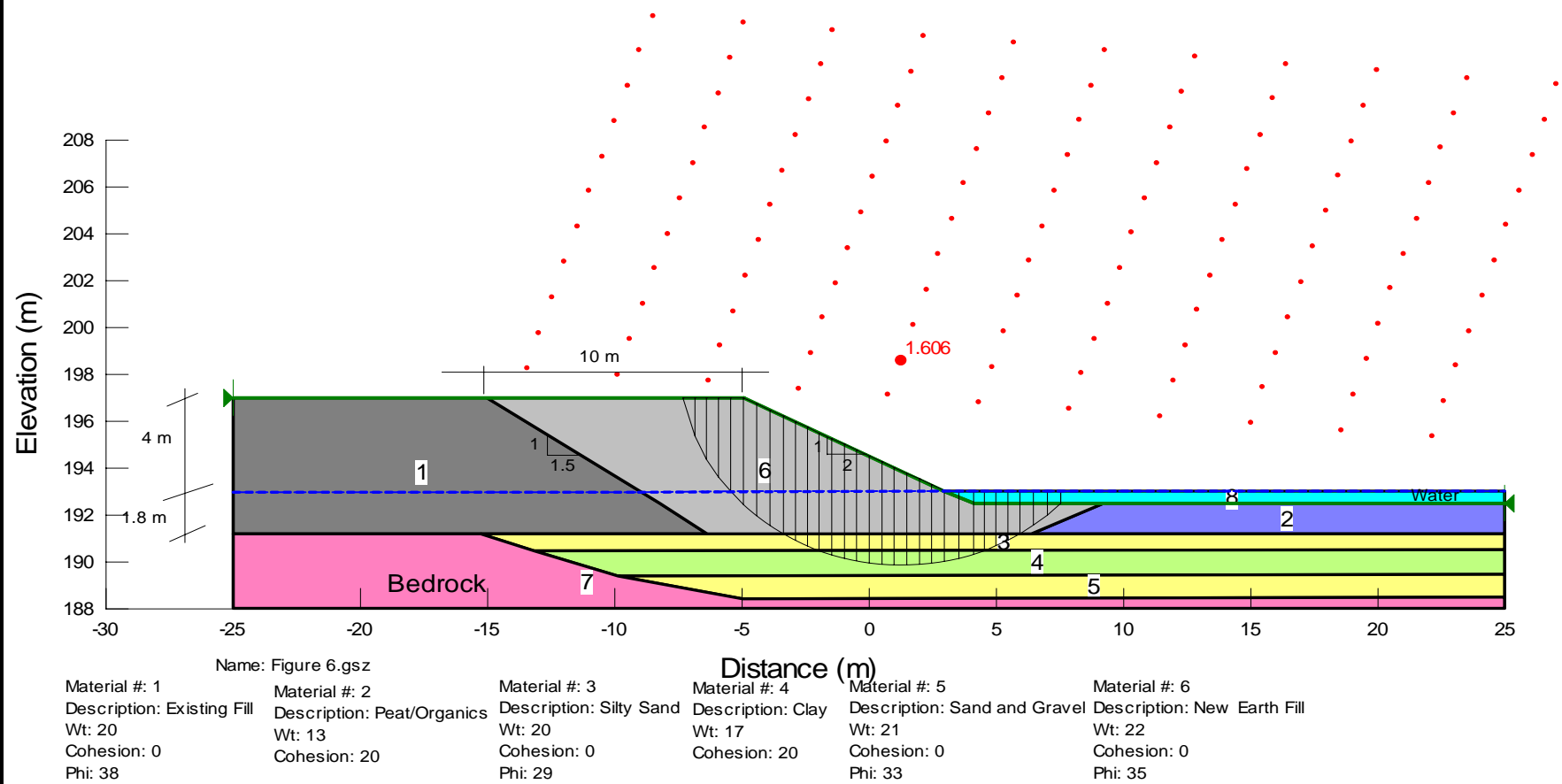
FIGURE 5



STABILITY ANALYSIS

Muskoka Road 33 S-EW Ramp (Earth Fill)

FIGURE 6



Date: October 2006
Project: 05-1191-029

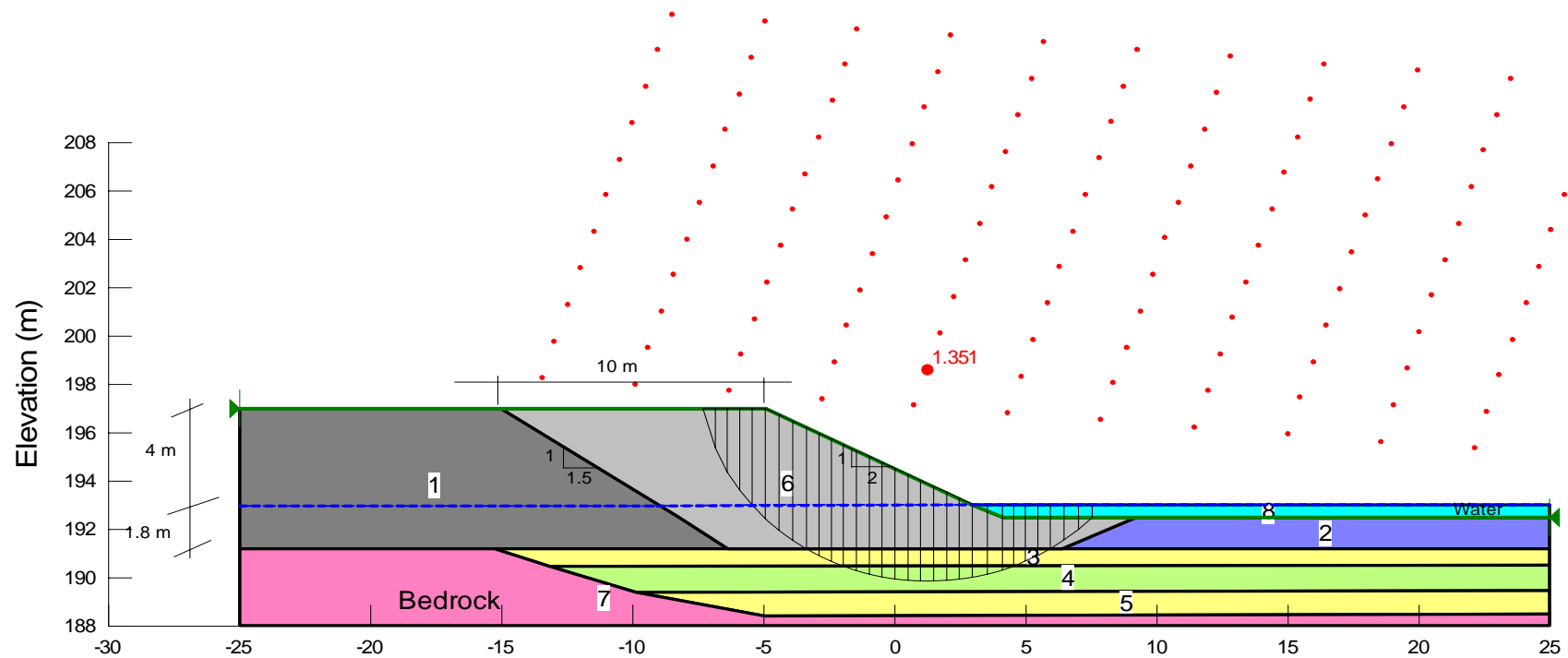
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STABILITY ANALYSIS

Muskoka Road 33 S-EW Ramp (Seismic Loading)

FIGURE 7



Name: Figure 7.gsz

Material #: 1	Material #: 2	Material #: 3	Material #: 4	Material #: 5	Material #: 6
Description: Existing Fill	Description: Peat/Organics	Description: Silty Sand	Description: Clay	Description: Sand and Gravel	Description: New Earth Fill
Wt: 20	Wt: 13	Wt: 20	Wt: 17	Wt: 21	Wt: 22
Cohesion: 0	Cohesion: 20	Cohesion: 0	Cohesion: 20	Cohesion: 0	Cohesion: 0
Phi: 38		Phi: 29		Phi: 33	Phi: 35

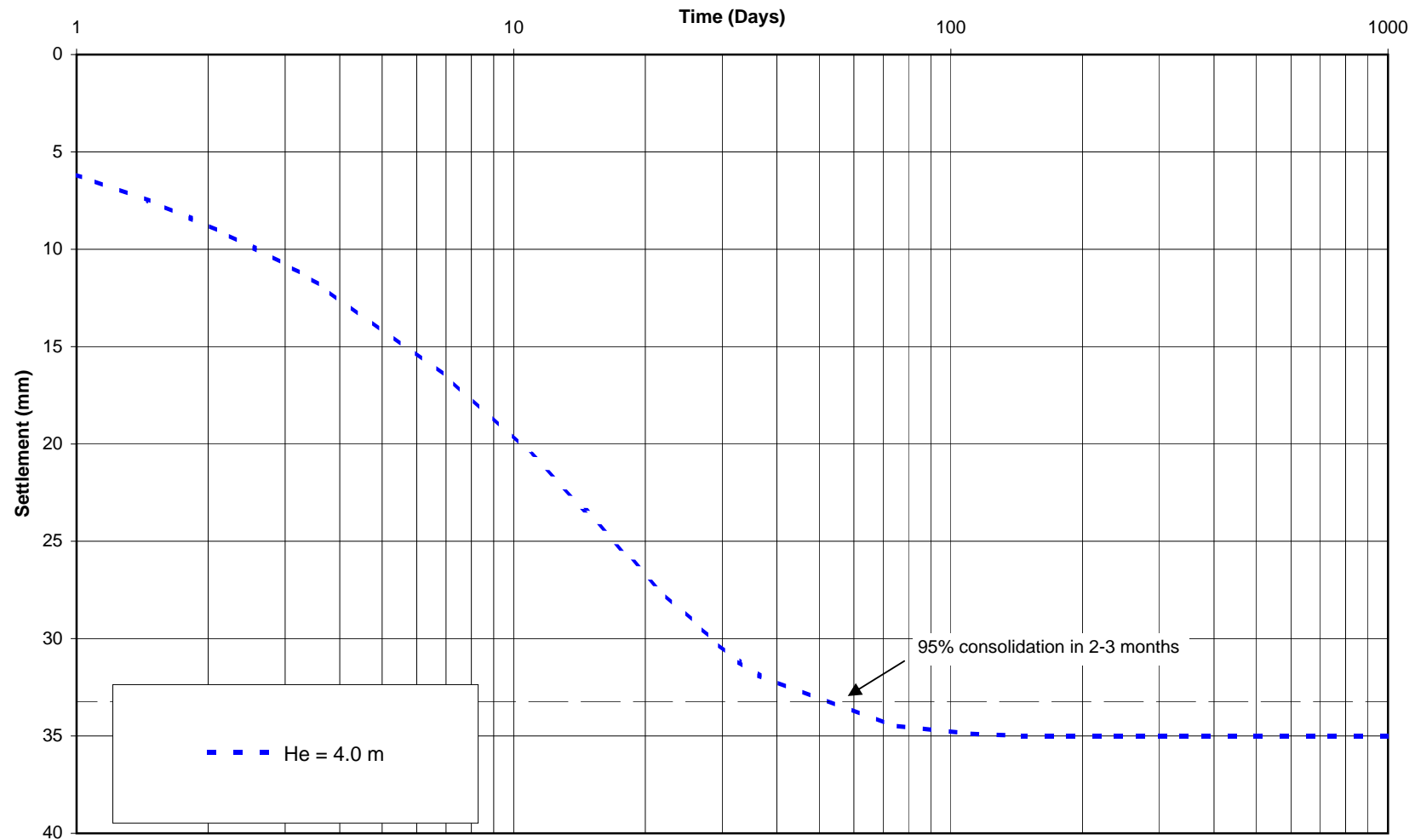
Date: October 2006
Project: 05-1191-029

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TIME RATE OF SETTLEMENT
Muskoka Road 33 S-EW Ramp (Earth Fill)

FIGURE 8



Date: October 2006
Project: 05-1191-029

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APPENDIX A
HEATHER PATH TRAIL

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

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

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.)

Consistency

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

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, 60° 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 cm² 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.

IV. SOIL TESTS

w water content
w_p plastic limit
w_i liquid limit
C consolidation (oedometer) test
CHEM chemical analysis (refer to text)
CID consolidated isotropically drained triaxial test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(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

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 Shear strength = (Compressive strength)/2



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W.P.		5622-02-00		LOCATION		N 4980554.0 ; E 282860.0												
DIST		HWY 400		BOREHOLE TYPE		Power Auger, 150 mm OD Solid Stem Augers												
DATUM		Geodetic		DATE		May 18, 2006												
						ORIGINATED BY ID												
						COMPILED BY AB												
						CHECKED BY SEP												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
194.5	0.0	GROUND SURFACE							20	40	60	80	100	20	40	60		
	0.6	Sand and Gravel (Fill) Brown Moist						194										
	193.9	Clay, containing silt seams, varved Firm to stiff Brown Wet		1	SS	6		193										
	0.6																	
	192.2			2	SS	12												
	2.3	Sand and Gravel Very dense Grey Wet						192										
	191.2			3	SS	32/0.23												
	3.4	End of Borehole Auger and Spoon Refusal		4	SS	53/0.15												
		Notes:																
		1. Spoon bouncing at 2.7 m depth (Elev. 191.8 m).																
		2. Water level in open borehole at 0.9 m depth (Elev. 193.6 m) upon completion of drilling operations.																

PROJECT		05-1191-029		RECORD OF BOREHOLE No G-2		1 OF 1		METRIC										
W.P.		5622-02-00		LOCATION		N 4980519.0 ; E 282859.0		ORIGINATED BY ID										
DIST		HWY 400		BOREHOLE TYPE		Power Auger, 150 mm OD Solid Stem Augers		COMPILED BY AB										
DATUM		Geodetic		DATE		May 18, 2006		CHECKED BY SEP										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
194.5	0.0	GROUND SURFACE																
0.0		Topsoil/Organics																
0.2		Sand and Gravel, trace silt Very dense Brown Moist		1	SS	67/0.30		194										
								193										
192.5				2	SS	109/0.45												55 38 (7)
2.0		End of Borehole Spoon and Auger Refusal																
		Notes:																
		1. Spoon bouncing at 1.1 m depth (Elev. 193.4 m).																
		2. Water level in open borehole at 0.8 m depth (Elev. 193.7 m) upon completion of drilling operations.																
		3. Auger sliding to the southeast at 2.0 m depth.																

PROJECT <u>05-1191-029</u>		RECORD OF BOREHOLE No G-3				1 OF 1 METRIC											
W.P. <u>5622-02-00</u>		LOCATION <u>N 4980494.0 ; E 282855.0</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Power Auger, 150 mm OD Solid Stem Augers</u>				COMPILED BY <u>AB</u>											
DATUM <u>Geodetic</u>		DATE <u>May 18, 2006</u>				CHECKED BY <u>SEP</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
193.8	GROUND SURFACE							20	40	60	80	100					
0.0	Topsoil/Organics Brown																
0.5	Clay Brown																
192.6	Sand and Gravel, silt seams Very dense Grey Wet		1	SS	65/0.30		193										
1.2	End of Borehole Auger Refusal																
Notes: 1. Spoon bouncing at 1.1 m depth (Elev. 192.7 m). 2. Water level in open borehole at 0.3 m depth (Elev. 193.5 m) upon completion of drilling operations. 3. Augers sliding to the east at 1.2 m depth.																	

PROJECT 05-1191-029			RECORD OF BOREHOLE No G-4			1 OF 1 METRIC											
W.P. 5622-02-00			LOCATION N 4980478.0 ; E 282858.0			ORIGINATED BY ID											
DIST HWY 400			BOREHOLE TYPE Power Auger, 150 mm OD Solid Stem Augers			COMPILED BY AB											
DATUM Geodetic			DATE May 18, 2006			CHECKED BY SEP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
193.8	GROUND SURFACE																
0.0	Topsoil/Organics Brown Wet																
193.3																	
0.5	Clay, containing silt seams, varved Stiff to firm Brown Wet		1	SS	10		193										
192.0																	
1.8	Sand and Gravel Loose to very dense Grey		2	SS	7		192										
			3	SS	60		191										
190.8																	
3.0	End of Borehole Auger Refusal																
	Notes: 1. Spoon bouncing at 2.7 m depth (Elev. 191.1 m). 2. Water level in open borehole at ground surface upon completion of drilling operations.																

PROJECT		05-1191-029		RECORD OF BOREHOLE No G-5		1 OF 1 METRIC												
W.P.		5622-02-00		LOCATION		N 4980470.0 ; E 282851.0												
DIST		HWY 400		BOREHOLE TYPE		Power Auger, 150 mm OD Solid Stem Augers												
DATUM		Geodetic		DATE		May 18, 2006												
						ORIGINATED BY ID												
						COMPILED BY AB												
						CHECKED BY SEP												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
194.0		GROUND SURFACE																
0.0	0.2	Topsoil/Organics Brown Wet																
193.4	0.6	Sand and Gravel Brown																
		Clay, containing silt seams, varved Hard Brown Wet		1	SS	32		193						○				
192.5																		
192.1		Sand and Gravel, some silt Very dense Grey Moist		2	SS	50/0.30								○				30 50 (20)
1.9		End of Borehole Auger Refusal																
		Notes: 1. Spoon bouncing at 1.8 m depth (Elev. 192.2 m). 2. Water level in open borehole at 0.3 m depth (Elev. 193.7 m) upon completion of drilling operations.																

PROJECT <u>05-1191-029</u>		RECORD OF BOREHOLE No G-6				1 OF 1 METRIC											
W.P. <u>5622-02-00</u>		LOCATION <u>N 4980419.0 ; E 282849.0</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Power Auger, 150 mm OD Solid Stem Augers</u>				COMPILED BY <u>AB</u>											
DATUM <u>Geodetic</u>		DATE <u>May 18, 2006</u>				CHECKED BY <u>SEP</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
198.0	GROUND SURFACE																
0.0	Topsoil/Organics Black		1	AS	-												
0.5	Silty Sand Reddish-brown Moist End of Borehole Auger Refusal Note: 1. Borehole dry upon completion of drilling. 2. Augers sliding at 0.5 m depth																



PROJECT		RECORD OF PENETRATION TEST				No DCPT G-1		1 OF 1		METRIC										
W.P.		LOCATION				ORIGINATED BY		ID												
DIST		BOREHOLE TYPE				COMPILED BY		AB												
DATUM		DATE				CHECKED BY		SEP												
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p		W		W _L		γ		GR SA SI CL	
194.3	0.0	GROUND SURFACE						194	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		20 40 60		20 40 60		20 40 60		kN/m ³			
192.7	1.6	End of Dynamic Cone Penetration Test Refusal to Further Penetration						193	100 blows / 100 mm											



PROJECT		RECORD OF PENETRATION TEST				No DCPT G-2		1 OF 1		METRIC					
W.P.		5622-02-00		LOCATION		N 4980471.0 ; E 282857.0		ORIGINATED BY		ID					
DIST		HWY 400		BOREHOLE TYPE		DYNAMIC CONE PENETRATION TEST		COMPILED BY		AB					
DATUM		Geodetic		DATE		May 18, 2006		CHECKED BY		SEP					
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
193.8	GROUND SURFACE														
0.0	Start of Dynamic Cone Penetration Test														
191.8															
2.0	End of Dynamic Cone Penetration Test Refusal to Further Penetration														

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD



PROJECT		RECORD OF PENETRATION TEST				No DCPT G-3		1 OF 1		METRIC										
W.P.		LOCATION				ORIGINATED BY		ID												
DIST		BOREHOLE TYPE				COMPILED BY		AB												
DATUM		DATE				CHECKED BY		SEP												
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p		W		W _L		γ		GR SA SI CL	
194.4	0.0	GROUND SURFACE						194	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		20 40 60		20 40 60		20 40 60		kN/m ³			
		Start of Dynamic Cone Penetration Test						193												
192.0	2.3	End of Dynamic Cone Penetration Test Refusal to Further Penetration							100 blows / 200 mm											

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD



PROJECT		RECORD OF PENETRATION TEST				No DCPT G-4		1 OF 1		METRIC										
W.P.		LOCATION				ORIGINATED BY		ID												
DIST		BOREHOLE TYPE				COMPILED BY		AB												
DATUM		DATE				CHECKED BY		SEP												
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p		W		W _L		γ		GR SA SI CL	
194.9	0.0	GROUND SURFACE							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		20 40 60		20 40 60		20 40 60		kN/m ³			
194.0	0.9	End of Dynamic Cone Penetration Test Cone sliding to northeast						194												

Heather Path Trail

Station 9+575 to 9+635, Referenced to C/L

05-1181-134

May - June, 2006

9+575 12.00 Rt C/L D-050 HA

0 - 290 Wat
290 - 700 Blk Org, Sat, Soft
700 - 790 Gry Cl Si Tr F Sa, Sat, Firm
790 - 1.20 Gry Sa Si Tr Cl Tr Gr, Sat, Comp*
1.20 - 2.40 Gry Cl Si Tr F Sa, Sat, Firm, Soft @
1.70
2.40 - 2.50 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.50 NFP BR

* Sample Depth = 900 - 1.20
w = 21 %

9+600 6.00 Rt C/L D-350 HA

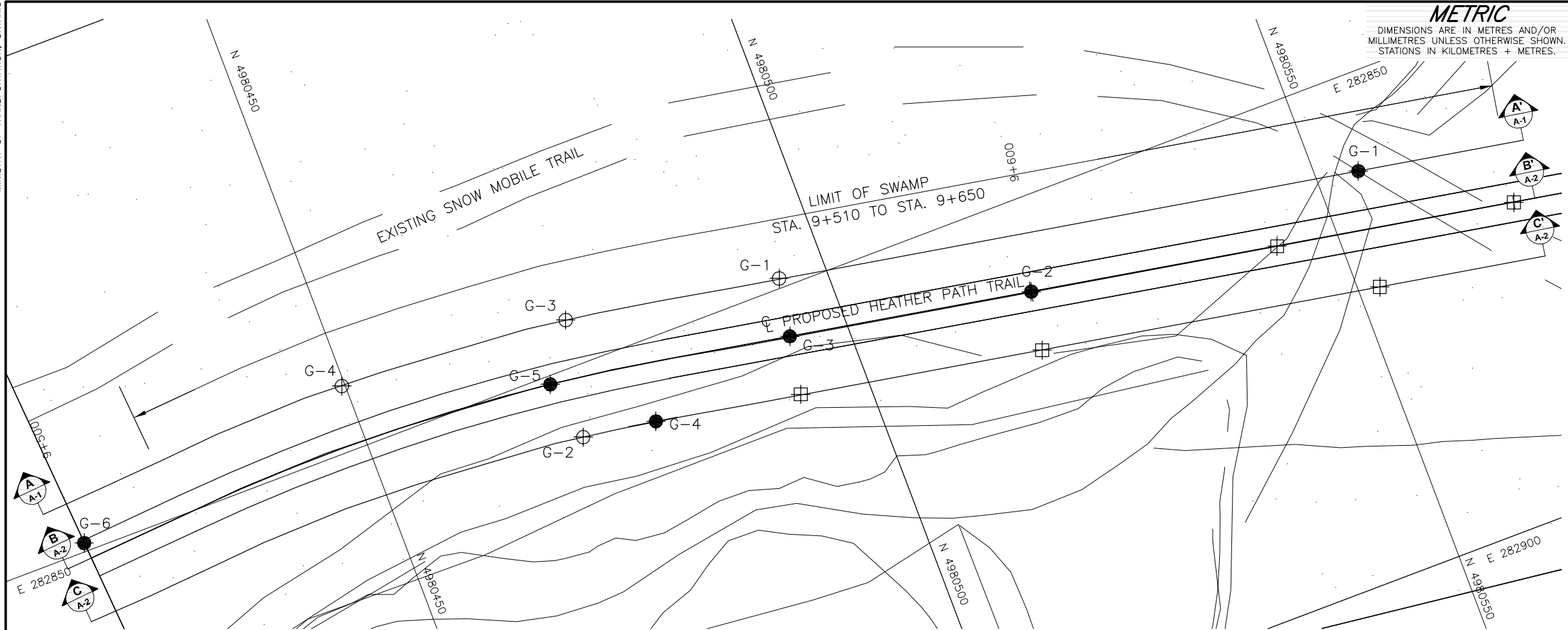
0 - 170 Wat
170 - 300 Blk Org, Sat, Soft
300 - 1.30 Gry Si Cl Tr F Sa, Sat, Soft
1.30 - 1.40 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 1.40 NFP BR

9+625 C/L D-0 HA

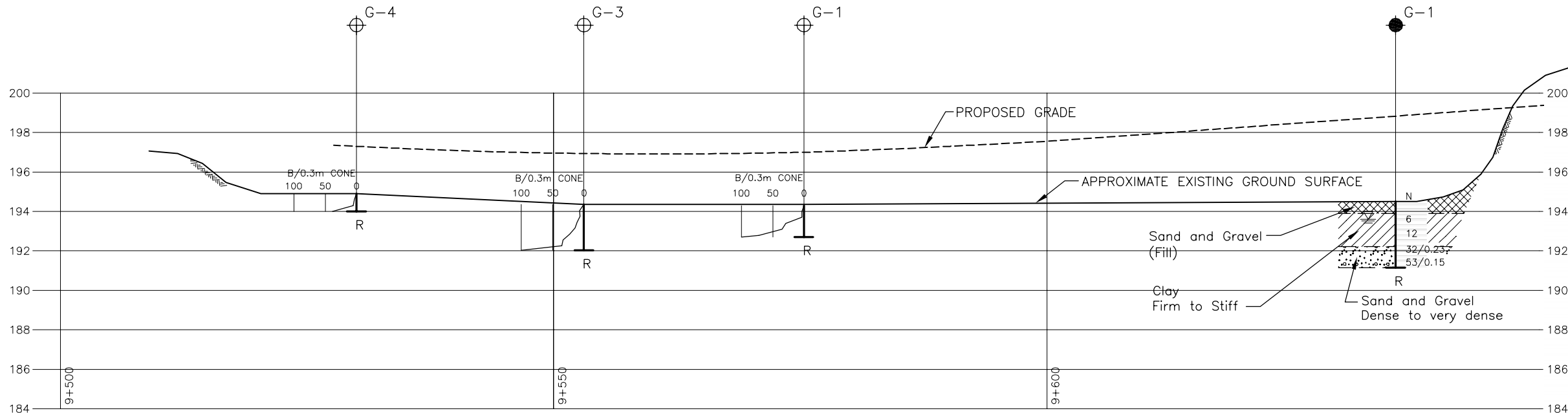
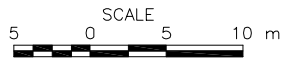
0 - 180 Dk Br Si Tps
180 - 800 Br Cl Si W Sa, Fr Wat @ 400, Sat, Firm
800 - 1.10 Br Sa Si Tr Cl Tr Gr Occ Cob, Sat,
Comp
1.10 - 2.20 Br Si Cl Tr F Sa, Sat, Soft @ 1.80
- 2.20 NFP Blds

9+635 6.00 Rt C/L D+800 HA

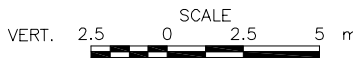
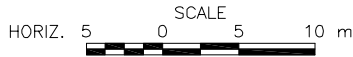
0 - 130 Dk Br Si Tps
130 - 900 Br Si Sa Tr Gr Occ Cob, Moist, Comp
- 900 NFP Blds



PLAN



PROFILE A-A'



METRIC

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

CONT No.
WP No. 5622-02-00

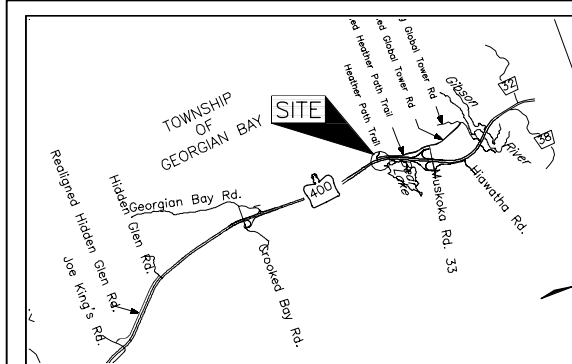
HEATHER PATH TRAIL
STA 9+510 TO 9+650
BOREHOLE LOCATIONS AND
SOIL STRATA



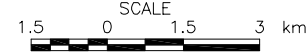
SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole
- Dynamic Cone Penetration Test
- Probehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- R Refusal
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
G-1	194.5	4980554	282860
G-2	194.5	4980519	282859
G-3	193.8	4980494	282855
G-4	193.8	4980478	282858
G-5	194.0	4980470	282851
G-6	198.0	4980419	282849
DCPT G-1	194.3	4980495	282849
DCPT G-2	193.8	4980471	282857
DCPT G-3	194.4	4980473	282845
DCPT G-4	194.9	4980450	282843
No.	ELEVATION	STATION	OFFSET
	193.9	9+575	6.0 Rt
	194.5	9+600	6.0 Rt
	193.7	9+625	
	193.7	9+635	6.0 Rt
	197.2	9+650	

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.

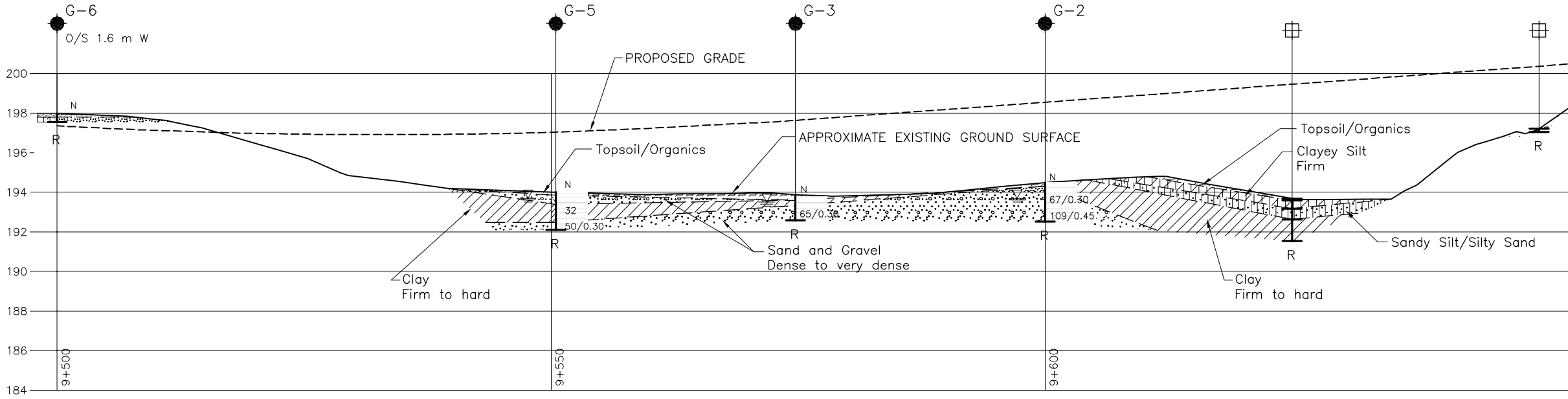
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

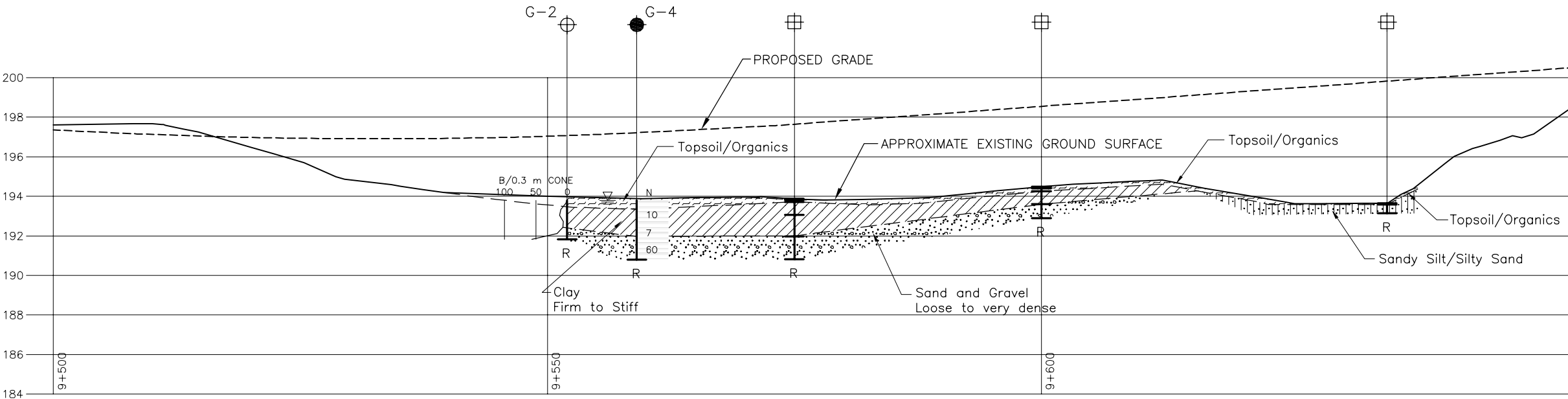
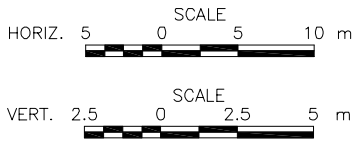
Base plan provided in digital format by URS, drawing file NC5-9+500_1+200.dwg, received July, 2006.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400	PROJECT NO. 05-1191-029		DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM/JFC	CHKD. SEP	APPD. JMAC	DWG. A-1

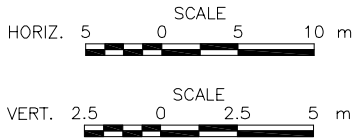
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.



PROFILE B-B'



PROFILE C-C'

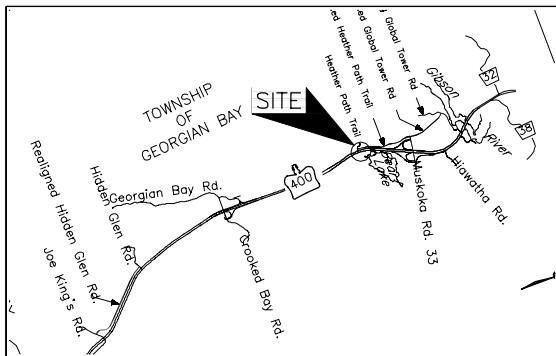


CONT No.
WP No. 5622-02-00

HEATHER PATH TRAIL
STA 9+510 TO 9+650
BOREHOLE SOIL STRATA
SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- Probehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
G-1	194.5	4980554	282860
G-2	194.5	4980519	282859
G-3	193.8	4980494	282855
G-4	193.8	4980478	282858
G-5	194.0	4980470	282851
G-6	198.0	4980419	282849
DCPT G-1	194.3	4980495	282849
DCPT G-2	193.8	4980471	282857
DCPT G-3	194.4	4980473	282845
DCPT G-4	194.9	4980450	282843
No.	ELEVATION	STATION	OFFSET
	193.9	9+575	6.0 Rt
	194.5	9+600	6.0 Rt
	193.7	9+625	
	193.7	9+635	6.0 Rt
	197.2	9+650	

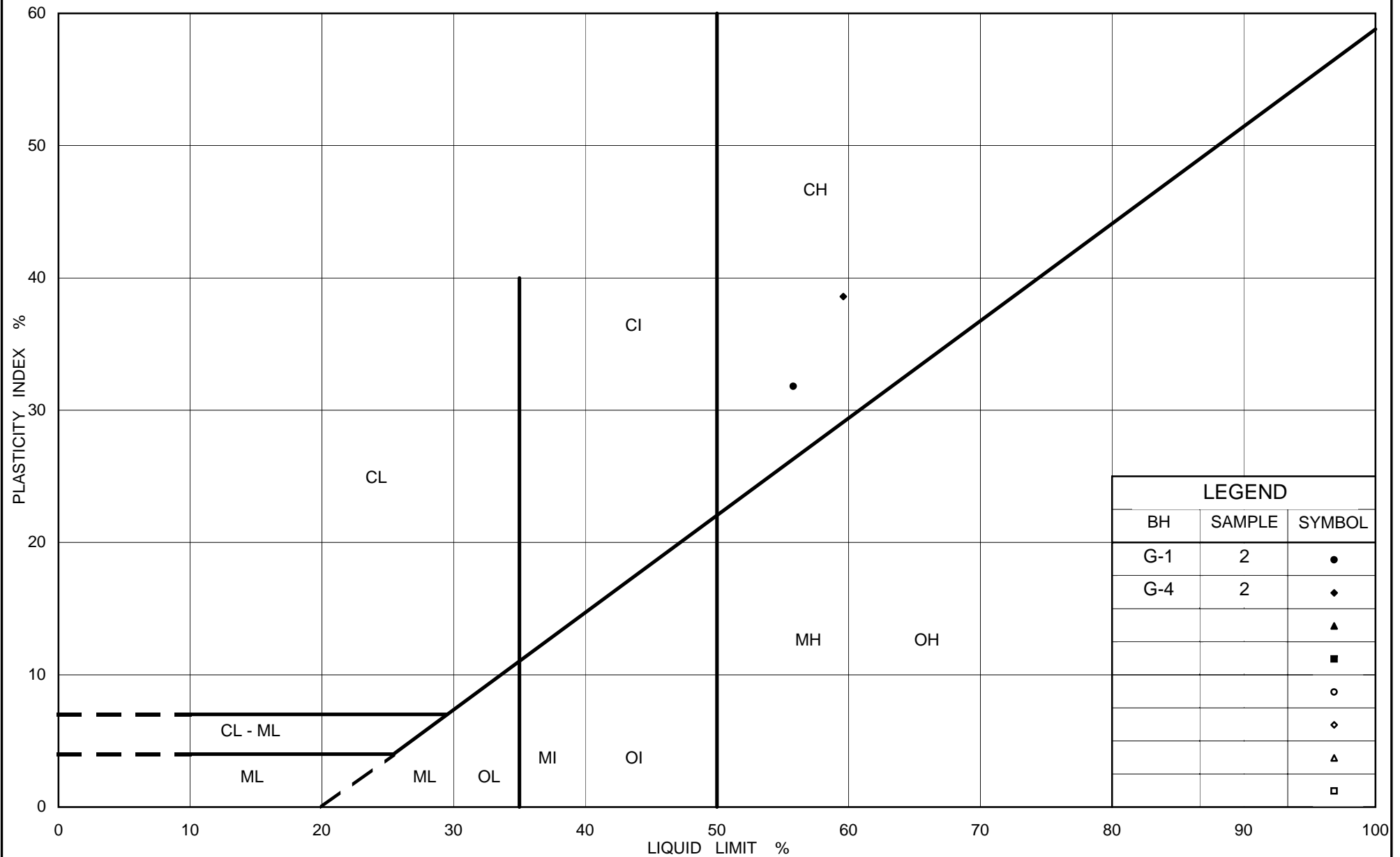
NOTES

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.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400		PROJECT NO. 05-1191-029	DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM/JFC	CHKD. SEP	APPD. JMAC	DWG. A-2



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



Ministry of Transportation

Ontario

PLASTICITY CHART Clay

FIG No. A-1

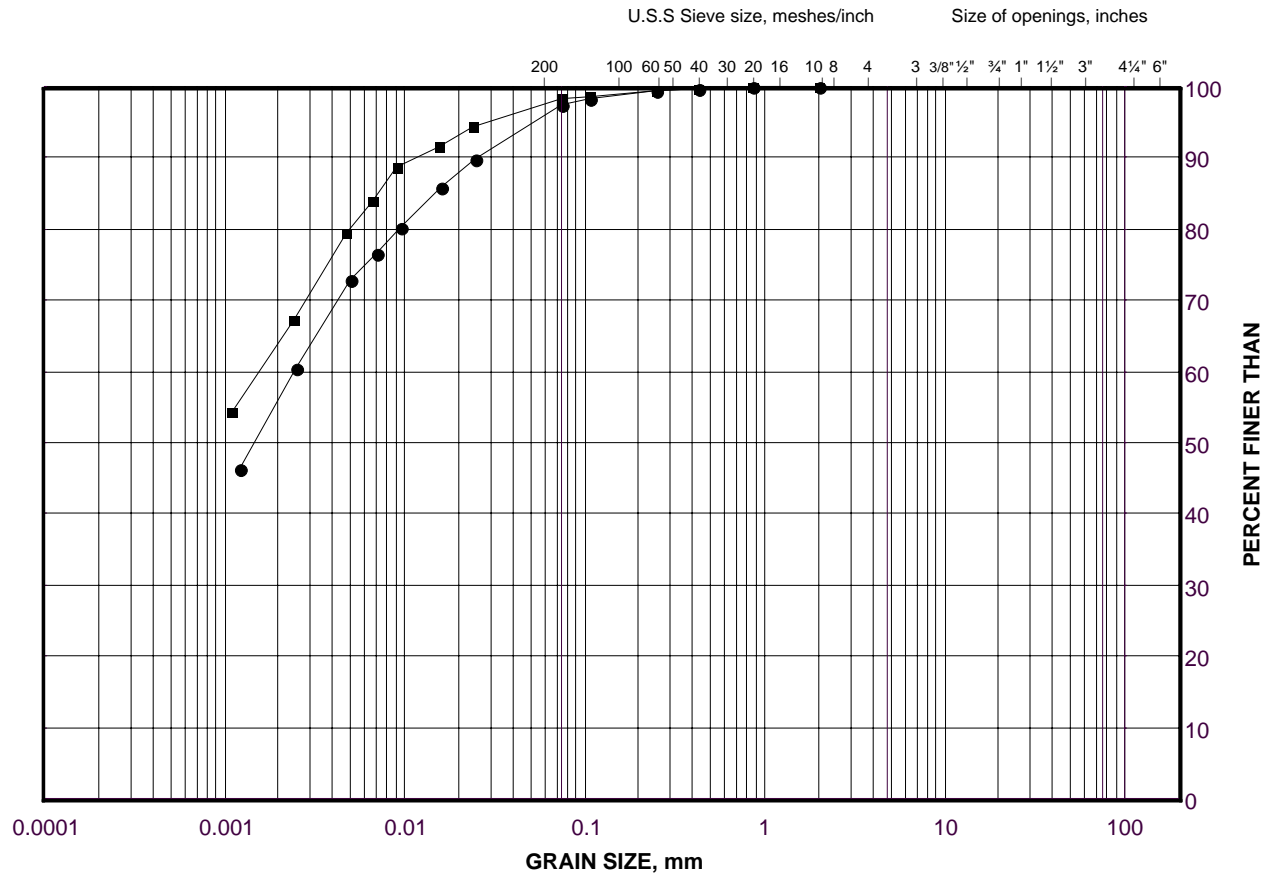
Project No. 05-1191-029

Checked By:

GRAIN SIZE DISTRIBUTION

Clay

FIGURE A-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	G-1	2	192.7
■	G-4	2	192.0

Project Number: 05-1191-029

Checked By: _____

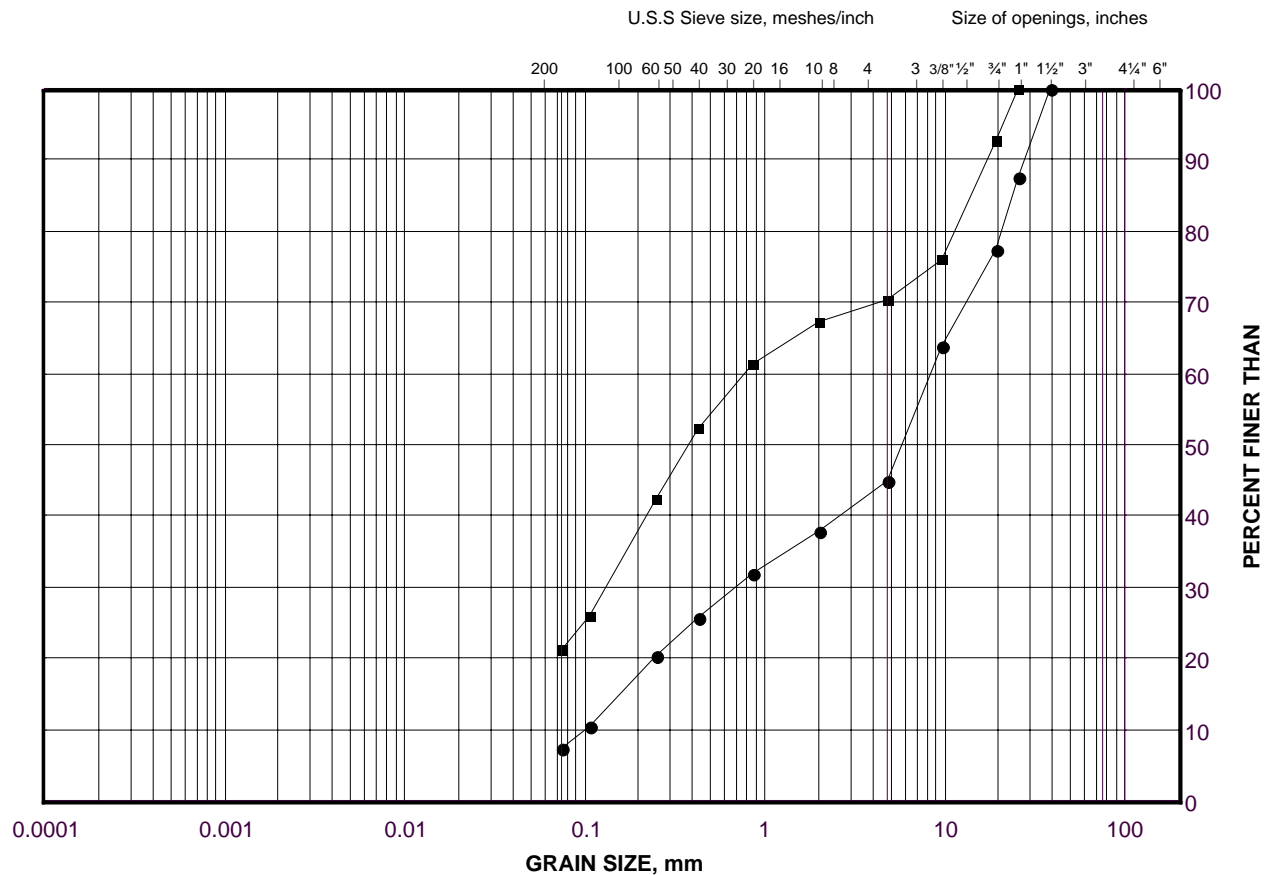
Golder Associates

Date: 16-Oct-06

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE A-3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	G-2	2	192.7
■	G-5	2	192.3

Project Number: 05-1191-029

Checked By: JMAC

Golder Associates

Date: 16-Oct-06

APPENDIX B
GLOBAL TOWER ROAD

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

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

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.)

Consistency

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

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, 60° 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 cm² 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.

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 test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(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

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

Global Tower Road

Station 10+075 to 10+150, Referenced to C/L

05-1181-134

May - June, 2006

10+075 C/L D-0 HA

0 - 420 Dk Br Si Tps, Fr Wat @ 180
420 - 520 Gry Sa Si Tr Cl Tr Gr, Sat, Comp
520 - 1.45 Gry Si Cl Tr F Sa, Sat, Firm
- 1.45 NFP BR

10+094 10.00 Rt C/L D-0 HA

0 - 150 Wat
150 - 500 Blk Org, Sat, Soft
500 - 1.90 Gry Si Cl Tr F Sa, Sat, Soft
1.90 - 2.10 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.10 NFP BR

10+098 C/L D-0 HA

0 - 200 Wat
200 - 700 Blk Org, Sat, Soft
700 - 2.20 Gry Si Cl Tr F Sa, Sat, Soft*
2.20 - 2.30 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.30 NFP BR

* Sample Depth = 1.50 - 1.80
w = 28 %

10+102 8.00 Lt C/L D-0 HA

0 - 300 Wat
300 - 1.20 Blk Org, Sat, Soft
1.20 - 2.00 Gry Si Cl W F Sa, Sat, Soft
2.00 - 2.10 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.10 NFP BR

10+125 C/L D-0 HA

0 - 250 Wat
250 - 900 Blk Org, Sat, Soft
900 - 2.50 Gry Si Cl Tr F Sa, Sat, Soft*
2.50 - 2.60 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.60 NFP BR

* Sample Depth = 2.00 - 2.30
w = 80 %

10+125 9.00 Lt C/L D-0 HA

0 - 300 Wat
300 - 1.10 Blk Org, Sat, Soft
1.10 - 4.70 Gry Si Cl Tr F Sa, Sat, Soft
4.70 - 5.00 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 5.00 NFP BR

10+125 9.00 Rt C/L D+1.40 HA

0 - 350 Dk Br Si Tps
- 350 NFP BR

10+150 C/L D-0 HA

0 - 050 Wat
050 - 500 Blk Org, Sat, Soft
500 - 2.20 Gry Si Cl Tr F Sa, Sat, Soft
2.20 - 2.30 Gry F-Co Sa W Si Tr Gr, Sat, Comp
- 2.30 NFP BR

Global Tower Road

Station 10+075 to 10+150, Referenced to C/L

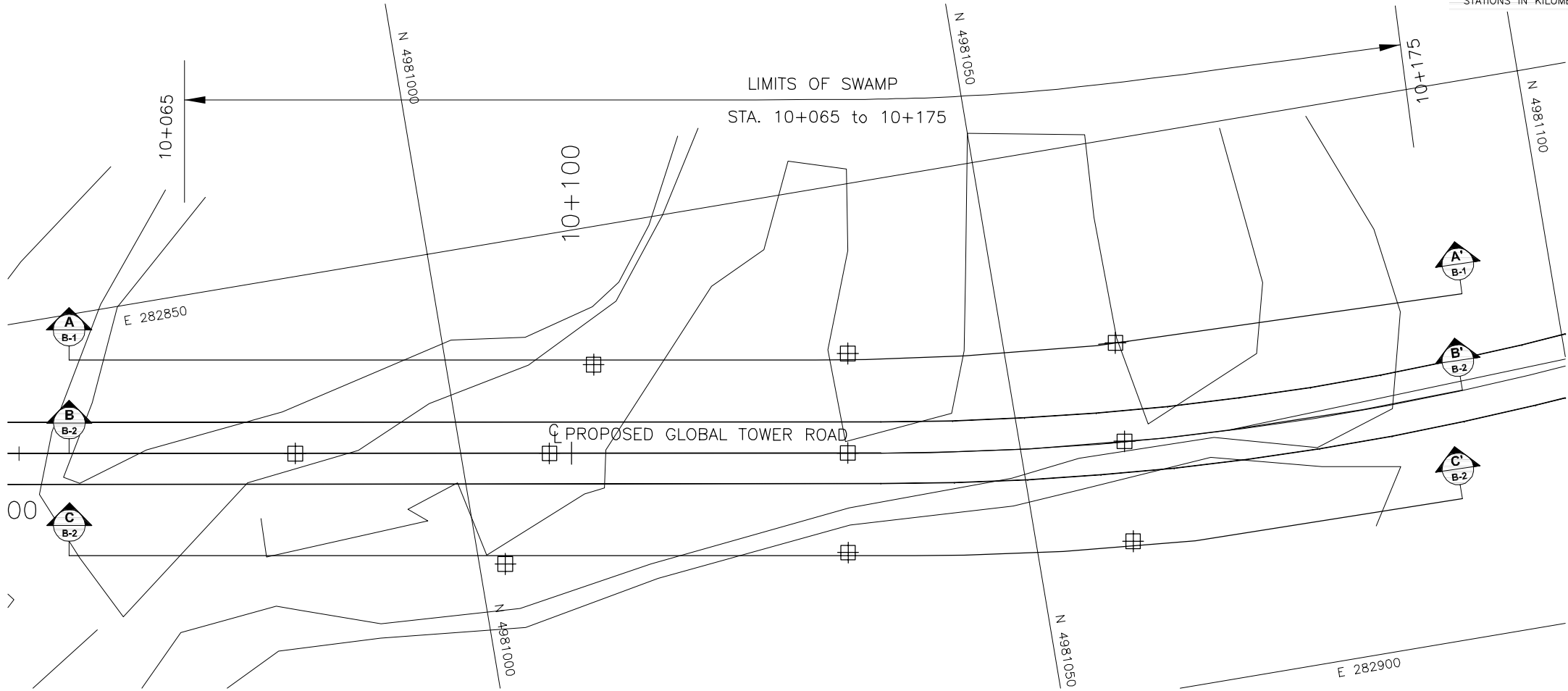
05-1181-134
May - June, 2006

10+150 9.00 Lt C/L D-0 HA

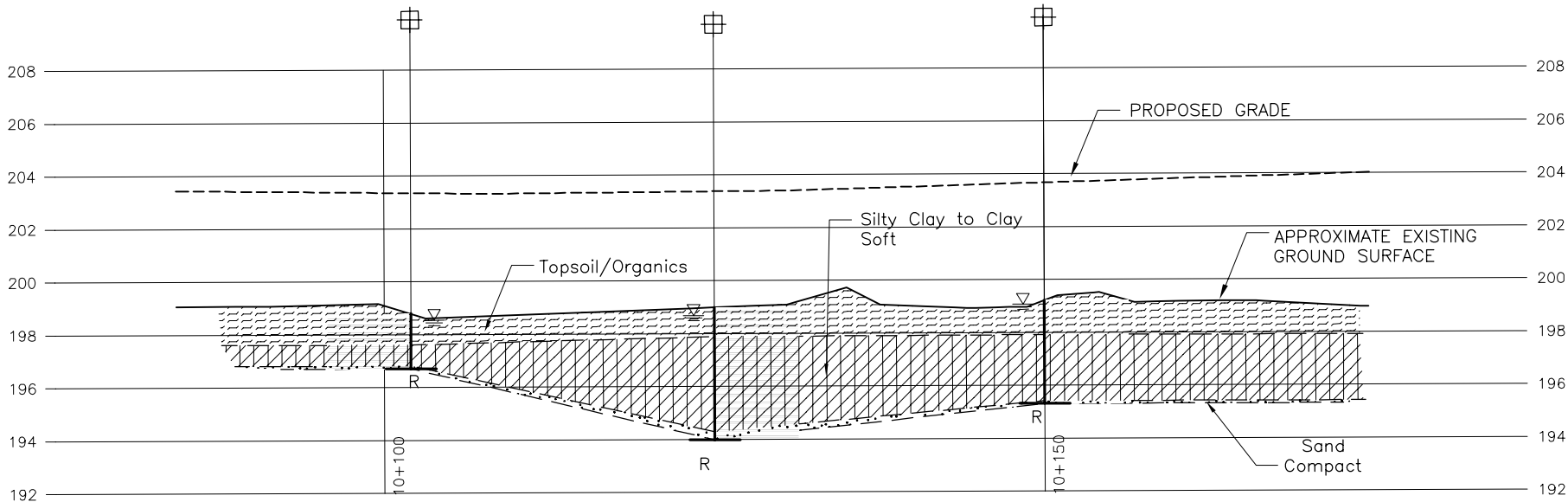
0	- 150	Wat
150	- 1.30	Blk Org, Sat, Soft
1.30	- 3.80	Gry Si Cl Tr F Sa, Sat, Soft
3.80	- 3.90	Gry F-Co Sa W Si W Gr, Sat, Comp
	- 3.90	NFP BR

10+150 9.00 Rt C/L D+1.10 HA

0	- 230	Dk Br Si Tps
	- 230	NFP BR



PLAN
SCALE
5 0 5 10 m



PROFILE A-A'
SCALE
HORIZ. 5 0 5 10 m
SCALE
VERT. 2.5 0 2.5 5 m

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

CONT No.
WP No. 5622-02-00

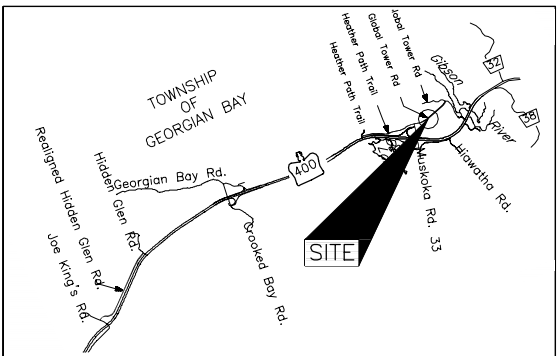
GLOBAL TOWER ROAD
STA 10+065 TO 10+175
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.5 0 1.5 3 km

LEGEND

- Probehole
- R Refusal

No.	ELEVATION	STATION	OFFSET
	199.2	10+075	
	199.1	10+094	10.0 m Rt
	199.1	10+098	
	198.8	10+102	8.0 m Lt
	199.0	10+125	
	199.0	10+125	9.0 m Lt
	200.4	10+125	9.0 m Rt
	199.2	10+150	
	199.2	10+150	9.0 m Lt
	200.3	10+150	9.0 m Rt

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file NCS-9+500_10+200.dwg, received July 26, 2006.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400		PROJECT NO. 05-1191-029	DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. SEP	APPD. JMAC	DWG. B-1

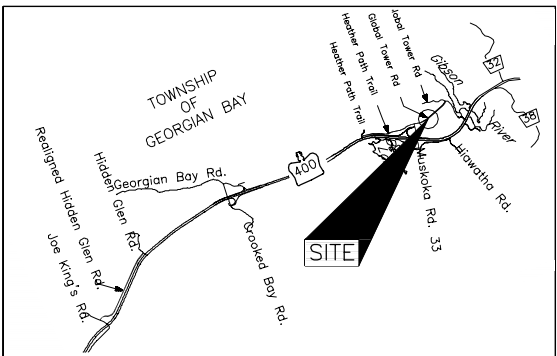
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5622-02-00

GLOBAL TOWER ROAD
STA 10+065 TO 10+175
BOREHOLE SOIL STRATA

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

	Probehole
R	Refusal

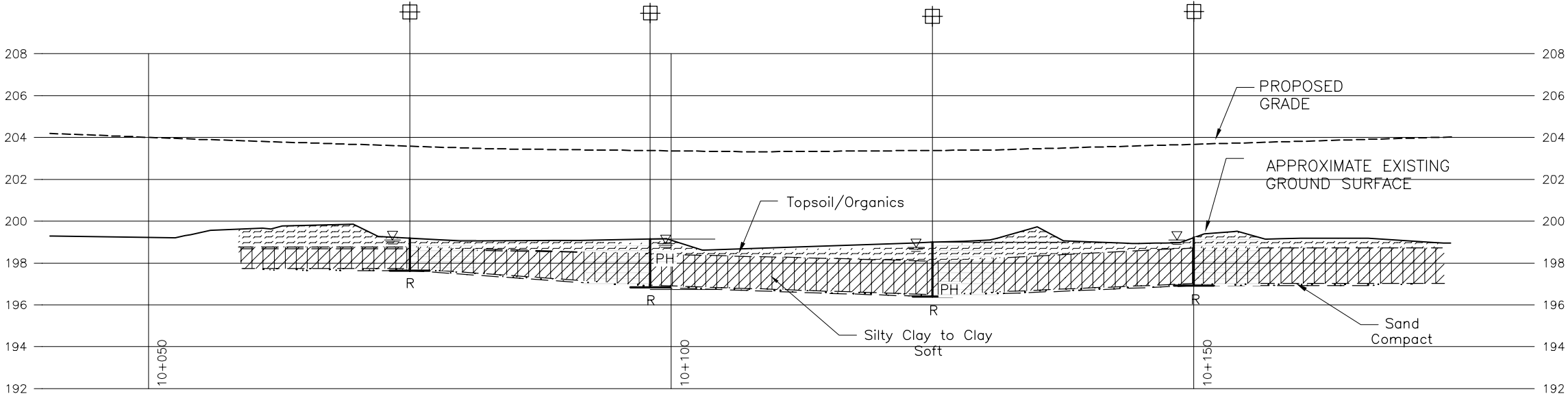
No.	ELEVATION	STATION	OFFSET
	199.2	10+075	
	199.1	10+094	10.0 m Rt
	199.1	10+098	
	198.8	10+102	8.0 m Lt
	199.0	10+125	
	199.0	10+125	9.0 m Lt
	200.4	10+125	9.0 m Rt
	199.2	10+150	
	199.2	10+150	9.0 m Lt
	200.3	10+150	9.0 m Rt

NOTES

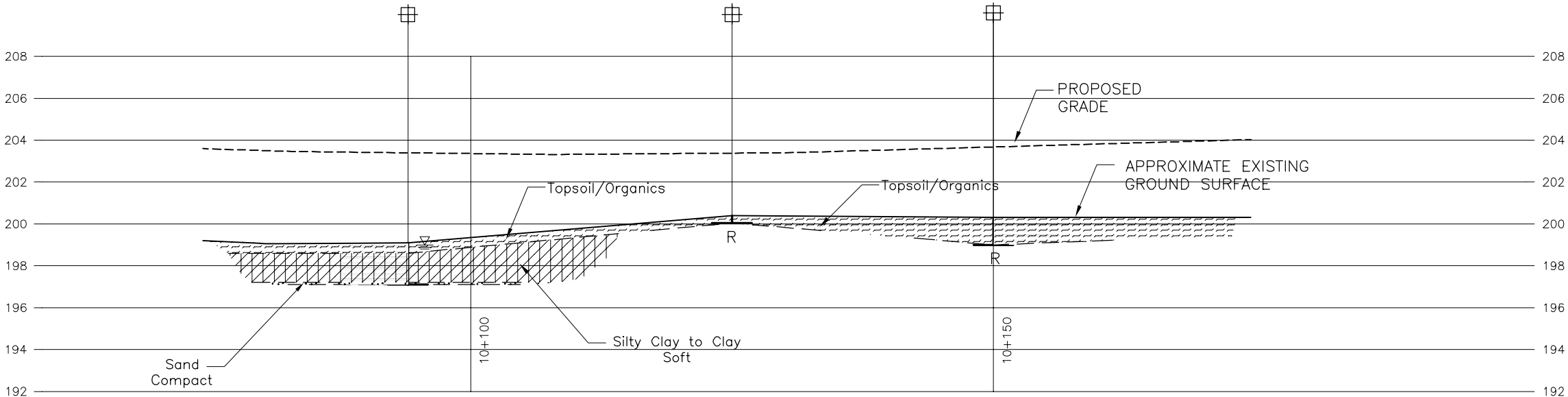
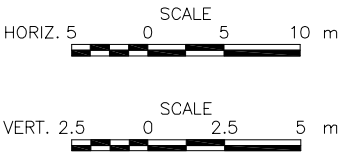
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

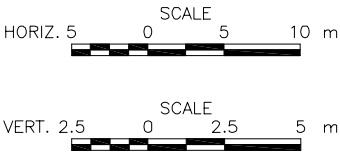
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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



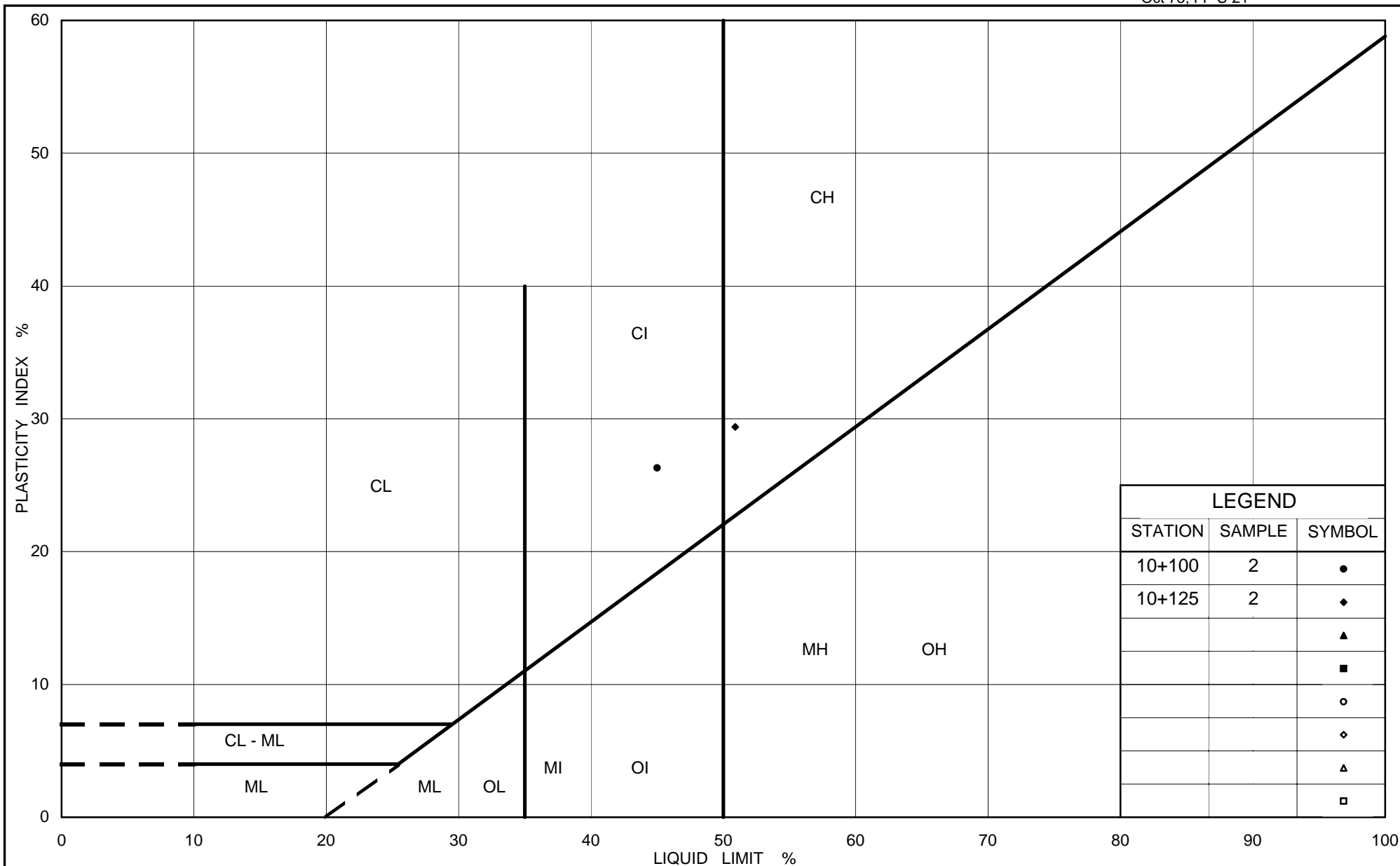
PROFILE B-B'



PROFILE C-C'



NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400		PROJECT NO. 05-1191-029	DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. SEP	APPD. JMAC	DWG. B-2



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay to Clay

FIG No. B-1

Project No. 05-1191-029

Checked By: JMAC

APPENDIX C
HIDDEN GLEN ROAD

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

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

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.)

Consistency

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

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, 60° 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 cm² 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.

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 test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(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

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 Shear strength = (Compressive strength)/2

PROJECT		RECORD OF BOREHOLE				No H-1		1 OF 1		METRIC							
W.P. 5622-02-00		LOCATION N 4973053.0 ; E 283846.0				ORIGINATED BY ID											
DIST HWY 400		BOREHOLE TYPE Wash Boring, BW Casing				COMPILED BY AB											
DATUM Geodetic		DATE March 20, 2006				CHECKED BY SEP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
203.2	GROUND SURFACE							20	40	60	80	100					
0.0	Peat, fibrous Very soft Brown Wet		1	SS	1		203										
202.6	Silty Clay Soft Grey Wet		2	SS	3		202										
0.6																	
201.7	Silty Sand, trace gravel Dense Wet		3	SS	50/0.1												
1.7	End of Borehole Spoon Refusal																
Notes: 1. Water level in open borehole at ground surface, upon completion of drilling operations. 2. Approximately 0.9 m of snow present at borehole location at time of drilling. 3. Spoon sliding to the northeast at 1.7 m depth.																	

PROJECT <u>05-1191-029</u>		RECORD OF BOREHOLE No H-2		1 OF 1 METRIC	
W.P. <u>5622-02-00</u>		LOCATION <u>N 4973072.0 ; E 283832.0</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Wash Boring, NW Casing</u>		COMPILED BY <u>AB</u>	
DATUM <u>Geodetic</u>		DATE <u>March 21, 2006</u>		CHECKED BY <u>SEP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
203.3	GROUND SURFACE							20 40 60 80 100		W _p W W _L					
0.0	Peat, fibrous Very soft Brown Wet		1	SS	1		203								
			2	SS	1		202								
201.6															
1.7	Silty Clay Soft Grey Wet		3	TO	PH										
200.9							201								
2.5	Silty Sand, trace gravel Very dense Grey Wet End of Borehole Spoon Refusal Notes: 1. Water level in open borehole at ground surface, upon completion of drilling operations. 2. Approximately 0.6 m of snow present at borehole location at time of drilling.		4	SS	60/100										

PROJECT		05-1191-029		RECORD OF BOREHOLE No H-3		1 OF 1 METRIC														
W.P.		5622-02-00		LOCATION		N 4973087.0 ; E 283812.0														
DIST		HWY 400		BOREHOLE TYPE		Wash Boring, BW Casing														
DATUM		Geodetic		DATE		March 20, 2006														
ORIGINATED BY		ID		COMPILED BY		AB														
CHECKED BY		SEP																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
203.2	0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			W _p W W _L 20 40 60			510					
		Peat, fibrous Very soft Dark brown Wet		1	SS	WH		203												
								202												
201.5	1.8	Alluvium Silty Clay Very soft Grey Wet		2	SS	WH		201												
200.5	2.8	Gravelly Sand Very dense Grey Wet		3	SS	WH														
200.0	3.2	End of Borehole Spoon Refusal		4	SS	17/0.2		200												
		Notes: 1. Water level in open borehole at ground surface, upon completion of drilling operations. 2. Approximately 0.3 m of snow and ice present at borehole location at time of drilling.																		

PROJECT		RECORD OF BOREHOLE No H-4				1 OF 1 METRIC							
W.P. 05-1191-029		LOCATION N 4973111.0; E 283801.0				ORIGINATED BY ID							
DIST HWY 400		BOREHOLE TYPE Wash Boring, NW Casing				COMPILED BY AB							
DATUM Geodetic		DATE March 21, 2006				CHECKED BY SEP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100						
203.2	GROUND SURFACE												
0.0	Peat, fibrous Very soft Dark brown Wet		1	SS	1								
201.8			2	SS	1								
1.4	Gravelly Sand, some silt Compact to very dense Grey Wet		3	SS	28								
200.4			4	SS	18/0.18								
2.8	End of Borehole Spoon Refusal Notes: 1. Water level in open borehole at ground surface, upon completion of drilling operations. 2. Approximately 0.6 m of snow present at borehole location at time of drilling.												

PROJECT		5-1191-029		RECORD OF BOREHOLE No H-5		1 OF 1 METRIC												
W.P.		5622-02-00		LOCATION		N 4973093.0 ; E 283783.0												
DIST		HWY 400		BOREHOLE TYPE		Wash Boring, NW Casing												
DATUM		Geodetic		DATE		March 21, 2006												
ORIGINATED BY		ID		COMPILED BY		AB												
CHECKED BY		SEP																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
203.2	0.0	GROUND SURFACE							20	40	60	80	100	20	40	60		
		Peat, fibrous Very soft Brown Wet		1	SS	1		203										
				2	SS	1		202										
201.4																		
201.1		Silty Clay Very soft Grey Wet		3	SS	16/0.15		201										
2.3		Gravelly Sand Very dense Grey End of Borehole Spoon Refusal																
		Notes:																
		1. Water level in open borehole at ground surface, upon completion of drilling operations.																
		2. Approximately 0.6 m of snow present at borehole location at time of drilling.																

PROJECT		05-1191-029		RECORD OF BOREHOLE No H-6		1 OF 1 METRIC											
W.P.		5622-02-00		LOCATION		N 4973120.0 ; E 283774.0											
DIST		HWY 400		BOREHOLE TYPE		Wash Boring, NW Casing											
DATUM		Geodetic		DATE		March 21, 2006											
ORIGINATED BY		ID		COMPILED BY		AB											
CHECKED BY		SEP															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR SA SI CL			
203.1	GROUND SURFACE																
0.0	Peat, fibrous Very soft Brown Wet		1	SS	1		203										
201.7			2	SS	6/0.15		202										
1.5	Gravelly Sand Very dense Brown to grey Wet End of Borehole Spoon Refusal Notes: 1. Water level in open borehole at ground surface, upon completion of drilling operations. 2. Approximately 0.5 m of snow present at borehole location at time of drilling. 3. Spoon sliding to the east at 1.5 m depth.																

PROJECT <u>05-1191-029</u>		RECORD OF BOREHOLE No H-7				1 OF 1 METRIC										
W.P. <u>5622-02-00</u>		LOCATION <u>N 4973137.0 ; E 283756.0</u>				ORIGINATED BY <u>ID</u>										
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Wash Boring, BW Casing</u>				COMPILED BY <u>AB</u>										
DATUM <u>Geodetic</u>		DATE <u>March 21, 2006</u>				CHECKED BY <u>SEP</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
203.0	GROUND SURFACE															
0.0	Peat	X	1	SS	55/0.10											
0.3	Sand and Gravel Very dense Brown and grey Wet End of Borehole Spoon Refusal Notes: 1. Water level in open borehole at ground surface upon completion of drilling operations. 2. Approximately 0.7 m of snow present at borehole location at time of drilling. 3. Moved borehole three times and encountered refusal at "shallower" depth.															



PROJECT		RECORD OF PENETRATION TEST				No DCPT H-1		1 OF 1		METRIC										
W.P.		LOCATION				ORIGINATED BY		ID												
DIST		BOREHOLE TYPE				COMPILED BY		AB												
DATUM		DATE				CHECKED BY		SEP												
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p		W		W _L		γ		GR SA SI CL	
203.2	0.0	GROUND SURFACE						203	20 40 60 80 100		20 40 60		20 40 60		20 40 60		kN/m ³			
		Start of Dynamic Cone Penetration Test						202												
								201												
200.1	3.1	End of Dynamic Cone Penetration Test Refusal to Further Penetration							60 blows / 75 mm											



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

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD



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

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD



PROJECT		RECORD OF PENETRATION TEST				No DCPT H-4		1 OF 1		METRIC							
W.P.		LOCATION				ORIGINATED BY		ID									
DIST		BOREHOLE TYPE				COMPILED BY		AB									
DATUM		DATE				CHECKED BY		SEP									
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p	W	W _L	WATER CONTENT (%)	γ	GR SA SI CL			
203.2	GROUND SURFACE																
0.0	Start of Dynamic Cone Penetration Test						203										
202.4																	
0.8	End of Dynamic Cone Penetration Test Refusal to Further Penetration																



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

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD

Hidden Glen Road

Station 11+102 to 11+225, Referenced to C/L

05-1181-134
May to July, 2006

11+102 9.00 Rt C/L D-0 HA

0 - 590 Wat
590 - 960 Muckamor, Sat, Soft
- 960 NFP BR

11+125 11.00 Lt C/L D-0 HA

0 - 200 Wat
200 - 1.75 Muckamor, Sat, Soft
1.75 - 1.80 Gry Co Sa W Si W Gr, Sat, Comp
1.80 - 2.50 Gry Si Cl Tr F Sa Tr Gr, Sat, Soft
- 2.50 NFP BR

11+150 9.00 Lt C/L D-0 HA

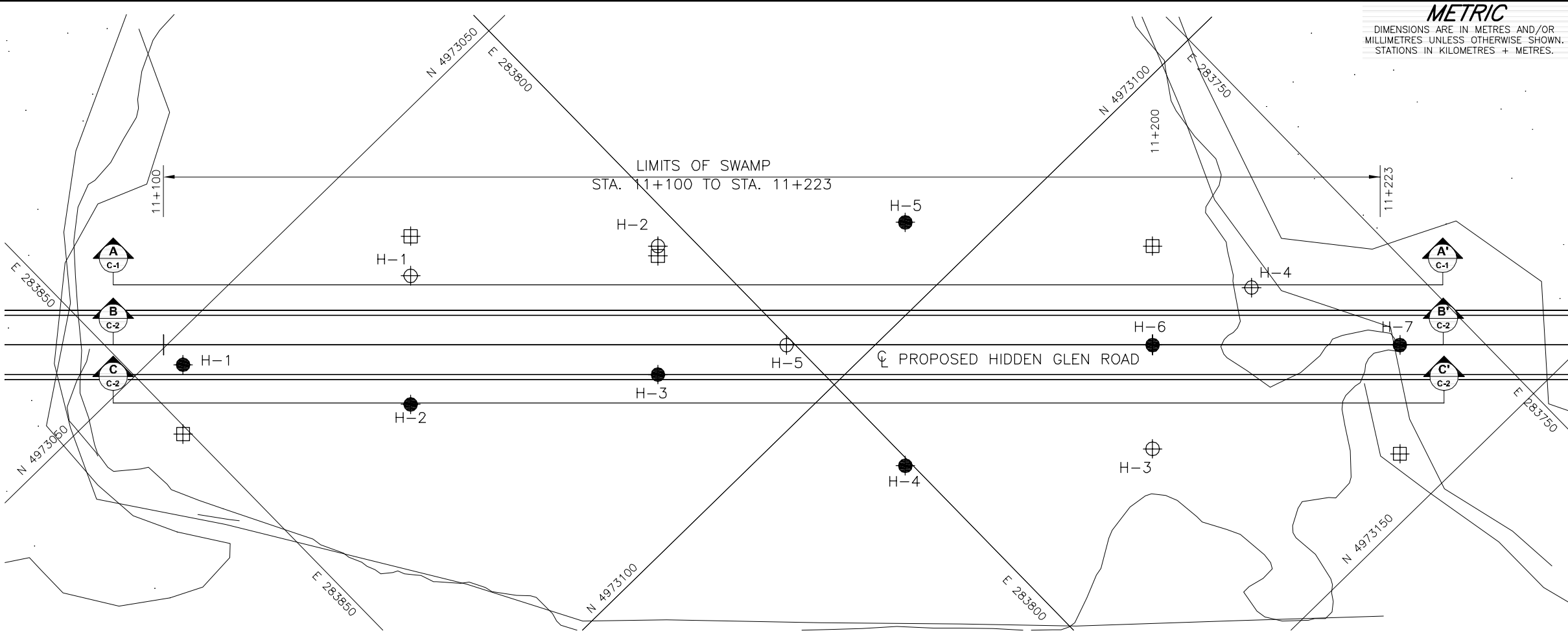
0 - 400 Wat
400 - 1.80 Muckamor, Sat, Soft
1.80 - 2.00 Gry Co Sa W Si W Gr, Sat, Comp
2.00 - 2.60 Gry Si Cl Tr F Sa Tr Gr, Sat, Soft
- 2.60 NFP BR

11+200 10.00 Lt C/L D-0 HA

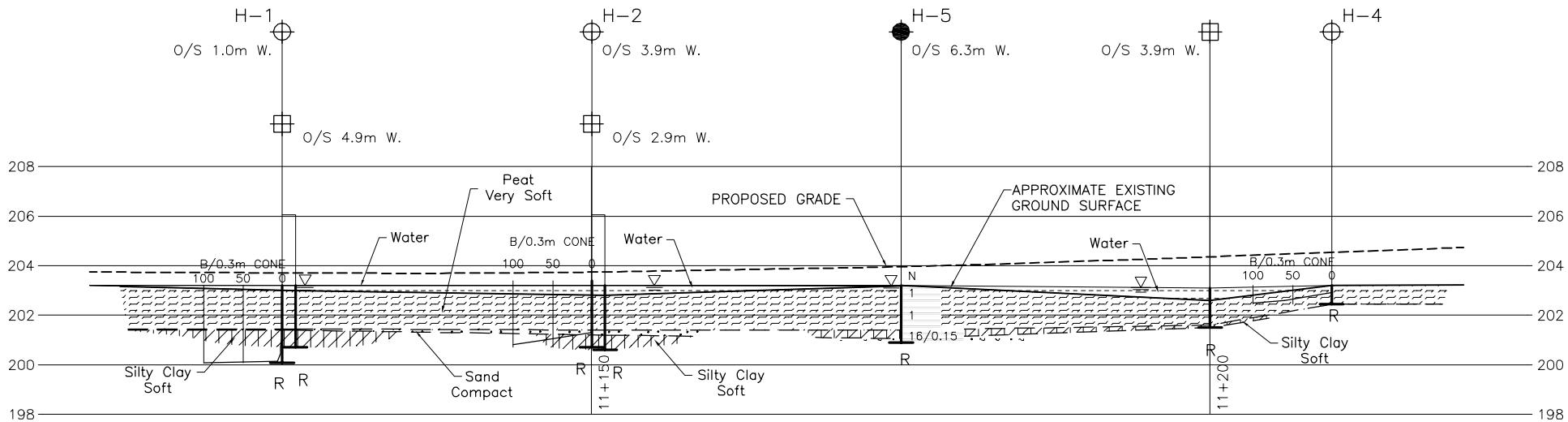
0 - 500 Wat
500 - 1.50 Muckamor, Sat, Soft
1.50 - 1.60 Gry Si Cl Tr F Sa Tr Gr, Sat, Soft
- 1.60 NFP BR

11+225 11.00 Lt C/L D+800 HA

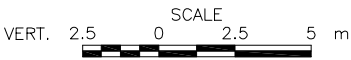
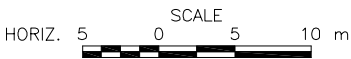
0 - 250 Dk Br Si Tps
- 250 NFP BR



PLAN



PROFILE A-A'



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

CONT No.
WP No. 5622-02-00

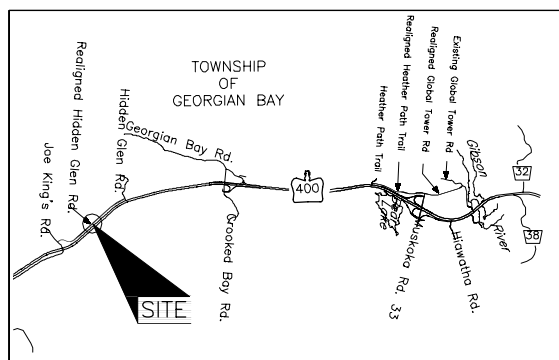


HIDDEN GLEN ROAD
STA 11+100 TO 11+223
BOREHOLE LOCATIONS AND
SOIL STRATA

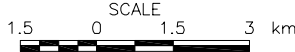
SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test
- ⊞ Probehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- R Refusal
- ≡ WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
H-1	203.2	4973053	283846
H-2	203.3	4973078	283803
H-3	203.2	4973087	283812
H-4	203.2	4973111	283801
H-5	203.2	4973093	283783
H-6	203.1	4973120	283774
H-7	203.0	4973137	283756
DCPT H-1	203.2	4973062	283823
DCPT H-2	203.4	4973085	283811
DCPT H-3	203.1	4973127	283781
DCPT H-4	203.2	4973122	283763
DCPT H-5	203.2	4973094	283801
No.	ELEVATION	STATION	OFFSET
	203.2	11+102	9.0 m Lt
	203.2	11+125	11.0 m Lt
	203.2	11+150	9.0 m Lt
	203.1	11+200	10.0 m Lt
	202.8	11+225	11.0 m Rt

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.

NO.	DATE	BY	REVISION
Geores No.			
HWY. 400	PROJECT NO. 05-1191-029		DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. SEP	APPD. JMAC	DWG. C-1

REFERENCE

Base plans provided in digital format by URS, drawing file NC2-10+600_11+300.dwg, received July, 2006.

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

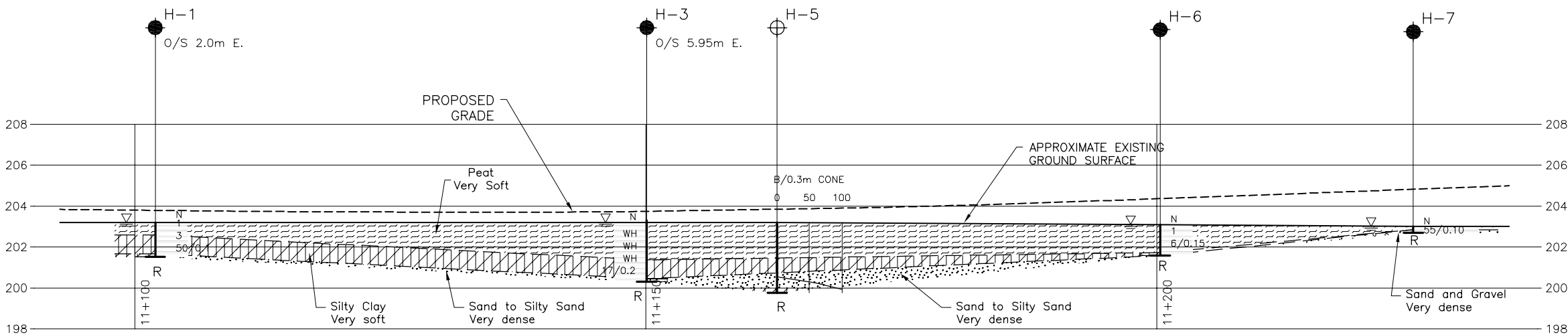
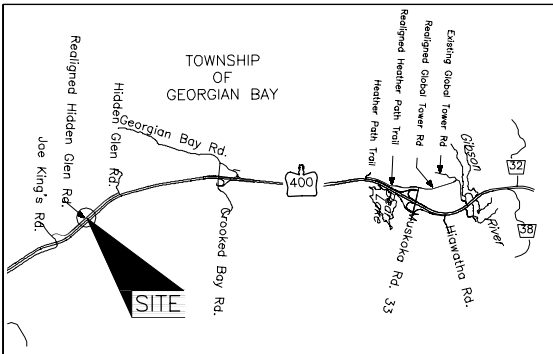
CONT No.
WP No. 5622-02-00

HIDDEN GLEN ROAD
STA 11+100 TO 11+223
BOREHOLE SOIL STRATA

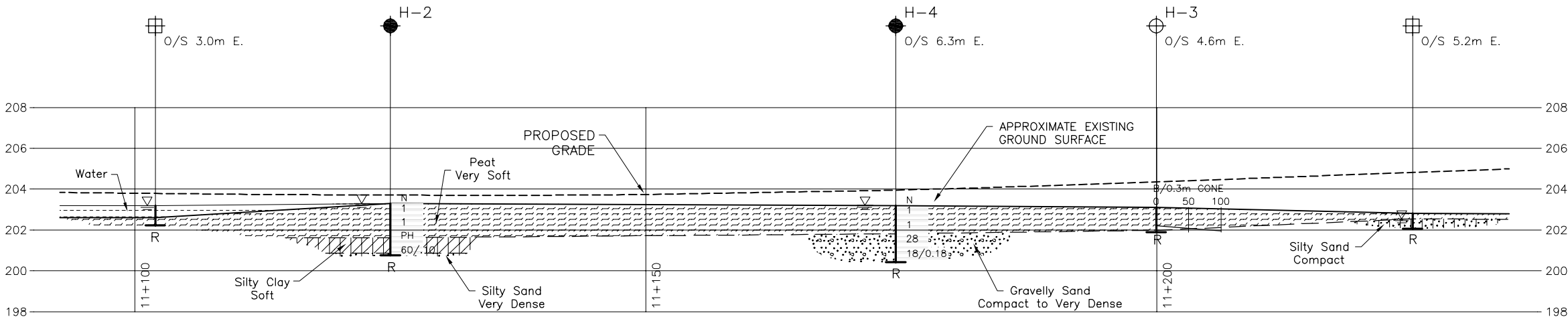
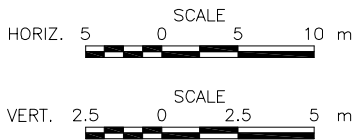
SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PROFILE B-B'



PROFILE C-C'



LEGEND

- Borehole
- Dynamic Cone Penetration Test
- Probehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
H-1	203.2	4973053	283846
H-2	203.3	4973078	283803
H-3	203.2	4973087	283812
H-4	203.2	4973111	283801
H-5	203.2	4973093	283783
H-6	203.1	4973120	283774
H-7	203.0	4973137	283756
DCPT H-1	203.2	4973062	283823
DCPT H-2	203.4	4973085	283811
DCPT H-3	203.1	4973127	283781
DCPT H-4	203.2	4973122	283763
DCPT H-5	203.2	4973094	283801
No.	ELEVATION	STATION	OFFSET
	203.2	11+102	9.0 m Lt
	203.2	11+125	11.0 m Lt
	203.2	11+150	9.0 m Lt
	203.1	11+200	10.0 m Lt
	202.8	11+225	11.0 m Rt

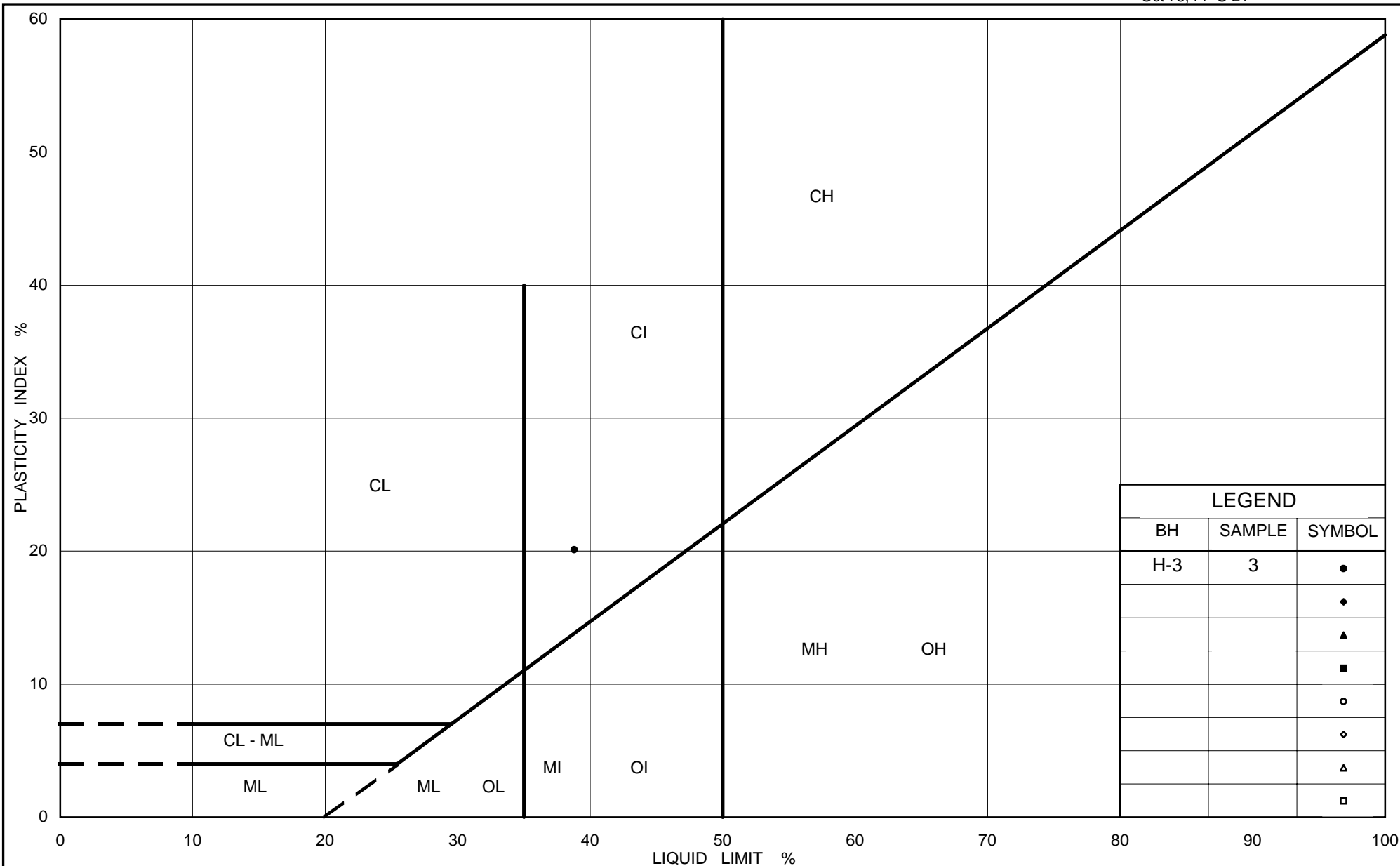
NOTES

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.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400		PROJECT NO. 05-1191-029	DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. SEP	APPD. JMAC	DWG. C-2



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay

FIG No. C-1

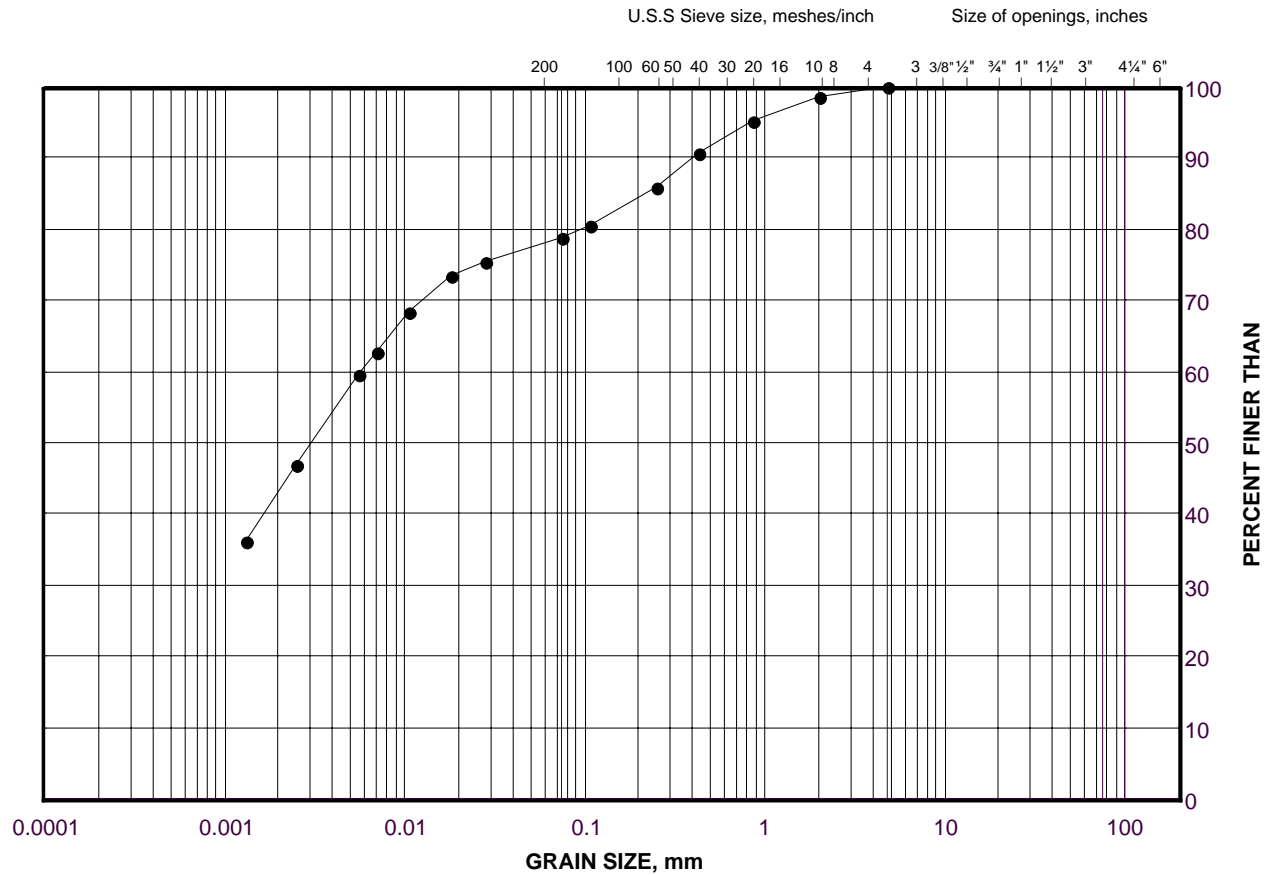
Project No. 05-1191-029

Checked By: JMAC

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE C-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	H-3	3	201.4

Project Number: 05-1191-029

Checked By: _____

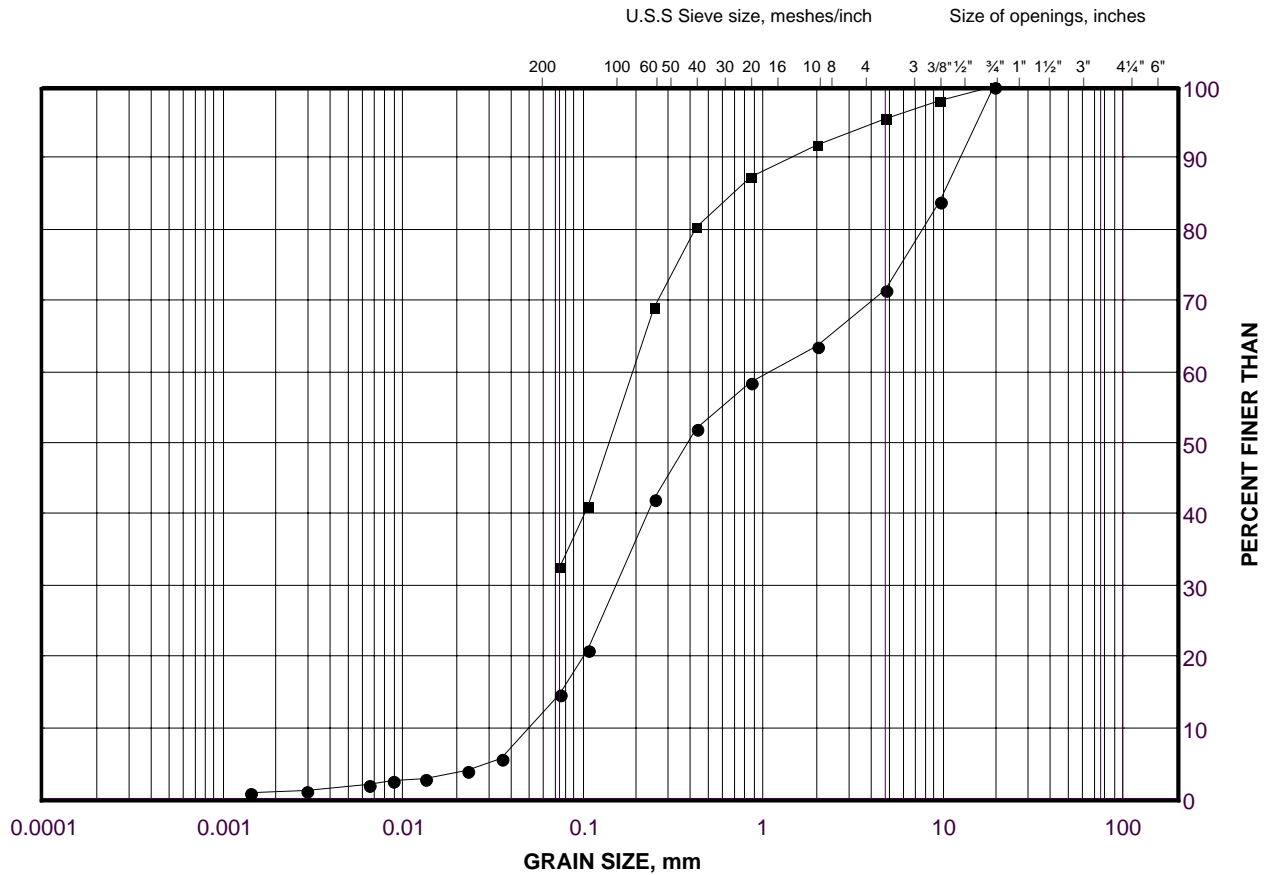
Golder Associates

Date: 16-Oct-06

GRAIN SIZE DISTRIBUTION

Silty Sand to Gravelly Sand

FIGURE C-3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	H-4	3	201.3
■	H-1	3	201.6

Project Number: 05-1191-029

Checked By: _____

Golder Associates

Date: 16-Oct-06

APPENDIX D
MUSKOKA ROAD 33 S-EW RAMP
(BEAR LAKE)

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

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

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.)

Consistency

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

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, 60° 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 cm² 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.

IV. SOIL TESTS

w water content
w_p plastic limit
w_i liquid limit
C consolidation (oedometer) test
CHEM chemical analysis (refer to text)
CID consolidated isotropically drained triaxial test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(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

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 Shear strength = (Compressive strength)/2

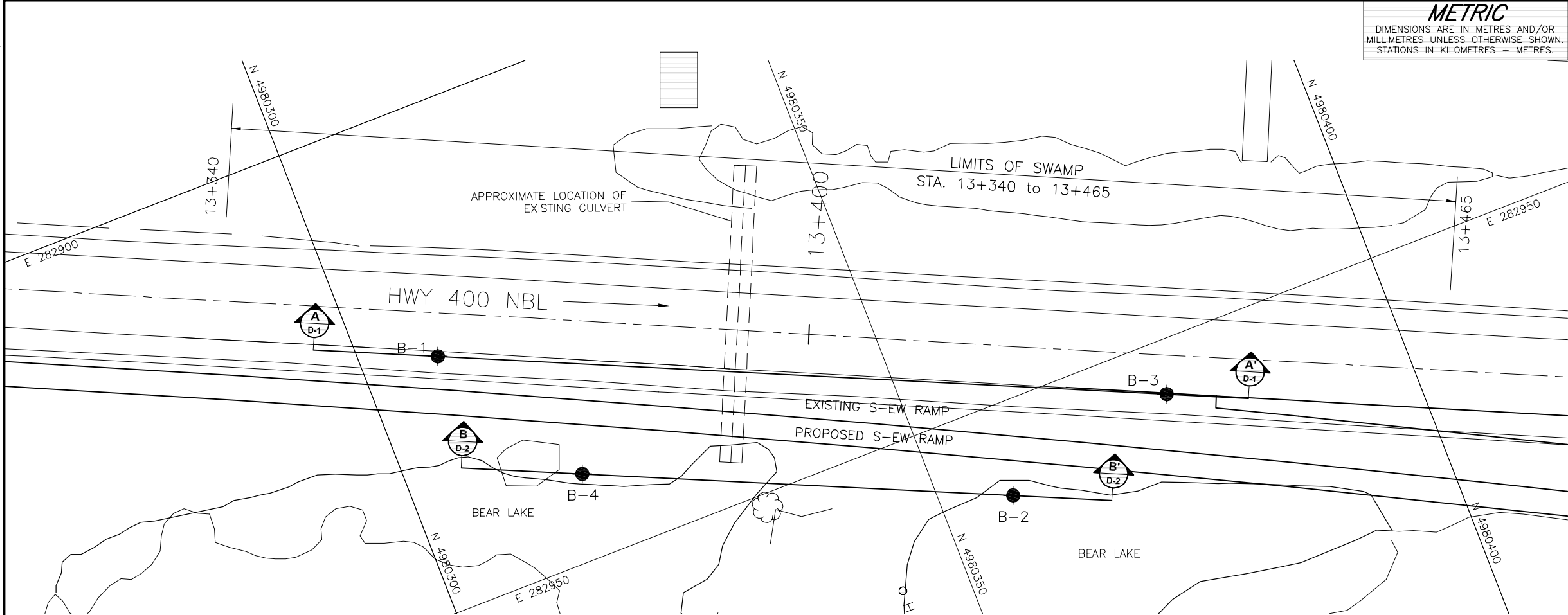
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

MIS-MTO 001 05-1191-029.GPJ GAL-MISS.GDT 17/10/06 DD

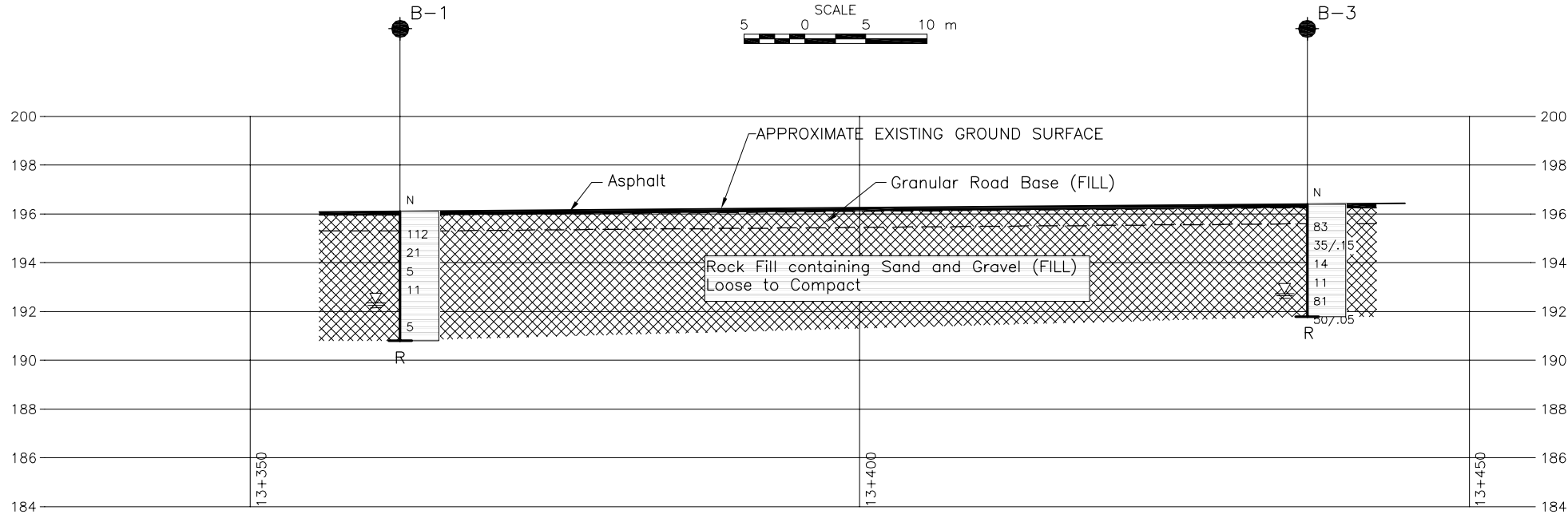
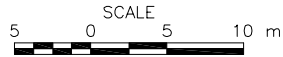
PROJECT <u>05-1191-029</u>		RECORD OF BOREHOLE No B-2				1 OF 1 METRIC											
W.P. <u>5622-02-00</u>		LOCATION <u>N 4980357.0 ; E 282959.0</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Power Auger, 83 mm ID Hollow Stem Augers</u>				COMPILED BY <u>AB</u>											
DATUM <u>Geodetic</u>		DATE <u>March 16, 2006</u>				CHECKED BY <u>SEP</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
192.9	GROUND SURFACE																
0.0	Ice																
192.4	Peat with occasional silty sand Very soft to soft Black to brown Wet Fibrous and spongy at 0.9 m depth		1	SS	1												
0.5			2	SS	1												
			3	SS	4												
190.3	Silty Sand, some gravel Very dense Grey Wet																
2.6			4	SS	51/0.18												
189.8																	
3.1	End of Borehole Auger and Spoon Refusal																
Notes: 1. Water level in open borehole at surface upon completion of drilling operations. 2. Approximately 0.3 m of snow present at borehole location at time of drilling. 3. Auger sliding to the east at 2.6 m depth.																	

PROJECT 05-1191-029			RECORD OF BOREHOLE No B-3			1 OF 1 METRIC																	
W.P. 5622-02-00			LOCATION N 4980376.0 ; E 282955.0			ORIGINATED BY ID																	
DIST HWY 400			BOREHOLE TYPE Power Auger, 83 mm ID Hollow Stem Augers			COMPILED BY AB																	
DATUM Geodetic			DATE March 16, 2006			CHECKED BY SEP																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
196.4	GROUND SURFACE																						
0.0	Asphalt																						
0.3	Granular 'A' (Fill)																						
195.6	Granular 'B' (Fill)																						
0.8	Blast Rock, containing sand and gravel, trace silt (Fill) Loose to compact Moist to wet		1	SS	83																		
	Voids observed between large rock fragments		2	SS	35/15																		
			3	SS	14																		
			4	SS	11																		
192.7																							
3.7	Sand, some silt, trace gravel, containing blast rock fragments (Fill) Very dense Grey to brown Wet		5	SS	81																		
191.8																							
4.6	End of Borehole Auger and Spoon Refusal		6	SS	50/05																		
Notes: 1. Auger refusal at 3.7 m depth. Borehole moved 2.1 m west and augered to 3.8 m without sampling. 2. Spoon bouncing at 1.7 m depth. 3. Water level in open borehole at 3.7 m depth (Elev. 192.7 m) upon completion of drilling operations. 4. Spoon sliding at 4.6 m depth.																							

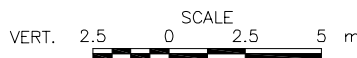
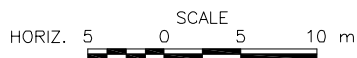
PROJECT 05-1191-029			RECORD OF BOREHOLE No B-4			1 OF 1 METRIC											
W.P. 5622-02-00			LOCATION N 4980317.0; E 282941.0			ORIGINATED BY ID											
DIST HWY 400			BOREHOLE TYPE Power Auger, 83 mm ID Hollow Stem Augers			COMPILED BY AB											
DATUM Geodetic			DATE March 16, 2006			CHECKED BY SEP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
193.0	GROUND SURFACE																
0.0	Peat, fibrous Stiff Brown Wet		1	SS	9												
192.4																	
0.6	Sand, trace silt, trace organics (Alluvium) Very loose to loose Grey Wet		2	SS	2		192									105.3	
			3	SS	5												
			4	SS	9		191										
190.6																	
2.4	Clay Firm Grey Wet		5	SS	2		190										
189.5																	
3.5	Gravelly Sand, trace silt Loose to compact Grey Wet		6	SS	10		189										
188.0																	
5.0	End of Borehole Auger Refusal																
	Notes: 1. Water level in open borehole at 0.6 m depth (Elev. 192.4 m) upon completion of drilling operations. 2. Approximately 0.6 m of snow present at borehole location at time of drilling. 3. Auger sliding to the east at 5.0 m depth.																



PLAN



PROFILE A-A'

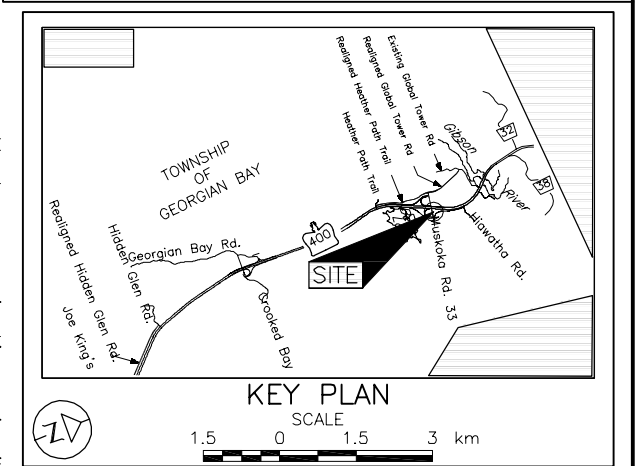




METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5622-02-00

MUSKOKA ROAD 33
S-EW RAMP (BEAR LAKE)
STA 13+340 TO 13+465
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
R	Refusal		
	WL upon completion of drilling		
CO-ORDINATES			
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B-1	196.1	4980308	282925
B-2	192.9	4980357	282959
B-3	196.4	4980376	282955
B-4	193.0	4980317	282941

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 plan provided in digital format by URS, drawing file nc6-13+450_10+550.dwg, received July, 2006.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400	PROJECT NO. 05-1191-029		DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM/JFC	CHKD. SEP	APPD. JMAC	DWG. D-1

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

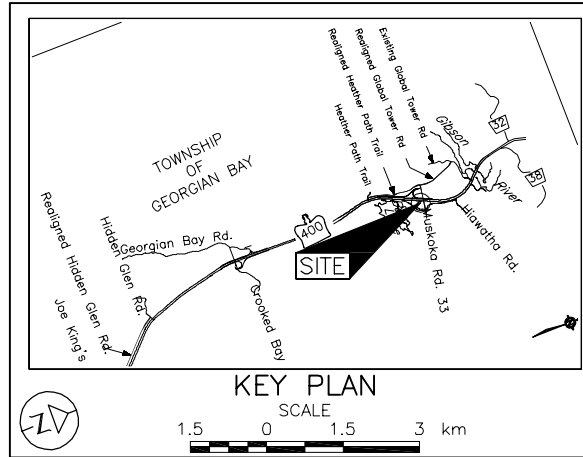
CONT No.
WP No. 5622-02-00

MUSKOKA ROAD 33
S-EW RAMP (BEAR LAKE)
STA 13+340 TO 13+465
BOREHOLE SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

-  Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
-  WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B-1	196.1	4980308	282925
B-2	192.9	4980357	282959
B-3	196.4	4980376	282955
B-4	193.0	4980317	282941

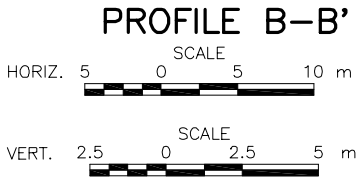
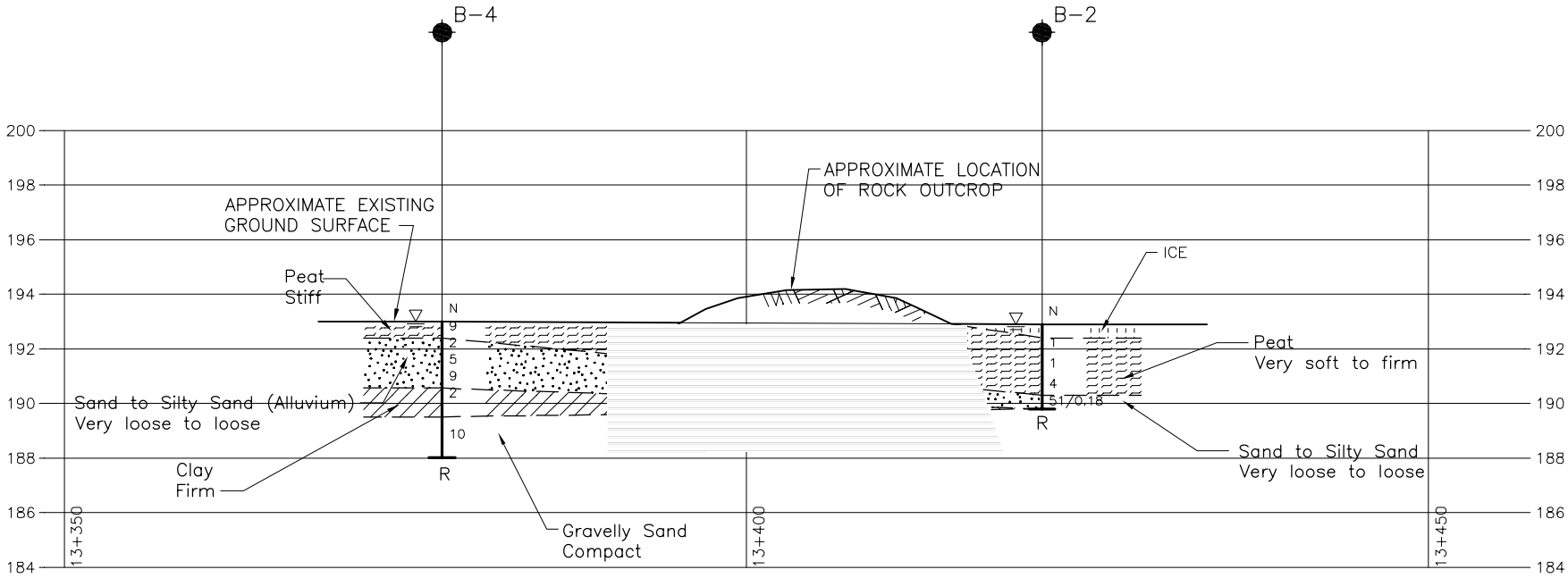
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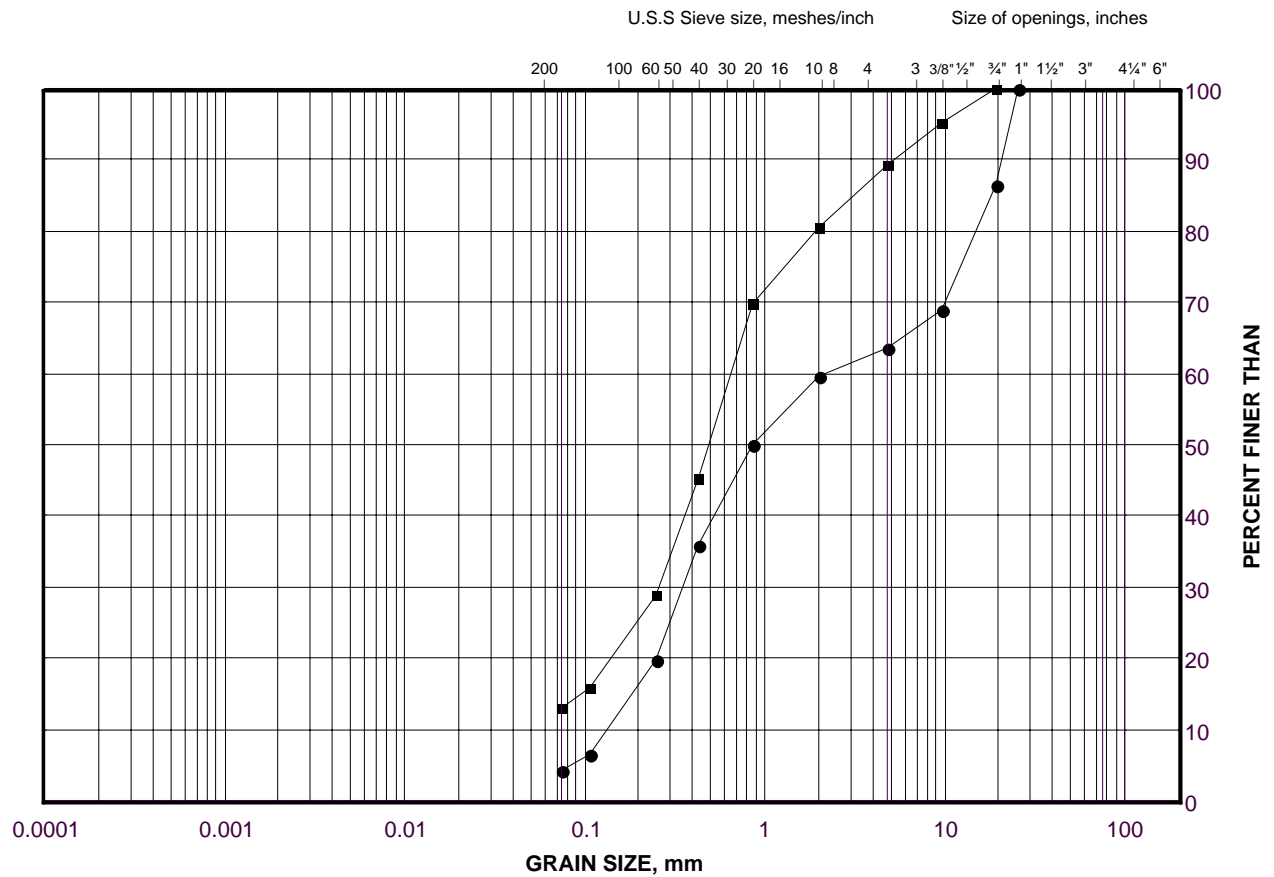
NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400		PROJECT NO. 05-1191-029	DIST.
SUBM'D. AB	CHKD. AB	DATE: OCT 2006	SITE:
DRAWN: MSM/JFC	CHKD. SEP	APPD. JMAC	DWG. D-2



GRAIN SIZE DISTRIBUTION

Rock Fill

FIGURE D-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B-1	5	191.2
■	B-3	5	192.2

Project Number: 05-1191-029

Checked By: _____

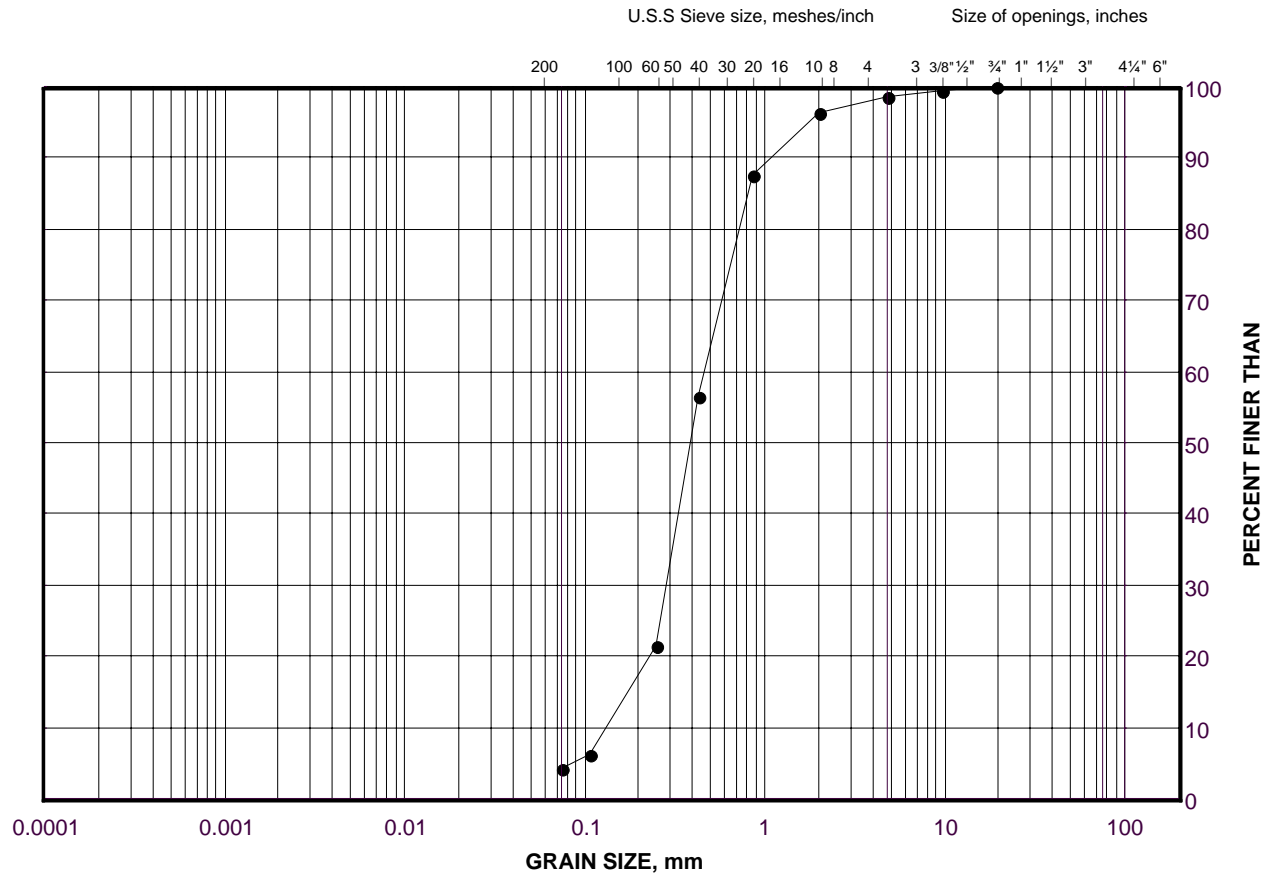
Golder Associates

Date: 16-Oct-06

GRAIN SIZE DISTRIBUTION

Sand (Alluvium)

FIGURE D-2



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

LEGEND

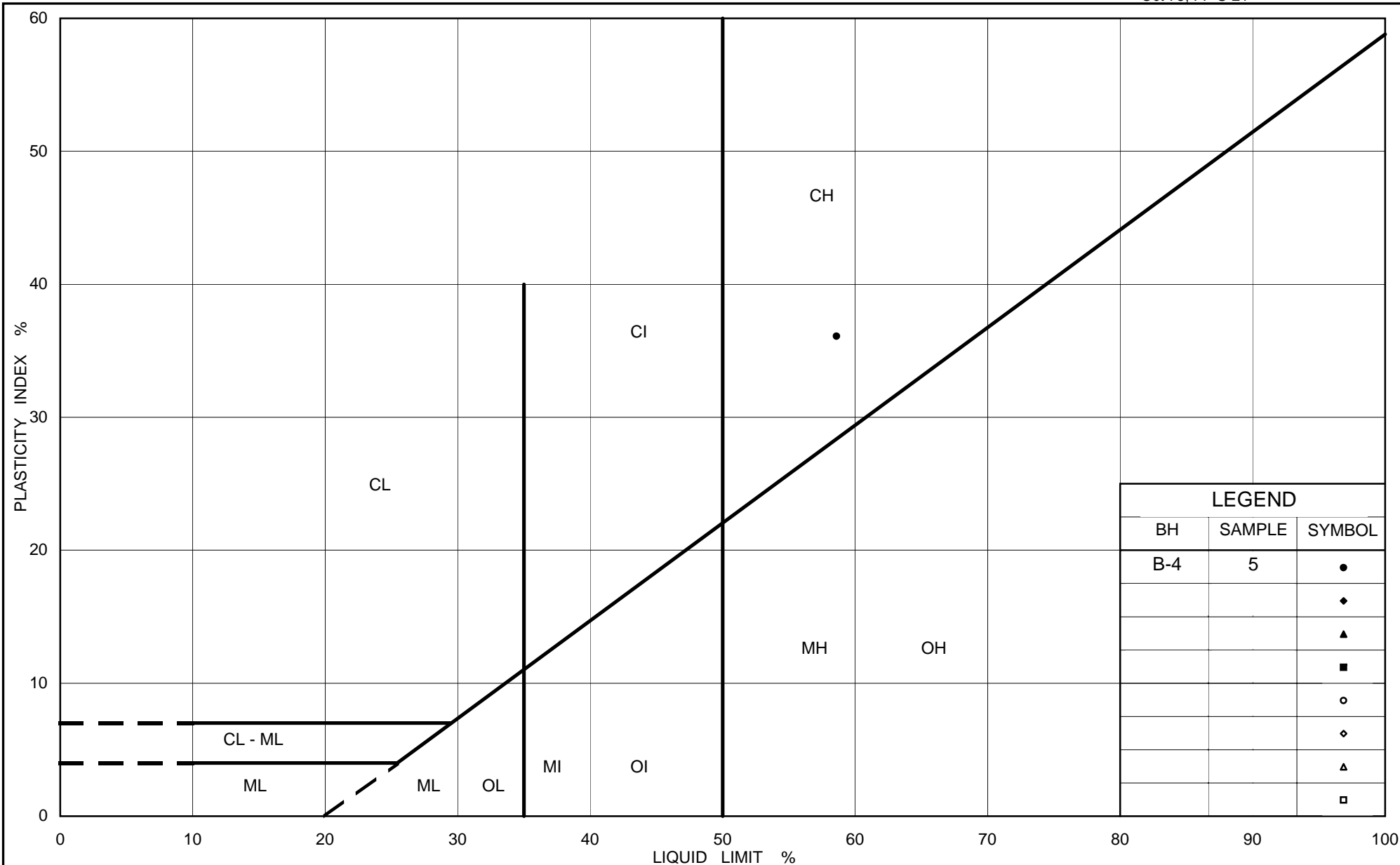
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B-4	3	191.5

Project Number: 05-1191-029

Checked By: _____

Golder Associates

Date: 16-Oct-06



Ministry of Transportation

Ontario

PLASTICITY CHART Clay

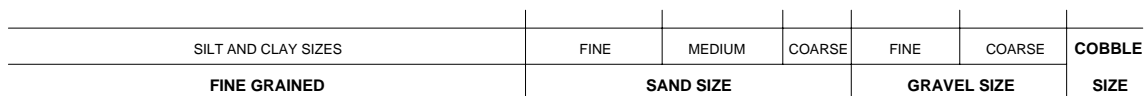
FIG No. D-3

Project No. 05-1191-029

Checked By:

Silty Sand to Gravelly Sand

FIGURE D-4



SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B-4	6	188.8
■	B-2	4	189.9

Date: 16-Oct-06

APPENDIX E

**ROCK CUT – MUSKOKA ROAD 33 STRUCTURE
(EAST ABUTMENT)**

METRIC

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

CONT No.
WP No. 5622-02-00

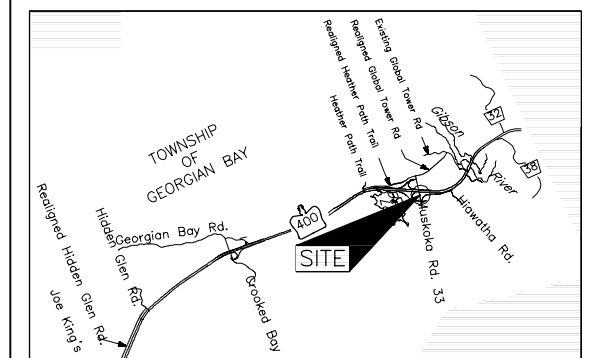


SHEET

MUSKOKA ROAD 33
STRUCTURE (EAST ABUTMENT)
FOUNDATION KEY PLAN

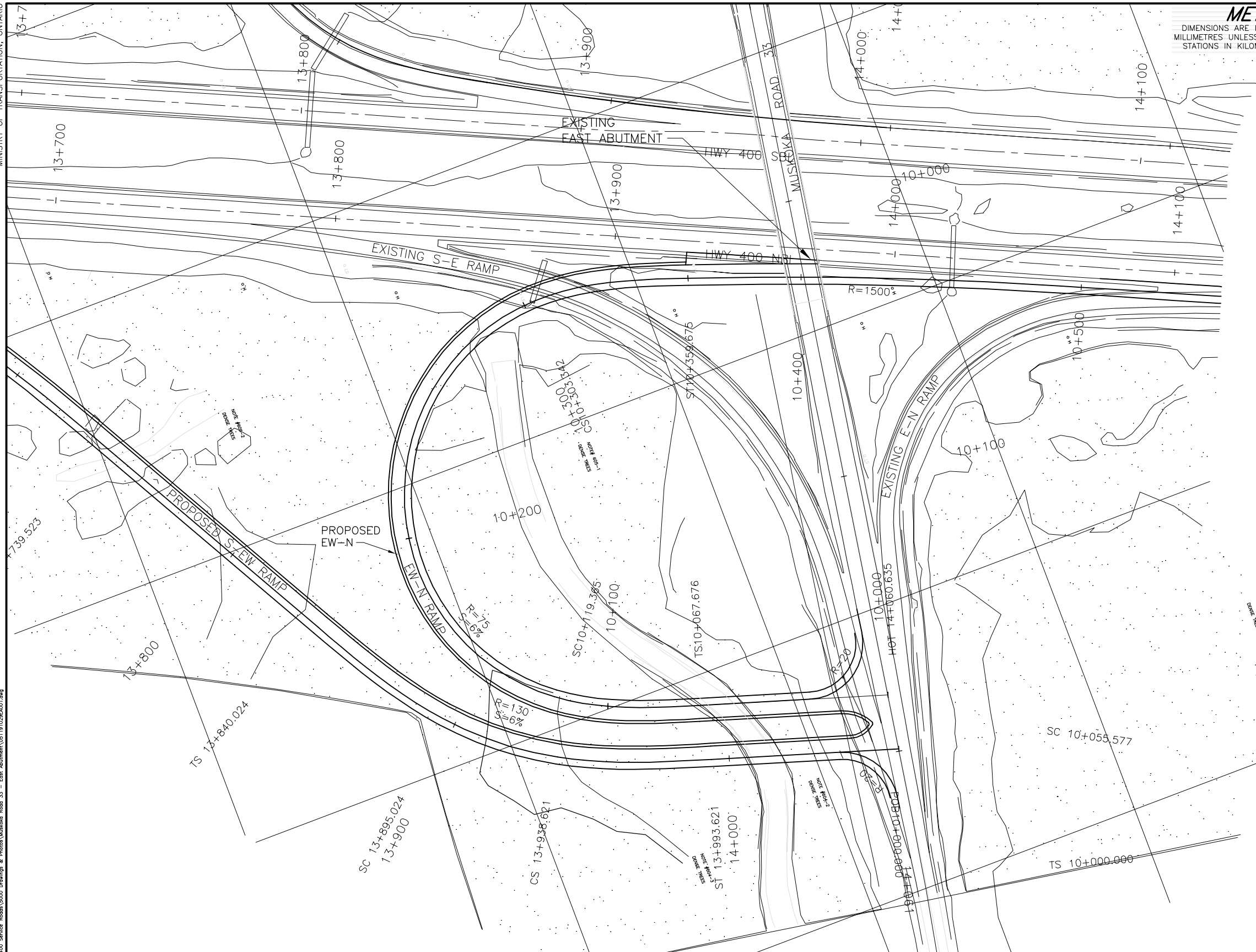


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MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.5 0 1.5 3 km



PLAN

SCALE

15 0 15 30 m

NOTES

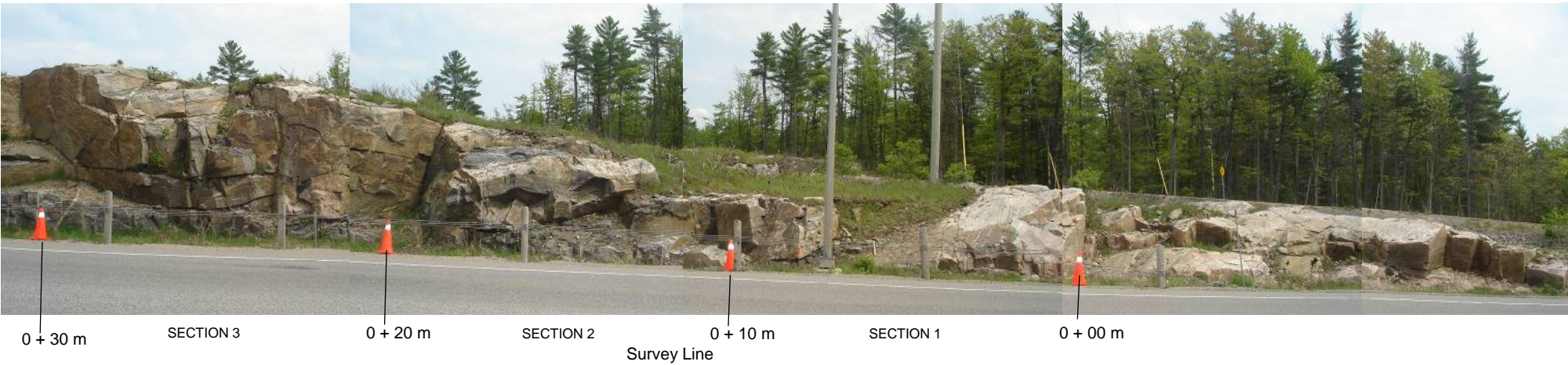
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REFERENCE

Base plan provided in digital format by URS, drawing file nc6-13+450_10+550.dwg, received July, 2006.

NO.	DATE		BY	REVISION					
Geocres No.									
HWY. 400				PROJECT NO. 05-1191-029				DIST.	
SUBM'D. AB		CHKD. AB		DATE: OCT 2006				SITE:	
DRAWN: JFC		CHKD. SEP		APPD. JMAC				DWG. E-1	

A) View from Highway 400 North Bound Lane – Looking East



B) Profile Looking North

C) Geometric Data



SECTION	SURVEY LINE	CLEARANCE ZONE WIDTH (CZW) (m)	HEIGHT (H) (m)
1	0+05	6.9	1.7
2	0+15	7.0	3.2
3	0+25	6.9	4.0
4	0+35	7.0	5.0
5	0+45	6.9	2.5

Note: Images and orientations are skewed due to varying camera angles.

A) Highway 400 North Bound – Looking East



C) Highway 400 North Bound – East Abutment



B) Highway 400 North Bound – Looking North



D) Highway 400 North Bound – Looking East at a saw-toothed wedge assumed to have either fallen out or was pulled out and removed during the original excavation.



E) Highway 400 North Bound – Looking South.



A) View from East Abutment – Facing South to West



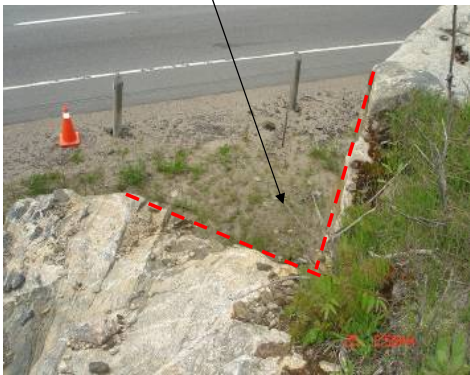
Note: Images and orientations are skewed due to varying camera angles.

- Observable joint or vein discontinuities. Mainly dipping sub-vertical.

B) East Abutment – Facing North



C) Loss of rock wedge, likely during blasting.



D) Directly Adjacent to Bridge Abutment



E) Under Bridge at East Abutment

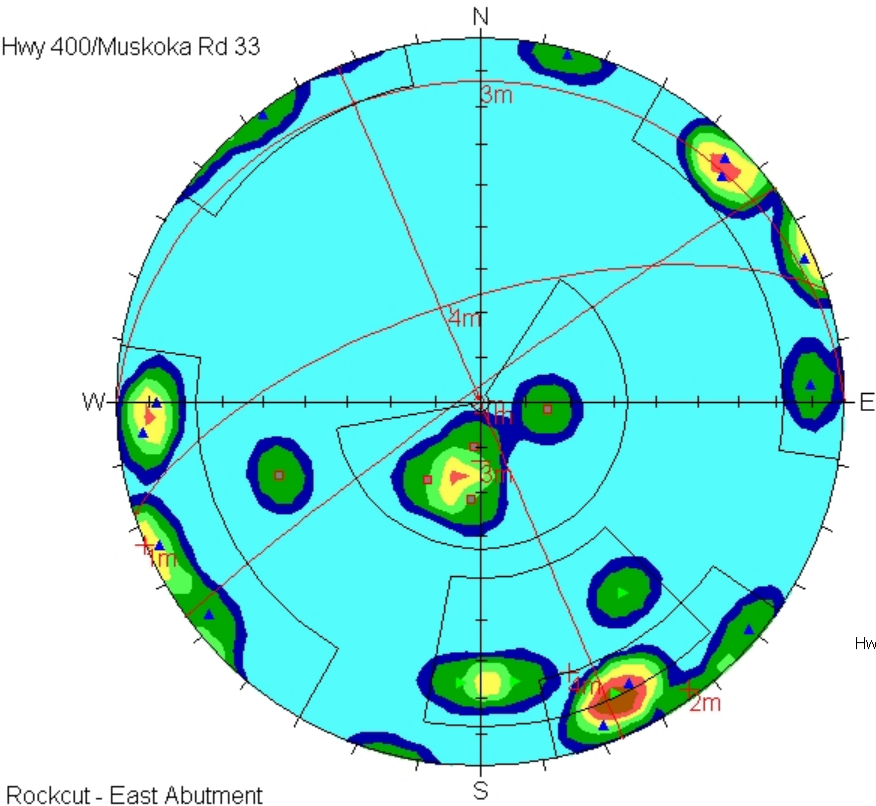


G.W.P. 5622-02-00

HIGHWAY 400 / MUSKOKA ROAD 33
East Abutment Rock Cut Discontinuity Mapping
Stereographical Plots

Figure E4

1. Discontinuity pole plot showing concentration contours and joint sets.



TYPE

- FO [5] (Foliation)
- JN [13] (Joints)
- VN [4] (Veins/Dykes)

2. Major planes plot showing dip and dip direction of major and minor joint sets.



Orientations
ID Dip / Direction

- 1 m 90 / 087 (Joint 1)
- 2 m 87 / 324 (Joint 2)
- 3 m 13 / 360 (Foliation 1)
- 4 m 67 / 342 (Vein 1)

Equal Area
Lower Hemisphere
22 Poles
22 Entries

G.W.P. 5622-02-00

HIGHWAY 400 / MUSKOKA ROAD 33
East Abutment Rock Cut Discontinuity Mapping
Kinematic Controls

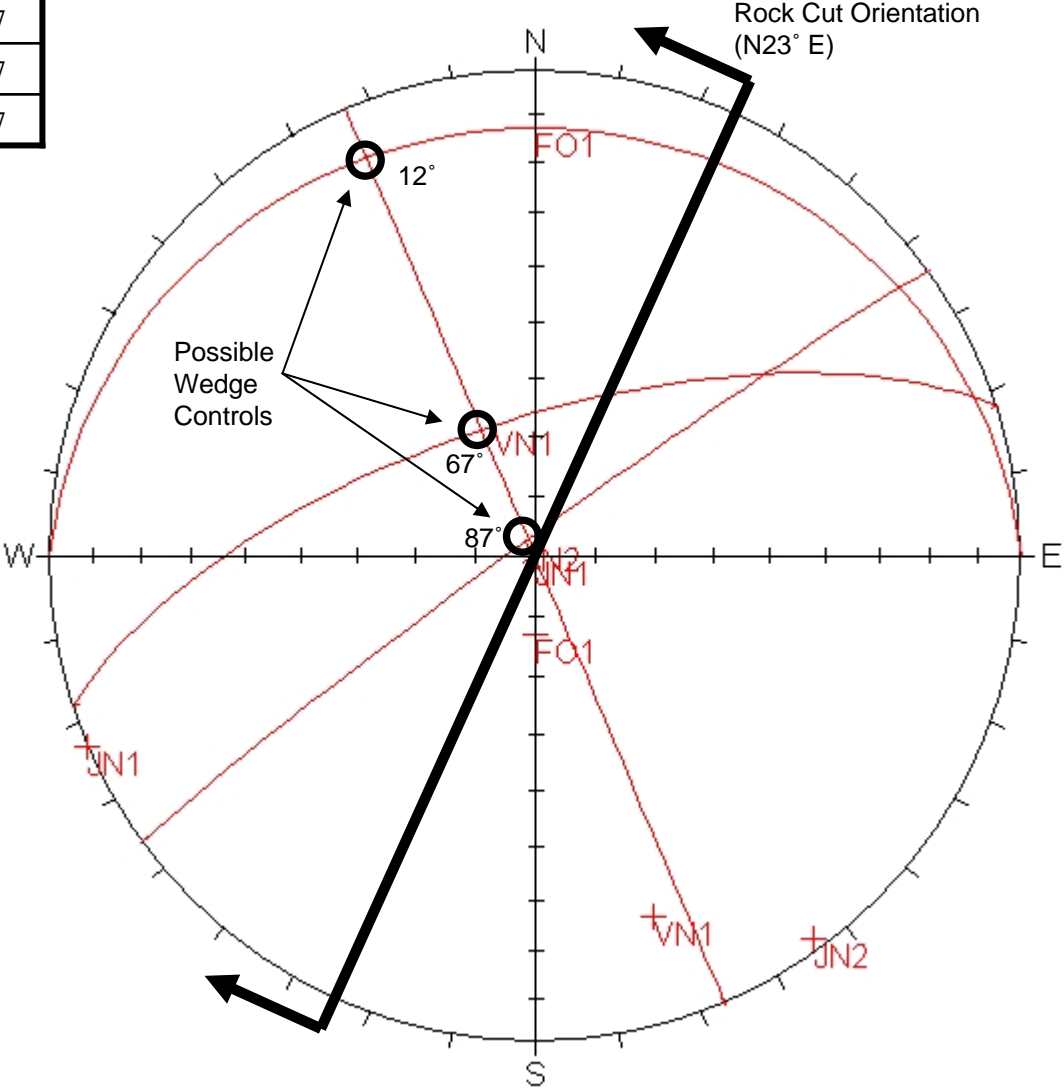
Figure E5

Wedge Controls

Set I	Set II	Plunge	Trend
1	2	87	337
1	4	67	337
1	3	12	337

Wedge controls between FO1 and JN1 only concern if FO1 dips greater than approximately 30°.

Set#	Dip	Dip Dir
1:JN1	90	067
2:JN2	87	324
3:FO1	13	360
4:VN	67	342



Planar Controls

None

Toppling Controls

None

APPENDIX F

**NON STANDARD SPECIAL PROVISIONS (NSSPs)
AND OPERATIONAL CONSTRAINTS (OCs)**

ROCK DOWELS - Item No.

Non-Standard Special Provision

Scope

Work under this item is for the installation of rock dowels for the stabilization of rock cut faces.

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of two (2) years experience in the field of installation of rock dowels or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents.

Construction

Install fully cement grouted, hot-dip galvanized deformed rock dowels at locations shown in the contract package and as directed by the Contract Administrator. The rock dowels are to consist of 5.0 m long 25 mm diameter (minimum) deformed rebar grade (minimum yield strength 400 MPa) bars as shown on contract drawings. Dowel locations shall be field verified and adjusted by the QVE once the rock outcrop is cleaned and inspected.

Drill the rock dowel holes at the diameter and inclination shown on the drawings or as otherwise directed by the Contract Administrator. Drill to a measured depth so that when bolts are fully inserted in the completed drill hole they are flush with the rock or shotcrete surface. Clean the holes using compressed air from the drill or a compressed air blowpipe, min. 500 cfm.

Cement grout for dowels shall be pre-mixed, non-metallic shrinkage compensating grout placed according to the manufacturer's specifications. Water for use in grout mixes shall be clean and free of deleterious substances. The water shall be filtered if necessary to reduce the suspended solids to less than 500 mg/litre.

All dowels shall be fully cement grouted using a water to cement ratio of 0.35 to 0.40.

Dowels shall be installed and grouted at least 48 hours prior to any rock excavation or rock blasting in any areas within 10 m of the area where the dowels are to be installed.

Corrosion protection (approved by the Contract Administrator) is to be applied to all exposed surfaces not already protected.

Each rock dowel not fully grouted (drill hole not completely filled with hardened grout) or which protrudes by an amount greater or less than the specified amount is to be replaced at the Contractor's expense by another installed alongside.

The Contractor shall submit the following information at least 2 weeks prior to doing the work under this item for approval by the Contract Administrator: rock dowel and grout supplier; type of grout, drill hole diameter and installation methodology.

All material resulting from the operation shall be managed and disposed of in accordance with OPSS 180 as specified elsewhere in the contract.

All costs associated with the management and disposal of materials are deemed to be included in the contract unit price.

Measurement for Payment

Measurement shall be for each rock dowel installed as specified.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all rock dowels, fittings, grout, corrosion protection and other materials, provision of cranes, lift equipment, scaffolding and other means of access, and labour and materials for drilling and installation.

NON-EXPLOSIVE ROCK EXCAVATION - Item No.

Non-Standard Special Provision

Scope

The new structure foundations are in close proximity to the existing Muskoka Road 33 structure east abutment footings. The contractor shall use non-explosive rock excavation techniques within the limits specified on the Contract Drawings.

The Contractor shall submit their procedures for non-explosive rock excavation to the Contract Administrator a minimum of 2 weeks prior to commencing with the work.

The Contractor shall install rock dowels and perform vibration monitoring as shown elsewhere in the Contract Documents.

Basis of Payment

Payment at the contract price shall include all labour, equipment and materials to carry out the above work.

Vibration Monitoring During Rock Excavation at Muskoka Road 33 East Abutment

The following sections describe the vibration monitoring required for the rock excavation at the Muskoka Road 33 East Abutment.

The vibration monitoring equipment shall be placed directly on the concrete foundation of the existing east bridge abutment such that it will not be disturbed. The location shall be as close as possible to the construction activities such that the existing structure is not disturbed.

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

The Contractor shall take readings during construction activities adjacent to the existing bridge including and not limited to blasting of rock for new EW-N Ramp outside the zone requiring non-explosive methods but within 50 m of the structure and rock excavation (non-explosive methods) in the immediate vicinity of the structure.

The construction works shall be conducted such that the activities progress from the point furthest away from the existing structure working towards the existing structure. The vibration monitoring should be carried out on a continuous basis during rock excavation activities.

The results shall be submitted to the Contract Administrator no less than two times per day during such activities – at mid-day and at the end of each day – and/or immediately after a blasting event. Additional submissions may be required at the discretion of the Contract Administrator. The results shall be immediately reviewed by the QVE and submitted to the Contract Administrator prior to the Contractor continuing with the construction activity. As a minimum, the type of construction activity, location and distance away from the existing bridge abutment must be submitted with vibration monitoring results.

If the results are acceptable, the Contractor may continue with the work and continue to take additional readings.

If the readings are not within the limits stated above, the Contractor must alter his construction procedures until the vibrations on the existing structure are within acceptable levels.

Bases of Payment

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

Swamp Excavation at Muskoka Road 33 S-EW Ramp Station 13+340 to 13+465

This special provision outlines the procedure to be used for excavation of the organics and clay deposits between Highway 400 Station 13+340 and 13+400 and between Station 13+410 and 13+465.

Removal of the organics/clay shall be in accordance with SP203.02 except as noted herein.

Removal of the organics/clay shall be carried out in short sections perpendicular to the highway alignment with the base of the excavation/trench not wider than 3 m at any time.

The crest of the excavation shall start no closer than 2 m from the existing edge of pavement.

Temporary excavation side slopes through the organics/clay shall be no steeper than 3H:1V.

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

The Contractor shall maintain the operation of the existing highway during excavation and backfilling operations including and not limited to traffic control, regrading and asphalt padding.

Basis of Payment

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