March 14, 2012

# FOUNDATION INVESTIGATION AND DESIGN REPORT

## HIGHWAY 529 OVERPASS NBL STRUCTURE
## HIGHWAY 69 FOUR-LANING FROM 0.4 KM NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD) NORTHERLY 11 KM
## MINISTRY OF TRANSPORTATION, ONTARIO
## GWP 5005-08-00, WP 5191-06-01

**Submitted to:**
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**GEOCRES No.: 41H-85**

REPORT

**Report Number:** 07-1191-0020-B4

3 Copies Ministry of Transportation, Ontario, North Bay, Ontario (Northeastern Region)
1 Copy Ministry of Transportation, Ontario, Downsview, Ontario (Foundations Section)
2 Copies MMM Group, Thornhill, Ontario
2 Copies Golder Associates Ltd., Sudbury, Ontario

Golder Associates

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## FOUNDATION INVESTIGATION REPORT
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# 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group (MMM) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Highway 69 Northbound Lane (NBL) structure crossing the future Highway 529 (i.e. Highway 529 Overpass NBL). This project is part of the detail design for the four-laning of Highway 69 from 0.4 km north of Highway 7182 (Shebeshekong Road) northerly for 11 km. The general location of this section of the Highway 69 four-laning alignment is shown on the Key Plan on Drawing 1, following the text of this report.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated March 28, 2007. Golder's proposal (P7-1191-0020, dated April 24, 2007) for foundation engineering services associated with the new NBL structure is contained in Section 6.8 of MMM's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5006-E-0031) for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 2007. The General Arrangement (GA) Drawing for the proposed Highway 529 Overpass NBL structure was provided to Golder by MMM.

This report addresses the investigation carried out for the Highway 529 Overpass NBL structure, the associated approach embankments and the Retained Soil System (RSS) walls. Separate reports will be submitted detailing the foundation investigations for the SBL structure and other bridge structures, a pond crossing and culverts for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure location, including the associated approach embankments and RSS walls, by borehole drilling, rock coring and laboratory testing on selected soil and bedrock core samples. The investigated areas are shown on Drawing 1, following the text of this report.

# 2.0 SITE DESCRIPTION

The proposed Highway 529 Overpass NBL structure is a 99.1 m long 3-span structure, located in the Township of Harrison, about 800 m south of South Shore Road. The proposed grade at the new Highway 69 NBL south and north approach embankments will be at about Elevation 200 m and 201.4 m, respectively, which is up to about 7 m above the existing ground surface at the south approach and up to about 4 m below the existing ground surface at the north approach.

In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps. The existing Highway 69 (future Highway 529) at the crossing is located in a bedrock "cut". The topography at the south approach area is generally flat, low-lying with tree cover. The ground surface at the borehole locations within the limits of the proposed structure and approach embankment areas ranges between Elevation 190.2 m and 205.2 m at the south and north ends of the site, respectively.

# 3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation at the proposed structure was carried out between January 29 and April 15, 2009, and on April 7, 2010, during which time a total of twenty (20) boreholes (designated as Boreholes B4-1 to B4-6 and B4-10 to B4-23) were advanced at the locations shown on Drawing 1, following the text of this report. The locations of the boreholes are generally summarized as follows:

- Five boreholes were advanced for each of the south and north abutments and north pier;
- Two boreholes were advanced for the south pier;
- Two boreholes were advanced at the approach embankments (i.e. one borehole at each approach); and
- One borehole was advanced near the south end of the proposed RSS wall on the east side of the south abutment.

The boreholes were advanced using a track- or truck-mounted CME-55 supplied and operated by Landcore Drilling Ltd. (Landcore) of Sudbury, Ontario. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers or NW casing and wash boring. Soil samples were obtained, where possible, continuously or at intervals of depths of 0.75 m to 2 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Rock core samples were obtained using an 'NQ' size core barrel. The groundwater conditions in the open boreholes were observed during the drilling operations. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation (O.Reg.) 903 (as amended by O.Reg. 372).

Bedrock was observed at ground surface in Boreholes B4-11, B4-12, B4-16, B4-17, B4-19, B4-20 and B4-22. The remaining boreholes were advanced to auger/split-spoon refusal or cored into the bedrock to depths ranging from 1.6 m to 14.6 m below existing ground surface. This included coring bedrock for lengths of between 2.9 m and 3.2 m in Boreholes B4-2, B4-3, B4-4, B4-10, B4-13 and B4-14 and 10.5 m in Borehole B4-18.

A piezometer was installed in Borehole B4-10 to permit monitoring of the groundwater level at this location. The piezometers consist of 51 mm diameter PVC pipe, with a 1.5 m long slotted screen sealed at a selected depth within the borehole. The non-instrumented borehole and the annulus surrounding the piezometer pipe above the sand pack were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A.

The fieldwork was supervised throughout by members of our engineering and technical staff, who located the boreholes based on the survey carried out by MMM, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Strength testing (uniaxial compression and point load index) was also carried out on selected specimens of the rock core.

MMM surveyed the location of the boreholes at the site in December 2008 prior to drilling, excluding Borehole B4-23, which was referenced to the staked highway alignment. Where boreholes were relocated from the original staked locations, Golder resurveyed and located the new boreholes relative to MMM's stakes. The borehole locations shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole/drillhole locations and ground surface elevations are as follows:

| Borehole | Location (m) | | Ground Surface Elevation (m) | Borehole Depth (m) |
|---|---|---|---|---|
| | Northing | Easting | | |
| B4-1 | 5050316.5 | 237115.6 | 191.8 | 4.2 |
| B4-2 | 5050321.4 | 237114.8 | 190.7 | 7.1 |
| B4-3 | 5050318.1 | 237110.3 | 192.3 | 8.5 |
| B4-4 | 5050314.8 | 237105.8 | 191.9 | 5.8 |
| B4-5 | 5050319.8 | 237104.9 | 192.1 | 4.1 |
| B4-6 | 5050342.9 | 237109.9 | 193.2 | 9.9 |
| B4-10 | 5050345.5 | 237098.3 | 190.2 | 14.6 |
| B4-11 | 5050387.2 | 237101.2 | 197.6 | 0 |
| B4-12 | 5050392.1 | 237100.4 | 199.3 | 0 |
| B4-13 | 5050388.8 | 237095.8 | 192.2 | 4.1 |
| B4-14 | 5050385.5 | 237091.3 | 192.8 | 5.7 |
| B4-15 | 5050390.4 | 237090.5 | 192.7 | 1.6 |
| B4-16 | 5050413.8 | 237096.6 | 203.8 | 0 |
| B4-17 | 5050418.7 | 237095.7 | 205.0 | 0 |
| B4-18 | 5050415.4 | 237091.2 | 203.9 | 10.7 |
| B4-19 | 5050412.1 | 237086.7 | 202.9 | 0 |
| B4-20 | 5050417.0 | 237085.9 | 203.2 | 0 |
| B4-21 | 5050303.3 | 237112.8 | 192.0 | 3.2 |
| B4-22 | 5050430.2 | 237088.7 | 205.2 | 0 |
| B4-23 | 5050293.9 | 237135.8 | 193.5 | 2.3 |

# 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

## 4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)[1], this section of Highway 69 lies within the physiographic region known as the Georgian Bay Fringe which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

This part of the Georgian Bay Fringe physiographic region was never submerged during periods of glacial recession. As a result, the surficial soils in this area consist of very shallow deposits of sand, silt and clay overlying metamorphic bedrock and numerous bare knobs and ridges of bedrock are present throughout the area. Localized low-lying swampy areas, containing peat and/or organic soils overlying soft/loose native soils, are present in valleys between the bedrock knobs and ridges.

The bedrock in the area consists typically of gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in Geology of Ontario, OGS Special Volume 4 (OGS, 1991)[2]. Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

## 4.2 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 and bedrock samples, are presented on the Record of Borehole and Drillhole sheets in Appendix A. The results of the laboratory tests carried out on selected soil and bedrock samples are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and the results of SPT measurements. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond the boreholes will exist and is to be expected.

In general, the subsoils in the area of the south approach/abutment/pier consist of organics/peat or fill materials underlain by cohesive deposits of silty clay to clay and/or cohesionless deposits of gravelly sand to sand and silt. Bedrock is exposed (i.e. along the existing Highway 69 rock cut) at the north approach/abutment and in the east half of the north pier. The total thickness of overburden is variable at the site, ranging from about 11.5 m in the south pier area to no overburden at the north abutment/approach where bedrock is exposed.

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

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

[2] Geology of Ontario, 1991. Ontario Geological Society Volume 4, Part 2. Ministry of Northern Development and Mines, Ontario.

### 4.2.1 Fill

Fill was encountered at ground surface in Boreholes B4-1 to B4-3, B4-6, B4-10, B4-13 to B4-15, B4-21 and B4-23. Boreholes B4-1 and B4-3 were advanced on the west side of the existing SBL ditch, Boreholes B4-2 and B4-10 were advanced in the existing SBL ditch and Boreholes B4-6 and B4-13 to B4-15 were advanced through the existing roadway. Borehole B4-23 advanced on the west side of the existing highway south of the south abutment.

#### *4.2.1.1 Roadway Fill*

In Boreholes B4-6, B4-14 and B4-15 (drilled from the roadway surface), between approximately 75 mm and 210 mm of asphalt was encountered. In Borehole B4-10, a 0.1 m thick layer of peat fill was encountered at ground surface. Underlying the asphalt and peat fill and from ground surface at Boreholes B4-13 and B4-23, the boreholes penetrated roadway fill which generally consists of sand to sand and gravel, trace silt. The ground surface/top of fill in Boreholes B4-6, B4-10, B4-13 to B4-15 and B4-23 ranges between Elevation 193.5 m and 190.2 m and the thickness of the embankment fill ranges from 0.2 m to 4.5 m.

SPT ‘N’-values measured within the sand to sand and gravel fill range between 0 blows (weight of hammer) and 98 blows per 0.3 m of penetration indicating a very loose to very dense relative density, but are typically below 25 blows per 0.3 m of penetration, indicating a generally very loose to compact relative density.

The natural moisture content measured on samples of the fill layer ranged between about 15 percent and about 22 percent.

Grain size distribution tests were carried out on three samples of the sand fill and the results are shown on Figure B-1.

#### *4.2.1.2 Organics Fill*

In Boreholes B4-1, B4-2 and B4-3, fill consisting of peat containing sand, silt and clay or clayey silt to sand containing organics was encountered at ground surface. The ground surface/top of fill in these boreholes ranges from Elevation 192.3 m to 190.7 m and the thickness of the organic fill layer ranges between 0.6 m and 1.8 m.

SPT ‘N’-values measured within the peat fill layer range between 2 and 5 blows per 0.3 m of penetration suggesting a soft to firm consistency.

The natural moisture content measured on samples of the peat fill layer ranges between about 27 percent and about 58 percent.

### 4.2.2 Peat

A deposit of moist to wet, brown to black peat was encountered at ground surface or below the fill in Boreholes B4-3 to B4-5, B4-18 and B4-21. The top of the peat layer was encountered between Elevation 190.8 m and 192.1 m in the south approach area and Elevation 203.9 m in the north approach area and the thickness of the deposit ranges between 0.1 m and 2.2 m.

SPT 'N'-values measured within the peat deposit range between 1 and 4 blows per 0.3 m of penetration suggesting a very soft to soft consistency.

The natural moisture content measured on samples of this deposit ranges between about 24 percent and about 361 percent.

### 4.2.3 Silty Clay to Clay

A deposit of wet, brown to grey, silty clay to clay containing trace to some sand was encountered below the fill in Boreholes B4-1, B4-2, B4-6 and B4-10 and below the peat in Boreholes B4-3 to B4-5 and B4-21. In Borehole B4-21, a 0.3 m thick clayey silt seam was encountered at the top of the silty clay to clay deposit at a depth of 1.0 m (Elevation 191.0 m). The top of the silty clay to clay deposit was encountered between Elevation 186.8 m and 191.1 m and its thickness ranges between 1.2 m and 3.3 m. In Boreholes B4-2 and B4-4, the bottom of this deposit was confirmed by coring of the underlying bedrock and in Borehole B4-5 the bottom of this deposit was defined by refusal to further split-spoon and auger advancement.

SPT 'N'-values measured within the clayey silt to clay deposit range from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 23 kPa to 48 kPa where the cohesive deposit was encountered below the fill (i.e. Boreholes B4-6 and B4-10) and ranging from 6 kPa to 18 kPa where fill was not generally encountered. The SPT 'N'-values together with the in situ field vane tests indicate the deposit has a very soft to firm consistency.

The natural moisture content measured on samples of this deposit ranges between about 40 percent and about 89 percent.

Atterberg limits testing carried out on eight (8) samples of the silty clay to clay deposit yielded liquid limits ranging from 49 percent to 60 percent, plastic limits ranging from 21 percent to 27 percent and plasticity indices ranging from 29 percent to 36 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-2 in Appendix B and indicate that the deposit consists of silty clay of medium plasticity to clay of high plasticity. An Atterberg limits test was also carried out on the clayey silt seam and yielded a liquid limit of 31 percent, a plastic limit of 18 percent and a plasticity index of 13 percent.

A grain size distribution test was carried out on one sample of the cohesive deposit and the results are shown on Figure B-3.

### 4.2.4 Gravelly Sand to Sand and Silt

A deposit of wet, brown to grey, gravelly sand to sand and silt containing trace to some clay was encountered below the silty clay to clay deposit in Boreholes B4-1, B4-3, B4-6, B4-10 and B4-21 and below the fill in Borehole B4-23. The top of the deposit was encountered between Elevation 189.3 m and 185.6 m in each of the boreholes except Borehole B4-23 where it was encountered at Elevation 193.3 m. The thickness of the deposit ranges from 0.2 m to 6.9 m. The bottom of this deposit was defined by either bedrock coring or refusal.

SPT 'N'-values measured within this deposit range from 2 blows to 51 blows per 0.3 m of penetration, indicating a very loose to very dense relative density.

The natural moisture content measured on samples of this deposit ranges between about 10 percent and about 21 percent.

Grain size distribution tests were carried out on four samples of the sand to sand and silt deposit and the results are shown on Figure B-4.

### 4.2.5 Refusal/Bedrock

Bedrock was encountered and cored in Boreholes B4-2 to B4-4, B4-10, B4-13, B4-14 and B4-18. Bedrock was also exposed in Boreholes B4-11, B4-12, B4-16, B4-17, B4-19, B4-20 and B4-22. The bedrock surface was inferred from spilt-spoon and/or auger refusal in the remaining holes. The bedrock surface (inferred or actual) was encountered at depths that ranged from 0.2 m to 11.5 m below ground surface, and ranges from Elevation 205.2 m to 178.7 m. Refusal and bedrock surface depths and elevations as encountered in the boreholes are summarized in Table B-1.

Based on a review of the bedrock core samples, the bedrock at the site consists of gneiss and the core samples are described as grey to pinkish grey, fine to coarse grained and fresh to slightly weathered. In Borehole B4-10, the lower 1.5 m was heavily fractured with zones of broken core. In Borehole B4-18, the bedrock below a depth of 8.4 m (Elevation 195.5 m) is described as pegmatite.

The Total Core Recovery (TCR) is 100 percent for all core samples. The Rock Quality Designation (RQD) measured on the core samples ranges from about 70 percent to 100 percent, indicating a rock mass of fair to excellent quality. The lower core sample in Borehole B4-10 measured an RQD value of about 38 percent, indicating a rock mass of poor quality. The Solid Core Recovery (SCR) ranges from about 66 percent to 100 percent. The lower core sample in Borehole B4-10 measured a SCR of 38 percent.

Laboratory Uniaxial Compressive Strength (UCS) testing was carried out on six core samples of the bedrock. The UCS ranges from about 83 MPa to 154 MPa for the gneiss bedrock as summarized in Table B-2, indicating strong to very strong rock.

Point load strength tests were performed on selected samples of the bedrock. Diametral point load strength index values are shown on the Record of Drillhole sheets and are summarized in Table B-3. The diametral point load index ($I_S50$) results from the laboratory tests carried out on core samples range from about 3 MPa to 6.5 MPa. These index values correspond to estimated UCS values ranging between 71 MPa and 150 MPa, as presented in Table B-3, based on a relationship between $I_S50$ and UCS which is given by a correlation factor (K) in accordance with ASTM 5731-08, which varies depending on the size of the core samples and the strength of the rock. For this site, these UCS values are based on an estimated average correlation factor (K) of 23, which was calculated based on a comparison of the UCS test results and the point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UCS tests. Based on the laboratory UCS tests and the point load strength test results, in accordance with Table 3.5 in CFEM (2006), the gneiss bedrock is classified as strong (R4, 50 MPa < UCS <100 MPa) to very strong (R5, 100 MPa < UCS <250 MPa).

### 4.2.6 Groundwater Conditions

The water levels were noted in the boreholes immediately after the drilling operations. In general, the soil samples taken in the boreholes were noted to be moist to wet. Where bedrock either was exposed, or was encountered at shallow depth below ground surface, the open boreholes were dry. The water level measured in Boreholes B4-1 to B4-6, B4-14, B4-21 and B4-23 upon completion of drilling was at depths that ranged between 0.5 m and 2.3 m below ground surface, ranging between Elevation 191.3 m and 190.3 m.

A standpipe piezometer was installed in Borehole B4-10 to permit monitoring of the water level within the cohesionless stratum at this location. Details of the piezometer installation are shown on the Record of Borehole sheets in Appendix A. The groundwater level measured in the piezometer installation was 0.7 m above ground surface as summarized below.

| Foundation Element | Borehole No. | Ground Surface Elevation (m) | Groundwater Elevation (m) | Date of Measurement |
|---|---|---|---|---|
| South Pier | B4-10 | 190.2 | 190.9 | March 31, 2009 |

It should be noted that groundwater levels in the area are subject to seasonal fluctuations and precipitation events.

## 5.0 CLOSURE

The field personnel supervising the drilling program were Mr. Ed Savard and Mr. Indulis Dumpis. This report was prepared by Mr. Evan Childerhose, P.Eng. and Mr. André Bom, P.Eng. The technical aspects were reviewed by Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project, who also carried out a quality control review of the report.

# Report Signature Page

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# PART B

## FOUNDATION DESIGN REPORT
## HIGHWAY 529 OVERPASS NBL STRUCTURE
## HIGHWAY 69 FOUR-LANING FROM 0.4 KM
## NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD)
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# 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Highway 69 NBL structure crossing over Highway 529. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations, associated RSS walls and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

## 6.1 General

The new Highway 529 Overpass NBL structure will be a three-span structure, comprised of a 45.1 m long centre span and 27 m long south and north spans. The new structure crosses the existing Highway 69 at a skew of about 25 degrees. Based on the General Arrangement (GA) drawing provided by MMM, the proposed Highway 69 NBL grade is at Elevation 199.9 m and 201.4 m at the south and north abutments, respectively. The proposed south approach embankment of the abutment will be up to about 7.5 m high while a rock cut of about 4 m below existing ground surface will be required at the north abutment. The future Highway 529 (i.e. existing Highway 69) is in an existing rock cut and the grade at the NBL crossing location is about Elevation 193 m, decreasing towards the north. Retained Soil System (RSS) walls are proposed for the east side of the south abutment and the west side of the north abutment as there is insufficient lateral clearance between the side/toes of the new abutment slopes to the existing/new embankments/structures.

At the south approach (i.e. from the south abutment to 20 m southerly), the ground surface is generally flat, at about Elevation 192 m. The thickness of overburden is between 2.9 m and 5.3 m and generally consists of up to 2.2 m of peat fill and/or native peat, underlain by up to 3.3 m of silty clay to clay and/or up to 0.5 m of gravelly sand to sand and silt, over bedrock. The bedrock surface (actual or inferred) was encountered between Elevation 186.8 m and 189.0 m.

The south pier is located on the west side of the existing Highway 69 with the ground surface in the vicinity varying between about Elevation 190 m and 193 m. The thickness of overburden is between 9.9 m and 11.5 m and consists of granular embankment fill up to 4.5 m thick, underlain by a deposit of clay up to 2.7 m thick, underlain by an up to 6.2 m thick deposit of sand to sand and silt. The bedrock surface (actual or inferred) was encountered between Elevation 178.7 m and 183.3 m.

The north pier is located on the east side of the existing Highway 69 with the west portion of the pier located in the NBL ditch and the east portion of the pier located in the rock cut. The ground surface varies between about Elevation 192 m and 199 m and the overburden is comprised of up to 2.6 m of granular embankment fill in the area of the existing highway. The bedrock surface ranges between Elevation 190.2 m along the NBL ditch and Elevation 199.3 m on the bedrock outcrop.

The north approach (i.e. from the north abutment to 20 m northerly) is located on a rock cut covered with up to 0.2 m of organic material. The bedrock surface ranges from Elevation 202.9 m to 205.2 m, sloping upwards from south (at the abutment) to north (along the approach embankment).

We understand that a detour will be constructed to the west of the existing highway prior to the construction of the Highway 529 Overpass NBL and SBL structures in this area. The two structures will then be constructed after traffic is routed onto the detour.

Golder carried out a rock fall hazard assessment of the existing rock cut on the east side of the existing highway in this area. The results of the assessment and recommendations for mitigation are provided in a separate report (Golder Report No. 09-1117-0022, dated May 28, 2010).

## 6.2 Bridge Foundation Options

At the proposed south abutment, the bedrock surface is present at a depth between 2.9 m and 5.3 m below ground surface. For the south abutment, feasible foundation alternatives include:

- Steel H-piles driven to the bedrock surface or socketted into bedrock, if required, to achieve the minimum pile length;
- A spread footing founded on bedrock; or
- Caissons socketted into the bedrock.

As the depth to bedrock at the proposed south abutment is variable between 3.2 m and 5.3 m below ground surface and up to about 4 m below the groundwater level at the proposed south abutment, it may not be feasible at this location to properly dewater the excavation for construction of a spread footing in-the-dry. Therefore, we recommend that the south abutment be supported on steel H-piles driven to or socketted into bedrock. The advantages, disadvantages, relative costs and risks/consequences for the foundation alternatives for the south abutment are summarized in Table 1.

At the proposed south pier, shallow foundations are not considered feasible due to the thickness (up to 11.5 m) and very loose relative density and very soft to firm consistency of the native soils overlying bedrock at this location. Further, due to the presence of the shallow groundwater level at Elevation 190.9 m (i.e. 0.7 m below ground surface as measured in the piezometer in Borehole B4-10 in March 2009) dewatering would be required to be able to properly place and compact an engineered fill pad if the silty clay deposit was excavated to Elevation 185.6 m. In addition, differential settlement would likely develop between the south pier and the other foundation elements if the south pier was founded on a granular pad constructed on the native cohesionless soil. Deep foundations consisting of steel H-piles driven to bedrock or caissons socketted into bedrock are both technically feasible alternatives. Steel H-piles are recommended as the preferred foundation alternative for the south pier, based on a comparison of the advantages/disadvantages, relative costs and risk/consequences, as summarized in Table 2.

At the proposed north pier and north abutment where bedrock is present at ground surface, we recommend founding the pier and abutment on spread footings constructed on bedrock. Bedrock excavation will be required for the east half of the north pier and for the north abutment to reach the founding elevation.

The following sections provide further discussion and design recommendations for shallow and deep foundations as discussed above.

## 6.3 Shallow Foundations

We recommend supporting the north abutment and north pier on spread footings placed directly on properly prepared gneiss bedrock. Consideration could also be given to supporting the south abutment on spread footings placed directly on the bedrock surface, however, as discussed in Section 6.2, dewatering will be required to sub-excavate the overburden and place concrete in-the-dry and dewatering may not be feasible at this location due to the variable bedrock surface. The details of the bedrock/refusal surface elevation and the recommended underside of footing elevation at the foundation elements are summarized below.

| Foundation Element | Location within Foundation Element | Relevant Borehole Numbers | Bedrock Surface/Refusal Elevation (m) | Recommended Underside of Footing Elevation (m) |
|---|---|---|---|---|
| North Abutment | East | B4-16 and B4-17 | 205.0 to 203.8 | At/Below 202.9 (exposed bedrock) |
| | Centre | B4-18 | 203.7 | |
| | West | B4-19 and B4-20 | 203.2 to 202.9 | |
| North Pier | East | B4-11 and B4-12 | 199.3 to 197.6 | 191.3 |
| | Centre | B4-13 | 191.3 | |
| | West | B4-14 and B4-15 | 191.1 to 190.2 | |
| South Abutment | East | B4-1 and B4-2 | 187.6 to 186.8 | 189.0 |
| | Centre | B4-3 | 187.0 | |
| | West | B4-4 and B4-5 | 189.0 to 188.0 | |

At the north abutment and east side of the north pier, several metres of bedrock excavation will be required to reach the underside of footing due to the height of the rock cut and the proposed grade lowering at these locations. The bedrock at the founding depth will be of good quality provided that the founding surface is properly prepared using controlled rock excavation/blasting techniques. Recommendations on bedrock excavation are provided in Section 6.10.1.

If a shallow foundation is adopted at the south abutment, it should be founded on bedrock referenced relative to the higher bedrock surface elevation as presented above for the underside of the footing. At the north pier, the bedrock surface at the centre of the pier is recommended as the reference elevation for the underside of the footing. Mass concrete will be required to raise the founding level to the underside of the footing after removal of the overburden at the south abutment and on the east section of the north pier. Based on the currently proposed underside of footing elevations indicated on the most recent GA, up to 2.2 m and 1.1 m of mass concrete would be required at the south abutment and north pier, respectively. A Non Standard Special Provision (NSSP) for mass concrete should be included in the Contract Documents; an example NSSP is provided in Appendix C.

If higher or lower founding elevations are desired, then mass concrete and/or bedrock excavation will be required accordingly.

### 6.3.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared gneiss bedrock or on mass concrete placed directly on the properly prepared bedrock may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10,000 kPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since bedrock (or mass concrete over bedrock) is considered to be an unyielding foundation and, as such, ULS conditions will govern for this foundation type. It is assumed that the mass concrete will be of compressive strength equal to or greater than the concrete footings (assumed to be 25 MPa or greater).

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with mass concrete. Ontario Provincial Standard Specification (OPSS) 902, Construction Specification for Excavating and Backfilling, should be included in the Contract Documents to address the requirements for construction and inspection of footings on bedrock.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC) and its Commentary.

### 6.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footings/mass concrete and the bedrock should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan $\delta$, may be taken as 0.70 between the base of the concrete footings or mass concrete and the bedrock. This value represents an unfactored value.

For footings on bedrock, the sliding/lateral resistance between the concrete footing/mass concrete and the bedrock may be supplemented by dowelling/anchoring into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels into the rock may be considered in the same way as dowels embedded into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a 1 m minimum embedded length within the bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels; an example is provided in Appendix C.

### 6.3.3 Frost Protection

For spread footings founded directly on the bedrock or mass concrete over bedrock, frost susceptibility is not an issue.

## 6.4 Deep Foundations

### 6.4.1 Steel H-Pile Foundations

We recommend that the south abutment and south pier be founded on steel H-piles driven to refusal on the bedrock surface. The structural designer should determine if the piles have sufficient length for design of the south abutment as socketting into the bedrock may be required to achieve the minimum pile length.

For design, the estimated tip elevations and estimated pile lengths are presented below. There should be a provision made in the Contract for dealing with varying pile lengths due to the variability of the bedrock surface and the lengths given below should be considered minimum lengths.

<table>
<tr><th>Foundation Element</th><th>Location within Foundation Element</th><th>Relevant Borehole Numbers</th><th>Estimated Underside of Pile Cap Elevation* (m)</th><th>Bedrock Surface/Refusal Elevation (m)</th><th>Estimated Tip Elevation** (m)</th><th>Estimated Design Pile Length** (m)</th></tr>
<tr><td rowspan="3">South Abutment</td><td>East</td><td>B4-1 and B4-2</td><td rowspan="3">191.1</td><td>187.6 to 186.8</td><td>186.8</td><td>4.3</td></tr>
<tr><td>Centre</td><td>B4-3</td><td>187.0</td><td>187.0</td><td>4.1</td></tr>
<tr><td>West</td><td>B4-4 and B4-5</td><td>189.0 to 188.0</td><td>188.0</td><td>3.1</td></tr>
<tr><td rowspan="2">South Pier</td><td>East</td><td>B4-6</td><td rowspan="2">190.8</td><td>183.3</td><td rowspan="2">178.7</td><td rowspan="2">12.1</td></tr>
<tr><td>West</td><td>B4-10</td><td>178.7</td></tr>
</table>

Note * Based on information in GA drawing dated July 2011.
** Estimated greatest depth to bedrock and longest pile required.

In order to mitigate potential settlement and stability issues at the south approach embankment, we recommend that the overburden (i.e. fill and underlying peat and clayey silt to clay) be removed and replaced with rock fill (see Section 6.8.3), however, in order to allow for pile driving, the backfill and embankment fill in the immediate abutment (i.e. 'core') area should consist of granular material such as Granular 'B' Type II with a maximum particle size of 75 mm. The compacted granular 'core' should be designed and constructed as described in Section 6.9.

At the south pier, the pile length and the design pile tip elevation assume practically, that the piles do not "hang up" on boulders within the sand to sand and silt deposit before reaching the bedrock surface.

#### *6.4.1.1 Geotechnical Axial Resistance*

For HP310X110 piles driven to refusal on the gneiss bedrock at the south abutment and/or south pier, a factored axial resistance at ULS of 2,000 kN per pile may be used for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

#### *6.4.1.2 Downdrag*

As indicated in Section 6.4.1, we recommend that the compressible clayey silt to clay deposit at the south abutment be sub-excavated in order to mitigate stability and settlement. Therefore, downdrag loads do not need to be considered in the design.

At the south pier, additional fill will not be required above the existing ground surface and downdrag loads do not need to be considered in the design.

#### *6.4.1.3 Pile Driving Notes and Set Criteria*

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should be fitted with rock points such as OPSD 300.201 (Foundation Piles Steel HP310 Oslo Point), Titus Injector or equivalent. For piles driven to bedrock, Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008) should be used on the drawings:

- Piles to be driven to bedrock.

For piles driven to bedrock, set criteria are highly dependent on the type of pile driving hammer and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to improve the process of seating of the piles on a sloping bedrock surface (bedrock sloping downwards towards the west in excess of 45 degrees at the north abutment), also to avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules;
- On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows;

- The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. This procedure is intended to improve the process of seating the pile on the bedrock surface; and
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy.

A NSSP, which indicates that rock points are to be used and outlines the above criteria for seating the piles on bedrock, should be included in the Contract and an example is included in Appendix C.

### 6.4.1.4 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilisation of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The lateral load response of a single vertical pile may be calculated using subgrade reaction theory (*CHBDC* 6.9.2 and C6.8.7, CFEM 1992) where the coefficient of horizontal subgrade reaction, $k_h$(kPa/m), is based on the equations given below for cohesionless soil:

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

where: $n_h$ = the constant of horizontal subgrade reaction (kPa/m)
$z$ = the depth (m)
$B$ = the pile diameter or width (m)

and for cohesive soils:

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

where: $s_u$ = undrained shear strength of the soil (kPa)
$B$ = the pile diameter or width (m)

At the south abutment and south pier, the lateral load response of the piles will be developed from the passive resistance of the soil or backfill. The values of $n_h$ and $s_u$ to be used to calculate the coefficient of horizontal subgrade reaction ($k_h$) to be assumed in the structural analysis for the piles at south abutment and south pier are given below.

| Foundation Element (Relevant Borehole) | Soil Unit | Elevation Section (m) | $n_h$ (kPa/m) | $S_u$ (kPa) |
|---|---|---|---|---|
| South Abutment (B4-1 to B4-5) | Compacted Granular Fill Core | u/s pile cap – 186.8 | 4,400 | - |
| South Pier (B4-6 and B4-10) | Existing Very Loose to Compact Sand Fill | u/s pile cap – 188.7 | 1,300 | - |
| | Soft to Firm Clay | 188.7 – 185.6 | - | 20 |
| | Very Loose to Compact Sand and Silt to Sand | 185.6 – 182.0 | 1,300 | - |
| | Very Dense Sand | 182.0 – 178.7 | 11,000 | - |

Note *Underside of pile cap assumed to be Elevation 191.1 m and 190.8 m at the South Abutment and South Pier, respectively.

At the south pier, for a single HP310X110 pile embedded into the existing fill and native deposits, the estimated maximum lateral resistance at ULS is 75 kN and at SLS, for 10 mm of deflection, is 45 kN (assuming a steel yield strength of 300 MPa). These values are based on analysis carried out using the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

The piles at the south abutment should be socketted into the bedrock by pre-drilling through the granular core. In this case, the lateral resistance of the piles will be developed primarily from the fixity (in concrete) of the socket. In this case, the structural resistance of the pile will govern the lateral resistance.

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

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

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

#### *6.4.1.5 Frost Protection*

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection (OPSD 3090.101, Foundation Frost Depths for Southern Ontario).

### 6.4.2 Caissons

Caissons socketted into the bedrock could be considered as an alternative to steel H-piles at the south abutment and south pier. The high axial capacity of the caissons would result in fewer units being required to support the abutment and pier than that required for the H-pile design, as well as the possible elimination of the pile cap. There will, however, be difficulty in socketting the large diameter caissons within the sloping gneiss bedrock and achieving an adequate seal. Temporary liners and tremie concrete will be required to install caissons at this site.

Based on our experience with caissons socketted into bedrock at similar sites, it is possible that the caissons could be advanced with specialized drilling equipment using a down-hole hammering system consisting of an external liner advanced as the bit is rotated and driven into the bedrock.

#### *6.4.2.1 Geotechnical Axial Resistance*

If caissons are considered as a foundation alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the bedrock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets, which will generally be below the water level and at great depth at the south abutment and south pier. The factored geotechnical axial resistance at ULS for different caisson diameters socketted a minimum of 2 m into the bedrock are given below.

| Caisson Diameter (m) | Gneiss Bedrock (minimum 2 m socket) | |
|---|---|---|
| | ULS (kN) | SLS for 25 mm |
| 0.9 | 4,000 | n/a |
| 1.5 | 8,000 | n/a |

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

Blow-up of the base of the caisson could occur during installation through the overburden at the south pier due to groundwater pressure as the measured groundwater level was noted to be above ground surface. Therefore, a sufficient head of water should be maintained inside the liner at all times to balance the hydrostatic pressures.

#### *6.4.2.2 Resistance to Lateral Loads*

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.4.1.4 using the horizontal subgrade reaction formulas. However, as we understand that caissons are not proposed for the foundation elements at this time, lateral capacities are not required by the designer.

#### *6.4.2.3 Frost Protection*

The pile caps for the caissons should be provided with a minimum of 1.8 m of conventional soil cover for frost protection (OPSD 3090.101).

## 6.5 Seismic Considerations

### 6.5.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site, in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I assuming the silty clay deposit is sub-excavated and replaced with granular fill at the south abutment as discussed in Section 6.4.1 and as recommended in Section 6.8.3.

### 6.5.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHDBC. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Parry Sound area is 0.05. Based on experience, for the subsurface conditions at this site, no amplification of the ground motion is recommended for design (i.e. Site Coefficient, S=1 for Soil Profile I from Table 4.4 of CHBDC), and the Peak Horizontal Acceleration (PHA) is 0.05 g at the ground surface.

We understand that this bridge structure is assigned Seismic Performance Zone 1 based on Section 4.4.4 of the CHBDC. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Zone Performance 1.

## 6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.5.2, seismic (earthquake) loading need not be analyzed for this structure.

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

- Select free-draining granular fill meeting the specifications of Special Provision SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirements) and 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirements).

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing (as outlined in Figure C6.20(b), of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

| | **Granular Fill** | **Rock Fill** |
|---|---|---|
| Soil unit weight: | 21 kN/m$^3$ | 19 kN/m$^3$ |
| Coefficients of static lateral earth pressure: | | |
| Active, $K_a$ | 0.31 | 0.22 |
| At rest, $K_o$ | 0.47 | 0.36 |

- For unrestrained structures, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHDBC.

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

## 6.7 Retained Soil System (RSS) Walls

RSS walls are required on the east side of the south abutment and the west side of the north abutment extending along the approach embankments where there is limited space to construct normal side slopes due to the proximity of the future Highway 529 (i.e. existing Highway 69). The walls will be roughly parallel to the Highway 529 alignment and the geometry of the walls is as follows:

- The southeast wall will be about 6.6 m high extending from the east side of the south abutment to about 7 m south of the abutment, decreasing to a height of 1 m at the south end of the wall about 46 m south of the south abutment.
- The northwest wall is generally located atop the existing rock cut and will be 8.0 m high from the north abutment to 7 m northerly, decreasing to a height of 1 m at the north end of the wall about 19 m north of the north abutment.

An RSS wall consists generally of granular fill placed and compacted in layers and reinforced with fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the vertical face of the reinforced soil structure and to prevent loss of fill material. A typical RSS wall has the front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. For design, a minimum founding depth of 0.8 m is recommended for the facing footing.

The facing footing and soil mass should be placed on a 150 mm thick granular fill levelling pad comprised of compacted SP 110S13 Granular 'A' or Granular 'B' Type II. Where sub-excavation of fill and unsuitable soils is required below the facing footing and soil mass, the sub-excavated area should be backfilled as applicable with rock fill, Granular 'A' or 'B' Type II over which the 150 mm thick granular levelling pad will be placed. Where bedrock is exposed at the facing footing and soil mass founding elevation, we recommend that the bedrock be benched/levelled prior to placement of the 150 mm thick granular fill levelling pad to increase the frictional resistance between the granular pad and the bedrock surface (i.e. to reduce the opportunity for sliding of the granular pad on the bedrock surface).

The proposed final ground surface at the front of the southeast RSS wall will be Elevation 192 m along the length of the wall, and the underside of the facing footing about 1 m below the final ground surface. Due to the variable foundation soils along the length of the southeast wall, we recommend the following:

- The cohesive soil at the south abutment/approach be sub-excavated (to as low as Elevation 186.8 m at Borehole B4-2) below existing ground surface to mitigate settlement and stability issues in this area. The sub-excavated area should be backfilled with Granular 'B' Type II at the abutments to facilitate pile installation. Beyond the abutment, the sub-excavated areas could be backfilled with rock fill to the underside of the RSS granular pad and soil mass and should extend a minimum of 2 m beyond the edge of the soil mass.
- The subgrade below the 150 mm thick granular levelling pad for the facing footing and soil mass will transition from rock fill at the north end of the wall to bedrock near the south end of the wall as Borehole B4-23 encountered auger refusal at Elevation 191.2 m. As the facing footing will be founded on a 150 mm thick granular fill levelling pad placed either on bedrock or rock fill, consideration should be given to placing steel reinforcement within the concrete footing to both sides of the transition to minimize the potential for differential settlement of the footing due to settlement of the foundations soils below the footing/granular levelling pad.

Bedrock is exposed along the footprint of the northwest RSS wall. The proposed final ground surface at the front of the wall will range between Elevation 192 m and 193 m, which is up to about 11 m below the ground surface (i.e. exposed bedrock). The facing footing and soil mass at this location may be founded directly on the bedrock surface, as bedrock excavation will be required for construction of the north abutment and the RSS wall. The facing footing should be constructed on a relatively level bedrock surface to avoid the need for rock dowels.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as 0.8 times the height of the wall, the factored geotechnical axial resistances at ULS and the geotechnical resistance at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared granular fill or rock fill at the south abutment and the bedrock at the north abutment.

| | Wall Height (m) | Assumed Reinforced Width* (m) | Factored Geotechnical Axial Resistance at ULS (kPa) | Geotechnical Resistance at SLS (for 25 mm of settlement) (kPa) |
|---|---|---|---|---|
| South Abutment | 6.6 | 5.3 | 700 | 350 |
| North Abutment | 8.0 | 6.4 | 10,000 | N/A |

* Assumed equivalent to 80 % of the wall height.

The resistance to lateral forces / sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan $\varphi'$, between the compacted granular fills of the RSS wall at the south abutment and the properly prepared granular or rock fill subgrade may be taken as 0.6. The coefficient of friction between the compacted granular fills of the RSS wall at the north abutment and the properly prepared bedrock surface may be taken as 0.7.

The static global stability of the RSS wall at the south abutment has been analyzed and the results of the analysis are discussed in Section 6.8.1. As the wall will be constructed within a rock cut at the north abutment, stability is not a concern at this location. The internal stability of the RSS wall should be checked by the RSS supplier/designer. As discussed in Section 6.8.3, provided that full sub-excavation of the very soft to soft silty clay to clay carried out at the south abutment, settlement below the RSS wall is anticipated to be negligible.

## 6.8 Approach Embankment Design

The south approach of the new Highway 529 Overpass NBL structure will be up to about 7 m high above the existing ground surface and the embankment stability and settlement analysis are focused in this area. At the north approach, excavation of the rock cut is required and as such, stability and settlement are not a concern.

The ground surface at the south approach is generally flat at about Elevation 192 m. The subsoils consist of up to 2.2 m of organic fill and/or peat, up to 3.3 m of very soft silty clay to clay and/or up to 0.5 m of gravelly sand to sand and silt. Beyond the toe of the slope (i.e. in the area of the proposed south pier), up to 4.5 m of very loose to compact granular embankment fill was encountered underlain by up to 2.7 m of soft to firm clay and up to 6.9 m of very loose to dense sand and silt to sand. The water level used for design is Elevation 191 m, consistent with the water level in the open boreholes at the south abutment and the piezometer in Borehole B4-10 at the south pier.

Rock fill is typically the preferred embankment fill material for the Highway 69 project. However, a portion of the approach embankment behind the abutment will consist of granular fill comprising the RSS wall and to facilitate pile driving at the north abutment. The stability and settlement analyses discussed in the following sections have been carried out on the basis that the north embankment will be constructed of granular fill immediately behind the abutment transitioning to rock fill within 20 m of the abutment. Rock fill embankments are assumed to have side slopes at 1.25H:1V.

The analysis assumes that as a minimum, the up to 2.2 m thick peat/organic fill layer will be removed from below the south approach embankment footprint.

The results of stability and settlement analysis for the south approach embankment are presented in the following sections.

### 6.8.1 Stability

Analyses were performed on the south embankment front slope as well as the west side slope located 20 m behind the south abutment as these are considered to be the critical sections for the south approach embankment.

#### *6.8.1.1 Methodology*

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), 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 minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum Factor of Safety was achieved for the design embankment height, slope geometry and stratigraphy. In general, circular slip surfaces were analysed in the design.

#### *6.8.1.2 Parameter Selection*

For the cohesionless fills and native soils, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ SPTs. The correlations proposed by Peck et al (1974) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive native soils, total stress parameters were employed in the analysis. The total stress parameters (i.e. undrained shear strength – $s_u$) for the cohesive soils were assessed based on the results of the in situ field vane tests and estimated from correlations with the SPT results and other laboratory test data. Where appropriate, a correction factor as a function of the plasticity index of the soil, based on Bjerrum (1973), was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the stability analysis.

| Soil Type | Unit Weight ($kN/m^3$) | Undrained Shear Strength (kPa) | Angle of Internal Friction |
|---|---|---|---|
| New Granular Fill for RSS Wall | 21 | -- | $35^o$ |
| New Rock Fill | 19 | -- | $40^o$ |
| Peat | 12 | -- | $27^o$ |
| Existing Granular Fill | 20 | -- | $29^o$ |
| Silty Clay to Clay (under new south approach) | 16 | 6 | -- |
| Silty Clay to Clay (under existing fill) | 16 | 20 | -- |
| Gravelly Sand to Sand and Silt | 20 | -- | $30^o$ |

#### *6.8.1.3 Results of Analysis*

For the 7 m high critical embankment sections described in Section 6.8.1, a Factor of Safety less than 1.0 was obtained for a deep-seated, global trial failure surface that would impact the operation of the roadway, and therefore stability mitigation measures will be required, as discussed in Section 6.8.3.

### 6.8.2 Settlement

Settlement of the south approach embankment can be expected as a result of the loading from the new fills on the foundation soils below the south approach. In addition, settlements may also occur due to compression of the embankment fill itself. Analyses were performed on the south embankment at the abutment and 20 m behind the abutment.

#### *6.8.2.1 Methodology*

To estimate the magnitude of the expected settlements at the south approach, analyses were carried out on the critical section at the south approach (i.e. at the abutment with the thickest cohesive deposit) using the commercially available program Settle$^{3D}$ (by Rocscience Inc.) and spreadsheet and hand calculations. The analyses assume that all fill and organic soils are removed from below the south approach embankment footprint.

#### *6.8.2.2 Settlement Criteria*

Based on MTO’s “Embankment Settlement Criteria For Design” Final Draft dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable within 20 years post-paving for the bridge approach embankments and the new Highway 69 embankment at this site.

| Location | Distance from Transition Point (i.e. Abutment) | Total Post-Construction Settlement (mm) | Differential Settlement Rate* |
|---|---|---|---|
| Transition/Taper to Bridge Abutments | 0 m to 20 m | 25 | -- |
| | 20 m to 50 m | 50 | -- |
| | 50 m to 75 m | 75 | -- |
| | >75 m | 100 | |
| New Highway 69 Embankments | - | - | 200:1 |

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

### *6.8.2.3 Parameter Selection*

The immediate compression of the cohesionless foundation strata was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the silty clay to clay was assessed using the results of the in situ field vane and SPT tests. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967) and Kulhawy and Mayne (1990).

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$s_{u(mob)} = 0.22\sigma_p'$ (after Mesri, 1975)

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$\sigma_p'$ = preconsolidation pressure (kPa)

and $s_{u(mob)} = \mu s_{u(FV)}$ (after Bjerrum, 1973)

where : $s_{u(FV)}$ = undrained shear strength from field vane tests (kPa)
$\mu$ = Bjerrum's correction factor based on Plasticity Index (i.e. about 0.9 for this site)

It is known that some secondary 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 normally consolidated clay at this site. The magnitude of secondary (creep) settlement (Mesri, 1975 as quoted in Holtz and Kovacs, 1981) was estimated using the following:

$S_c = C_{\alpha\varepsilon} \times L_o \times (\Delta \log t)$

Based on Mesri (1975), the following empirical correlation was utilized to estimate $C_{\alpha\varepsilon}$ from water content:

$C_{\alpha\varepsilon} = w_n/100$

where:
- $S_c$ = secondary (creep) settlement (mm)
- $C_{\alpha\varepsilon}$ = modified secondary compression index (%)
- $L_o$ = initial thickness of compressible deposit (mm) in the normally consolidated portion of the deposit
- $w_n$ = water content (decimal)
- $t$ = time period of interest (years)

The following simplified stratigraphy and deformation parameters have been employed in the settlement analysis for the south approach based on Boreholes B4-1 to B4-5 and B4-21.

| Material | Maximum Thickness (m) | | Unit Weight ($kN/m^3$) | Estimated Deformation Properties |
|---|---|---|---|---|
| | Behind Abutment | 20 m Behind Abutment | | |
| New Granular Fill at Abutment* | 10.0 | N/A | 21 | Refer to Section 6.8.2.4 |
| New Rock Fill* | N/A | 8.8 | 19 | Refer to Section 6.8.2.4 |
| Silty Clay to Clay | 2.5 | 1.8 | 16 | (see below) |
| Gravelly Sand to Sand | 0.5 | 0.5 | 20 | E' = 5 MPa |

* Includes removal of organic and/or fill material.

The following consolidation parameters were estimated for the clay deposit based on empirical correlations using the results of the in situ tests and laboratory index tests.

| Location | Elevation (m) | $\sigma_{vo}'$ (kPa) | $\sigma_p'$ (kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v$ ($cm^2/s$) | $C_{\alpha\varepsilon}$ |
|---|---|---|---|---|---|---|---|---|---|
| South Abutment (Borehole B4-3) | 190.0 to 187.5 | 39 | 39 | 1.0 | 2.0 | 0.4 | 0.04 | $1.5\times10^{-3}$ | 0.0075 |

### 6.8.2.4 *Settlement of Embankment Fill*

#### Granular Fill

Settlement of granular fill placed behind and below the new abutments will be nominal provided the granular material is properly placed and compacted. For the case of the granular pad and the granular fill behind the abutment, the settlement from properly compacted granular fill is expected to be less than about 25 mm and will occur during construction.

## Rock Fill

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- Type of rock/strength of particles;
- Size and shape of rock particles;
- Gradation of rock fill;
- Total height/thickness of rock fill (stress level); and
- Method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e. compacted versus dumped rock fill) as outlined in MTO's "Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates", dated September 14, 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with SP 206S03 (Rock Embankment). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

### *Short-Term Rock Fill Settlement*

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO's Guideline (September 2010), as follows:

| Height of Rock Fill, H | Short-Term Rock Fill Settlement | |
|---|---|---|
| | Compacted Rock Fill | Dumped Rock Fill |
| Up to 5 m | 0.5% H | 1.0% H |
| >5 m to 10 m | 0.75% H | 1.5% H |
| >10 m to 15 m | 1.0% H | 2.0% H |

Approximately 90 percent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

### *Long-Term Rock Fill Settlement*

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO's Guideline (September 2010), as follows:

| Height of Rock Fill, H | Long-Term Settlement (m) | |
|---|---|---|
| | Compacted Rock Fill | Dumped Rock Fill |
| Up to 15 m | 0.1% H | 0.2% H |

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction to full height, over the life of the embankment.

## *6.8.2.5 Results of Analysis*

Presented below are the results of the estimated settlement of the foundation soils and embankment rock fill as a result of the embankment construction in the south approach area.

| Location of Embankment | Approximate Embankment Height* (m) | Estimated Settlement of Granular Fill (mm) | Estimated Settlement of Rock Fill (mm) | Estimated Settlement of Cohesionless Soils (mm) | Estimated Consolidation of Clay (mm) |
|---|---|---|---|---|---|
| South Approach (behind abutment) | 9.5 | < 25 | N/A (Granular fill below abutment to facilitate piling as per Section 6.9; RSS wall below and behind abutment) | 0 to 20 | 250 to 450 |
| South Approach (20 m behind abutment) | 8.4 | N/A | 75 | 15 | 220 |

* Includes removal of organic and/or fill material.

The settlements of the cohesionless foundation soils are expected to occur rapidly (i.e. during or shortly after construction) in response to filling.

It is anticipated that 90 percent of the short-term rock fill settlement will occur in the first six months after embankment construction with the remaining settlement expected to occur over the remaining design life of the roadway embankment.

Assuming a coefficient of consolidation ($c_v$) of $1.5 \times 10^{-3}$ $cm^2/s$ (from empirical correlations with laboratory testing) and assuming two-way drainage of the up to 2.5 m thick clay deposit, it is estimated that about 90 percent of the consolidation settlement will be completed in about 4 months after completion of embankment construction.

The magnitude of creep settlement for the clay deposit is estimated to be about 20 mm per log-cycle of time at this location. Therefore, the magnitude of creep settlement is estimated to be about 25 mm from the end of primary consolidation to 10 years after construction.

Since the estimated post-construction settlement of the new embankment within 20 m of the abutment will be greater than the settlement criteria referenced in Section 6.8.2.2, mitigation of post-construction settlement will be required at the south approach. Alternatives to mitigate this settlement are discussed in Section 6.8.3.

### 6.8.3 Mitigation of Stability and Settlement

As discussed in Section 6.8.1.3, the Factor of Safety for stability is less than unity for the proposed 7 m high (above existing grade) south approach embankment constructed over the clayey subsoils after the peat and fill have been removed. Further, as discussed in Section 6.8.2.5, post-construction settlement due to the rock fill and consolidation of the clay deposit is estimated to be up to 75 mm and 450 mm, respectively, at the south approach. Therefore, mitigation measures are required to enhance stability and reduce the magnitude of settlement in this area.

The following sections outline the options and provide recommendations for achieving the target Factor of Safety for the required embankment geometry and for minimizing post-construction settlements that could affect roadway performance. We recommend full sub-excavation of the very soft to soft silty clay to clay below the footprint of the south approach embankment as the preferred alternative from a foundation perspective to mitigate both stability and settlement. Preloading of the rock fill embankment for six months will be required to mitigate (i.e. reduce) the post-construction settlement of the rock fill beyond the RSS wall fill.

#### *6.8.3.1 Full Sub-Excavation and Preloading*

Since the bottom of the cohesive deposit is at relatively shallow depth, up to 4.8 m below the existing ground surface, we recommend full sub-excavation of the cohesive deposit to achieve the best technical solution for the long-term performance of the roadway.

The sub-excavation will likely be carried out ‘in the wet’ (i.e. below the water level) unless dewatering is carried out in the area in advance of the excavation operations. Excavation ‘in the wet’ to remove the cohesive deposit in this area should be carried out with side slopes no steeper than 1H:1V to limit the risk of instability. Removal of the cohesive deposit should extend to a horizontal distance beyond the toe of the proposed embankment equal to the horizontal component of the side slope profile (i.e. 1.25H for rock fill) multiplied by the depth to the bottom of the cohesive deposit below the ground surface (in accordance with OPSD 203.010, Embankments Over Swamp New Construction). Groundwater and surface water control is discussed in Section 6.10.3.

The sub-excavated area within the footprint of the south abutment should be backfilled with granular fill with maximum aggregate size of 75 mm to facilitate pile installation, as further discussed in Section 6.9.

Stability analyses for the critical embankment sections as constructed after full sub-excavation of the silty clay to clay deposit results in a Factor of Safety greater than 1.3, as shown on Figures 1 and 2 for the front and side slopes, respectively.

Following the removal of the cohesive soils at the south approach, the effective thickness of the rock fill embankment will be up to about 10.6 m. The estimated settlement of the new rock fill embankment after construction is estimated to be 100 mm. Since the estimated post-construction settlement of the new rock fill embankment within 20 m of the abutments is greater than the settlement criteria referenced in Section 6.8.2.2, preloading the rock fill embankment for six months is required to mitigate the magnitude of post-construction settlement. After a six-month preload, the post-construction settlement is estimated to be approximately 20 mm for the south approach embankment.

### *6.8.3.2 Other Alternatives*

Mitigation alternatives that were assessed but are not considered feasible at this site include:

- A combination of preloading and surcharging (to mitigate settlement) and toe berms (to mitigate stability) could also be considered depending on the overall construction schedule and property requirements. Given that the east side slope and the east half of the front slope do not have sufficient space for a toe berm due to the location of the proposed Highway 529 (existing Highway 69), this alternative is considered not feasible.
- The loading imposed by the new embankment fill on the soft to firm compressible foundation soils in this area could be reduced by using ultra-lightweight expanded polystyrene (EPS) fill. However, use of this material would be cost prohibitive relative to the cost of full sub-excavation and replacement with rock fill.
- Ground improvement techniques such as rammed aggregate piers to create densified columns within the silty clay to clay deposit resulting in increased stability (although toe berms may still be required depending on the final design of the piers) and reduced settlements. The design and construction of this alternative would not be considered practical or cost effective at this site.
- Wick drains to enhance the rate of consolidation settlement of the silty clay to clay deposit would not be practical at this site since the thickness of the clay is relatively small. The drainage path would not be significantly decreased such that the time required for preloading would be reduced and the strength gain within the clay would not be sufficient to support the proposed embankment height. Therefore, the extra cost of design, installation and monitoring for this alternative is not considered economically or technically feasible.
- Increasing the length of the bridge could have a positive impact on reducing the settlement and enhancing stability of the approach embankments. If the bridge was lengthened so that the south abutment was at least 30 m south of the currently proposed location, then stability and settlement may not be an issue (additional boreholes would be required to confirm). If the final road grade was lowered, then the load on the foundation soils would be reduced and stability and settlement may not be as much of a concern (depending on the final grade). However, the substantial increase in cost associated with a longer bridge makes this alternative impractical.

## 6.9 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all existing fill, peat and topsoil/vegetation/organic soils must be removed from below the footprint of the proposed embankments. In addition, full sub-excavation of the cohesive deposit at the south approach should be carried out as recommended in Section 6.8.3.

Placement of rock fill material should be carried out in accordance with the requirements as outlined in SP206S03 (Rock Embankment). Side slopes for rock fill embankments should be no steeper than 1.25H:1V and any section of the approach embankment higher than 10 m should incorporate a 2 m wide bench in the slope as per OPSD 202.010 (Slope Flattening on Rock Embankment).

As the south abutment will be founded on steel H-piles driven to or socketted into bedrock, a compacted granular 'core' will be required after the cohesive soils are removed. The compacted granular 'core' should be constructed in accordance with the geometry shown on Figure 3 such that it is integrated into the rock fill embankment. We recommend the 'core' be constructed using Granular 'A' material. Alternatively, Granular 'B' Type II with maximum particle size of 75 mm could be considered. An NSSP should be included in the contract for the restriction of the size of the Granular 'B' Type II to 75 mm; an example is included in Appendix C. The granular core should extend at least 1 m beyond the plan limits of the pile cap area and be sloped no steeper than 1H:1V. The granular 'core' should be constructed in accordance with SP206S03, Earth Excavation, Grading.

### 6.9.1 Embankment Widening

In accordance with the requirements of MTO NRE 98-200 (Embankment Design Guidelines), the minimum required embankment widening to account for future pavement overlays is 2 m per embankment side.

## 6.10 Design and Construction Considerations

### 6.10.1 Blasting for Rock Excavations

At the north abutment and north pier, bedrock excavation will be required to create a level platform for the shallow foundations. For bedrock excavation, the overall slope of the cut face may be formed vertical or at a steep slope (i.e. 0.25H:1V). All bedrock excavation within and near footing areas should be carried out using controlled blasting techniques in order to minimize shattering and over-break. The use of line drilling, pre-shearing or cushion blasting is recommended in order to provide a neat excavation line and minimize face instability resulting from blast damage to the rock mass. Good blasting practices will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations. We recommended that the Contractor retain a blast engineer and submit proposed blast plans for review at least three weeks in advance of rock excavation.

For the north abutment, upon completion of blasting and prior to footing construction, we recommend that a rock quality specialist be retained by the Contract Administrator to review the bedrock surface within and rock mass surrounding the footprint of the footing and address any remedial measures that may be required as a result of blasting. Additional recommendations for bedrock excavation and foundation preparation for the north abutment are provided in Golder Associate's Report No. 09-1117-0022 dated May 28, 2010, dealing with the rock fall hazard assessment of the existing rock cut (referenced in Section 6.1).

### 6.10.2 Excavations

Sub-excavation of the subsoils up to 4.8 m below ground surface at the south abutment and up to 3.2 m about 20 m behind the abutment will be required to mitigate settlement and stability at the south approach and for spread footing at the south abutment. At the south pier, up to 3.2 m of existing embankment fill will require removal prior to pile cap construction. Conventional excavators should be suitable for the excavating operations at these locations.

In general, excavations will extend below the groundwater level and dewatering will be required to carry out the construction of the south abutment footing and south pier pile cap in-the-dry, as discussed in Section 6.10.3.

Temporary excavation side slopes above the water level within the fill and native soils should be no steeper than 1H:1V. The existing fill, peat and silty clay to clay may be classified as a Type 4 soil while the native cohesionless soils at this site may be classified as a Type 3 soil. All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA) and good construction practices.

Given the traffic will be routed to the new detour located west of the proposed structure, excavations can be carried out without the need for temporary shoring.

### 6.10.3 Groundwater and Surface Water Control

Excavations to expose the bedrock surface at the south abutment/approach will extend up to about 4 m below the water table. Excavations to the underside of the south pier pile cap will extend up to 1 m below the water table. The excavation can be carried out below the water level without dewatering measures, in accordance with OPSS 209, Construction Specification for Embankments Over Swamps and Compressible Soils. However, dewatering may required in order to construct the south abutment and pier foundations in-the-dry.

Excavations at the north pier and north abutment/approach will generally be carried out in-the-dry given the bedrock outcrop topography, and existing and proposed ground surface elevations at these locations. Groundwater/surface water may drain into the new highway cut.

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

### 6.10.4 Obstructions

Cobbles and boulders were encountered within the cohesionless soils at the south pier. We recommend that an Operational Constraint (OC) be included in the Contract Documents to alert the Contractor of the presence of these obstructions as they may affect pile driving operations; an example is included in Appendix C.

## 7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., Associate. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal, conducted an independent quality control review of the report.

## Report Signature Page

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N:\Active\2007\1190 Sudbury\1191\07-1191-0020 MMM Hwy 69 Twinning\7000 Reporting\Final\Hwy 529 Overpass\NBL\07-1191-0020-B4 RPT 12Mar14 Hwy 529 OP NBL.Docx

## REFERENCES

Bjerrum, L., 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State of the art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.

Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.

Canadian Foundation Engineering Manual, 4th Edition, Canadian Geotechnical Society 1992.

Canadian Foundation Engineering Manual, 4th Edition, Canadian Geotechnical Society 2006.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06, 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.

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

Holtz, D.R. and Kovacs, W.D., 1981. An Introduction to Geotechnical Engineering, Prentice Hall Inc.

Kulhawy, F.H. and Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Mesri, G., 1975. Discussion on new design procedure for stability of soft clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.

NAVFAC Design Manual DM 7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy, 1982. Alexandria, Virginia.

Ontario Geological Society, 1991. Geology of Ontario, Special Volume 4, Part 2. Ministry of Northern Development and Mines, Ontario.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.

Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

**STANDARDS:**

ASTM International:

| | |
|---|---|
| ASTM D1586-08a | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D5731-08 | Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications |

Commercial Software

LPile (Version 5.0) by Ensoft Inc.

GeoStudio (Version 7.17) by Geo-Slope International Ltd.

Settle 3D by Rocscience Inc.

Ministry of Transportation, Ontario, 2008. Structural Manual. Quality and Standards, Transportation Engineering Branch, Bridge Office, Design Section.

Ministry of Transportation, Ontario. 2010. Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity, April 12, 2010.

Ministry of Transportation Ontario, 2010. Embankment Settlement Criteria for Design, Final Draft, March 2, 2010

Ministry of Transportation, Ontario. 2002. Backfill to Structures Adjacent to Rock Embankment Approaches, Northern Region Directive, November 2002.

Ministry of Transportation, Ontario. 1998. Northern Region Embankment Design Guidelines, Northern Region Directive, NRE, 98-200, issued by Geotechnical Section, October 1998.

Ministry of Transportation Ontario Special Provisions

| | |
|---|---|
| SP 110S13 | Material Specifications for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |
| SP 206S03 | Rock Excavation, Grading, Rock Embankment |

Ontario Provincial Standard Specification:

| | |
|---|---|
| OPSS 209 | Construction Specification for Embankments over Swamps and Compressible Soils |
| OPSS 501 | Construction Specification for Compacting |
| OPSS 539 | Construction Specification for Temporary Protection Systems |

Ontario Provincial Standard Drawings Ontario Provincial Standard Drawings

| | |
|---|---|
| OPSD 202.010 | Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment |
| OPSD 203.010 | Embankments Over Swamp, New Construction |
| OPSD 208.010 | Benching of Earth Slopes |
| OPSD 3090.101 | Foundation Frost Depths for Southern Ontario |
| OPSD 3101.150 | Walls Abutment, Backfill Minimum Granular Requirement |
| OPSD 3101.200 | Walls Abutment, Backfill Rock |

| | |
|---|---|
| OPSD 3121.150 | Walls Retaining, Backfill Minimum Granular Requirement |

Ontario Water Resources Act:

| | |
|---|---|
| Ontario Regulation 372/97 | Amendment to Ontario Regulation 903 |
| Ontario Regulation 903/90 | Wells |

CONT No.
GWP No. 5191-06-01

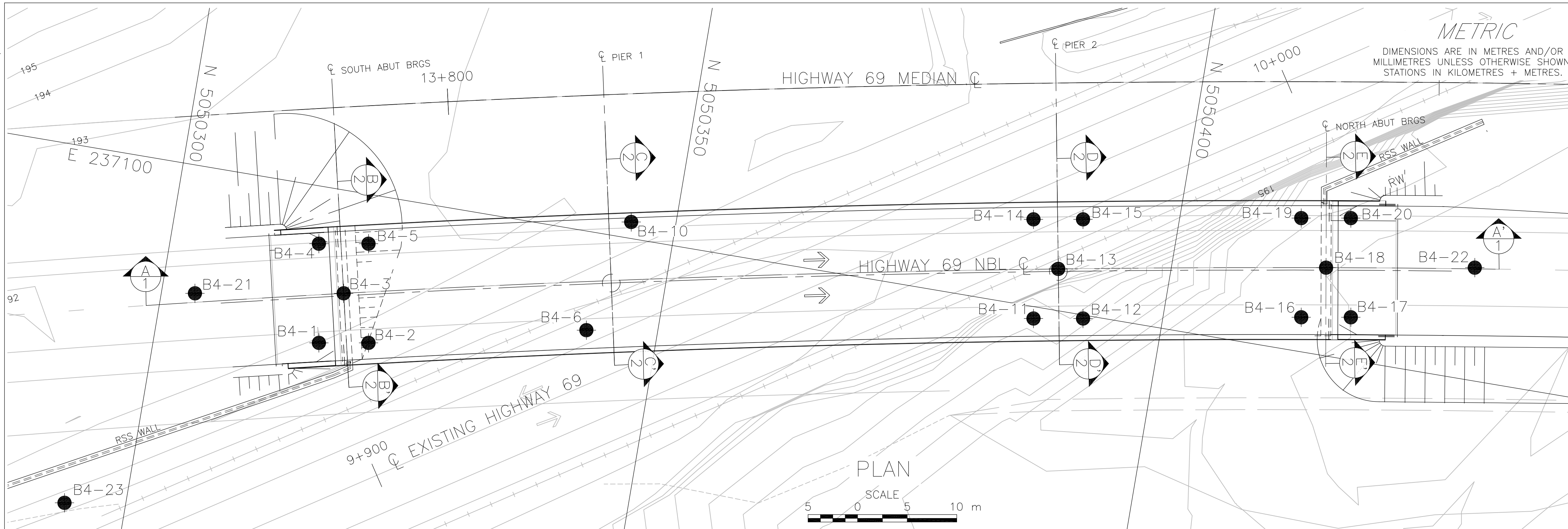
HIGHWAY 69
HWY 529 OVERPASS (NBL)
BOREHOLE LOCATION
AND SOIL STRATA

SHEET

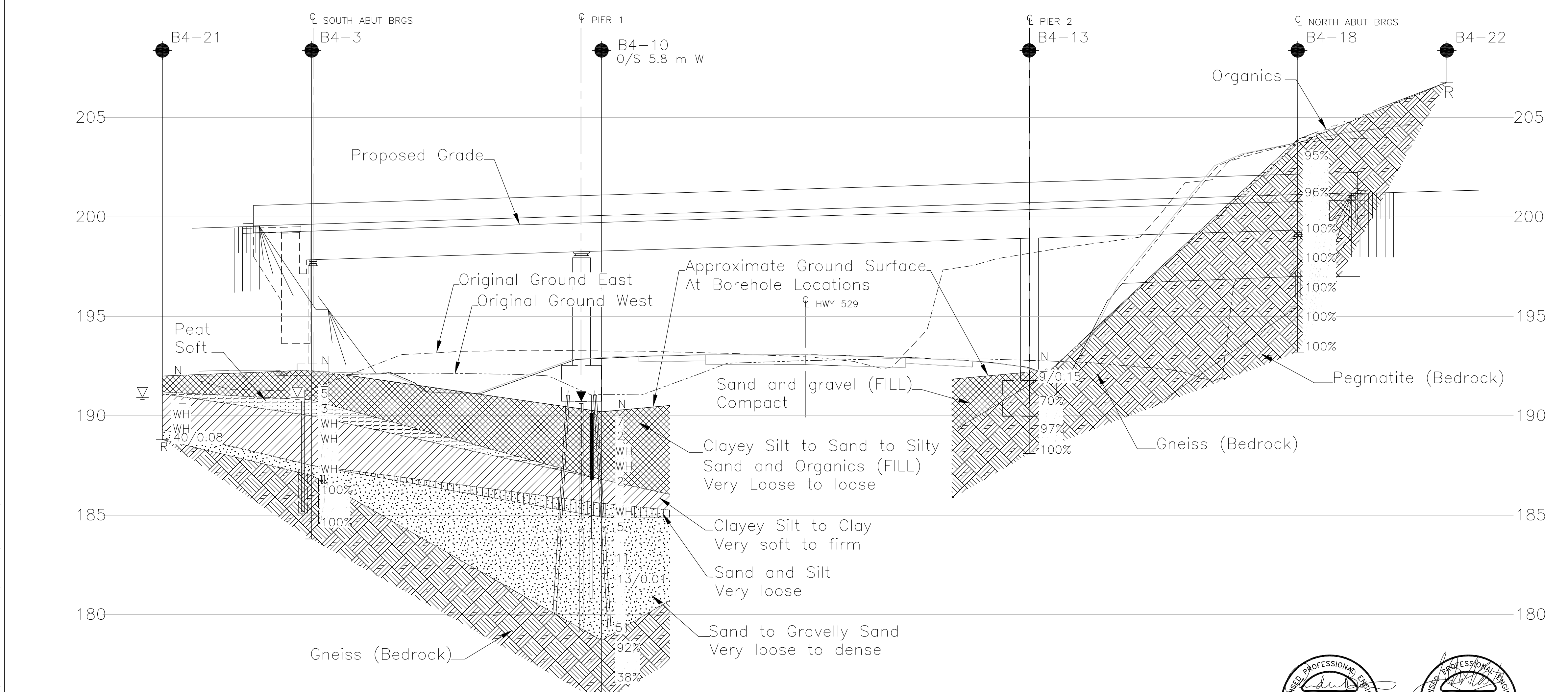
**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA

MINISTRY OF TRANSPORTATION, ONTARIO

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

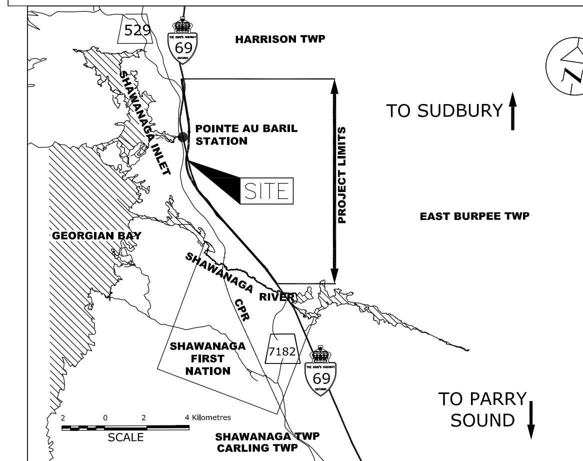



PLAN
SCALE
5 0 5 10 m




A-A' 1 CENTRELINE PROFILE
HIGHWAY 529 OVERPASS (NBL)

HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE




KEY PLAN

## LEGEND

- ● Borehole
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- ∇ WL upon completion of drilling
- ▼ WL in piezometer, measured on March 20, 2009
- Seal
- Piezometer
- R Refusal
- 100% Rock Quality Designation (RQD)

| No. | ELEVATION | CO-ORDINATES | |
|---|---|---|---|
| | | NORTHING | EASTING |
| B4-1 | 191.8 | 5050316.5 | 237115.6 |
| B4-2 | 191.1 | 5050321.4 | 237114.8 |
| B4-3 | 192.3 | 5050318.1 | 237110.3 |
| B4-4 | 192.2 | 5050314.8 | 237105.8 |
| B4-5 | 192.1 | 5050319.8 | 237104.9 |
| B4-6 | 193.2 | 5050342.9 | 237109.9 |
| B4-10 | 190.9 | 5050345.5 | 237098.3 |
| B4-11 | 197.6 | 5050387.2 | 237101.2 |
| B4-12 | 199.3 | 5050392.1 | 237100.4 |
| B4-13 | 192.2 | 5050388.8 | 237095.8 |
| B4-14 | 192.8 | 5050385.5 | 237091.3 |
| B4-15 | 192.7 | 5050390.4 | 237090.5 |
| B4-16 | 203.8 | 5050413.8 | 237096.6 |
| B4-17 | 205.0 | 5050418.7 | 237095.7 |
| B4-18 | 203.9 | 5050415.4 | 237091.2 |
| B4-19 | 202.9 | 5050412.1 | 237086.7 |
| B4-20 | 203.2 | 5050417.0 | 237085.9 |
| B4-21 | 192.0 | 5050303.3 | 237112.8 |
| B4-22 | 205.2 | 5050430.2 | 237088.7 |
| B4-23 | 193.5 | 5050293.9 | 237135.8 |

## 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.

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LICENSED PROFESSIONAL ENGINEER A.J.K. BOM 100075715 MAR 14, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA MAR 14, 2012 PROVINCE OF ONTARIO

## REFERENCE

Base plans provided in digital format by MMM, drawing file nos. HWY 529 OVERPASS NBL P1-GENERAL ARRANGEMENT.DWG, received Nov. 14, 2011.

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

Geocres No. 41H-85

| | | | |
|---|---|---|---|
| HWY. 69 | PROJECT NO. 07-1191-0020 | | DIST. |
| SUBM'D. EC | CHKD. AB | DATE: MAR 2012 | SITE: 44-446/1 |
| DRAWN: JJL | CHKD. SEMC | APPD. JMAC | DWG. 1 |

PLOT DATE: March 13, 2012

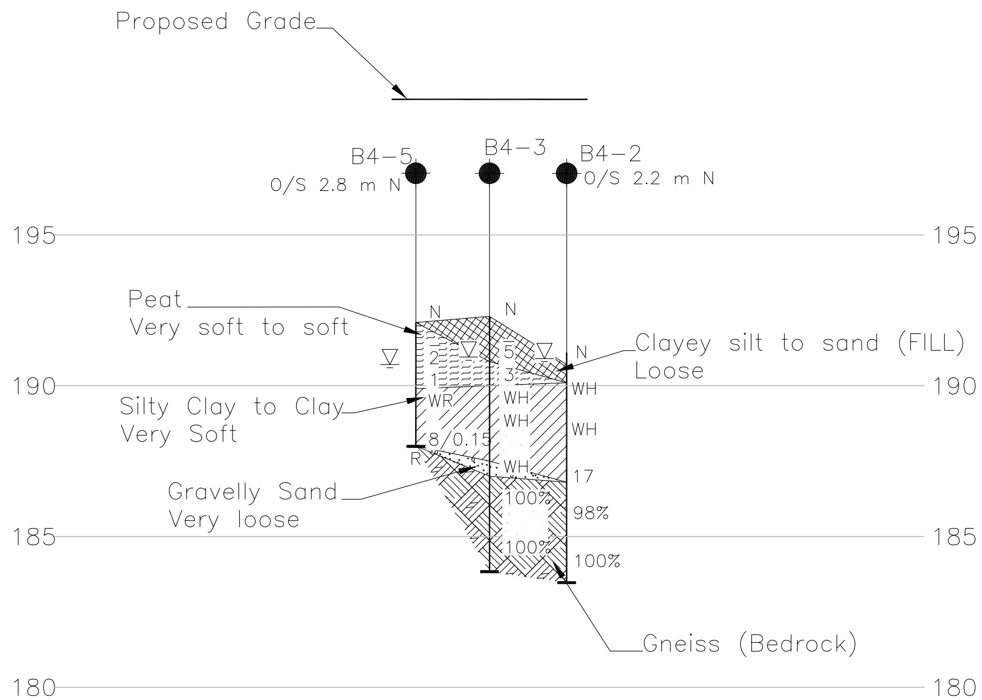
*METRIC*
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5191–06–01

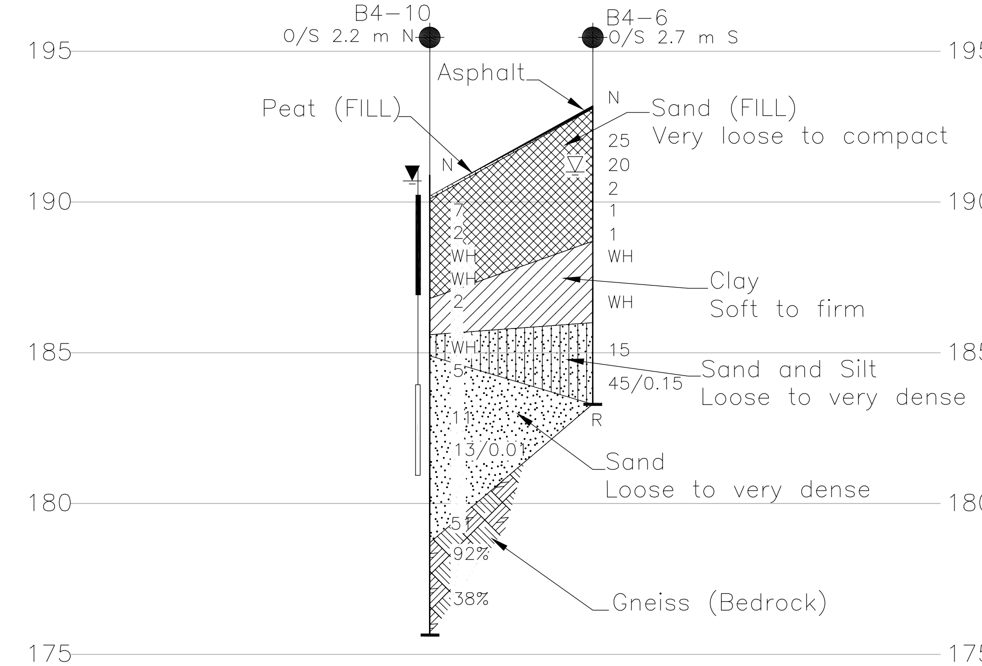
HIGHWAY 69
HWY 529 OVERPASS (NBL)
SOIL STRATA

SHEET

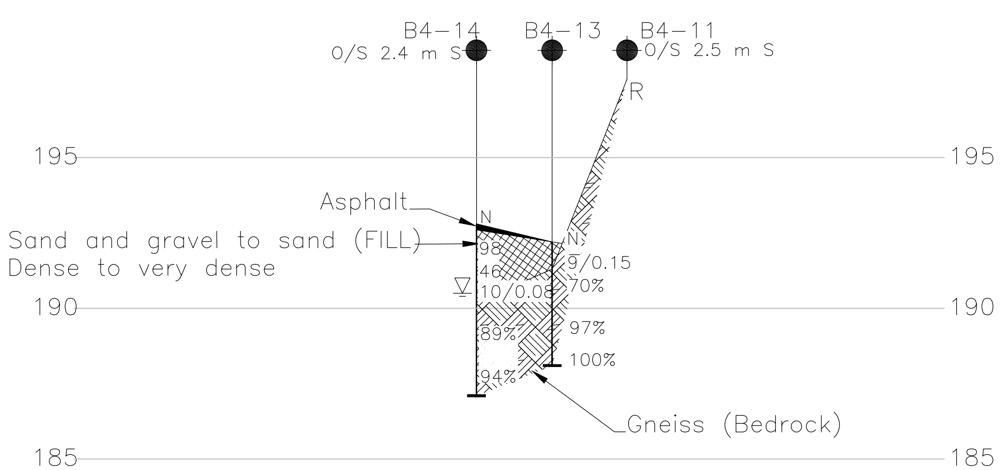
Golder Associates
**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA



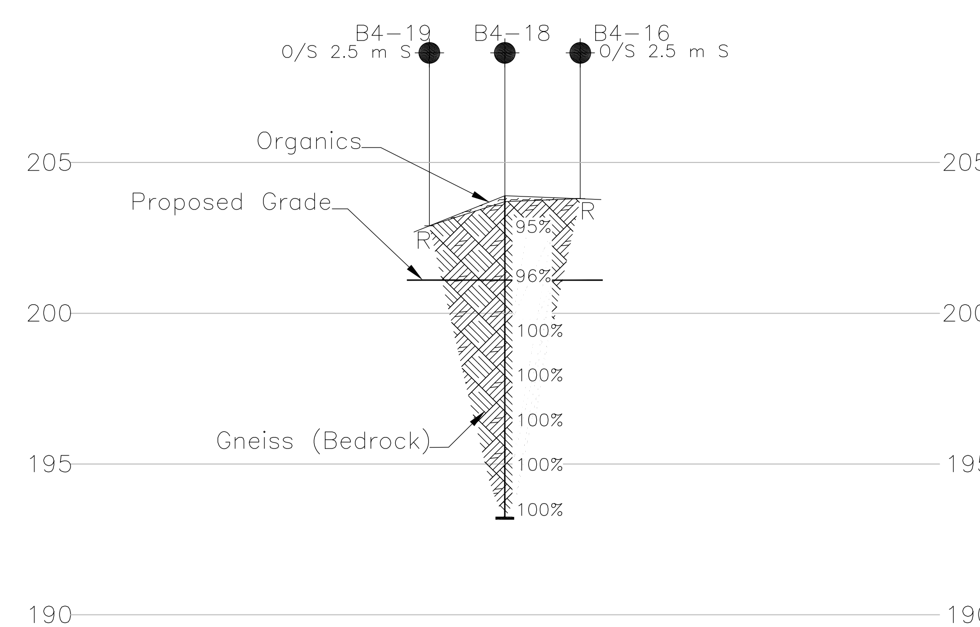

B-B' 1 SOUTH ABUTMENT SECTION
HIGHWAY 529 OVERPASS (NBL)
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE




C-C' 1 PIER 1 SECTION
HIGHWAY 529 OVERPASS (NBL)
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE




D-D' 1 PIER 2 SECTION
HIGHWAY 529 OVERPASS (NBL)
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE




E-E' 1 NORTH ABUTMENT SECTION
HIGHWAY 529 OVERPASS (NBL)
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE

LICENSED PROFESSIONAL ENGINEER A.J.K. BOM 100075715 MAR 14, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA MAR 14, 2012 PROVINCE OF ONTARIO

## LEGEND

| | |
|---|---|
| ● | Borehole |
| N | Standard Penetration Test Value |
| 4 | Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow) |
| ▽ | WL upon completion of drilling |
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| ▮ | Seal |
| ▯ | Piezometer |
| R | Refusal |
| 100% | Rock Quality Designation (RQD) |

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| B4–6 | 193.2 | 5050342.9 | 237109.9 |
| B4–10 | 190.9 | 5050345.5 | 237098.3 |
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| B4–12 | 199.3 | 5050392.1 | 237100.4 |
| B4–13 | 192.2 | 5050388.8 | 237095.8 |
| B4–14 | 192.8 | 5050385.5 | 237091.3 |
| B4–15 | 192.7 | 5050390.4 | 237090.5 |
| B4–16 | 203.8 | 5050413.8 | 237096.6 |
| B4–17 | 205.0 | 5050418.7 | 237095.7 |
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| B4–19 | 202.9 | 5050412.1 | 237086.7 |
| B4–20 | 203.2 | 5050417.0 | 237085.9 |
| B4–21 | 192.0 | 5050303.3 | 237112.8 |
| B4–22 | 205.2 | 5050430.2 | 237088.7 |

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| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

Geocres No. 41H–85

| | | | |
|---|---|---|---|
| HWY. 69 | | PROJECT NO. 07–1191–0020 | DIST. |
| SUBM'D. EC | CHKD. AB | DATE: MAR 2012 | SITE: 44–446/1 |
| DRAWN: JJL | CHKD. SEMC | APPD. JMAC | DWG. 2 |

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: March 13, 2012
FILENAME: N:\Active\2007\1190 Sudbury\1191\07-1191-0020 MMM Hwy 69 Twinning\5000 Drawings\Structure Location Plan\Hwy 529 Overpass Structure (B3 and B4)\0711910020B4001(B4).dwg

**Table 1: Evaluation of Foundation Alternatives - South Abutment**

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Steel H-Piles Driven to Bedrock | 1 | ■ Standard construction.<br>■ Dewatering not required. | ■ Requires granular pad below the abutment to allow for pile installation.<br>■ Pre-drilled holes into bedrock (sockets) may be required to achieve minimum pile length.<br>■ Pile tips need to be reinforced with rock points to help seat pile on sloping bedrock (if driven) or specialized drilling equipment required for creating 0.6 m diameter socket in strong to very strong bedrock.<br>■ Temporary liners would be required for groundwater control and support through overburden where rock socketting is required. | ■ Higher relative cost than spread footings on bedrock (excluding higher costs associated with dewatering to expose bedrock for spread footings) but lower relative costs compared with caisson option. | ■ May not achieve minimum pile lengths, unless pre-drilling/ socketting is carried out.<br>■ Appropriate pile driving operation required to seat pile on sloping bedrock. |
| Spread Footings Founded on Bedrock Surface | 2 | ■ Standard construction. | ■ Requires removal of overburden to a depth of about 3.2 m to 5.3 m to expose bedrock surface.<br>■ Mass concrete or bedrock excavation required to level the foundation due to sloping rock surface.<br>■ Dewatering of footing excavation required and may not be readily possible due to the variable bedrock surface.<br>■ May require rock anchors to provide for lateral sliding resistance. | ■ Lower relative cost compared to deep foundation alternatives.<br>■ Additional costs required for dewatering. | ■ Difficulties associated with properly constructing/anchoring footings on sloping bedrock.<br>■ Potential for not being able to achieve adequate dewatering of the excavation due to varying depth and sloping bedrock surface. |

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Caissons Socketted into Bedrock | 3 | ■ Reduced number of deep elements compared to steel H-piles (higher axial resistance per unit).<br>■ Possible elimination of pile cap by extending caisson to underside of bridge. | ■ Concrete for caissons would have to be placed by tremie methods below the water level.<br>■ Difficulty socketting caissons into gneiss bedrock. | ■ Cost many times higher than for piles. | ■ Risk of difficulties achieving seal and drilling large diameter socket into gneiss bedrock. |

**Table 2: Evaluation of Foundation Alternatives - South Pier**

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Steel H-Piles Driven to Bedrock | 1 | ■ Standard construction. | ■ Pile tip needs to be reinforced with rock points to seat pile on sloping bedrock. | ■ Lower relative costs compared with caisson option. | ■ Appropriate pile driving operation required to seat pile on sloping bedrock |
| Caissons Socketted into Bedrock | 2 | ■ Reduced number of deep elements compared to steel H-piles (higher axial resistance per unit).<br>■ Possible elimination of pile cap. | ■ Temporary liners would be required for soil support and groundwater control through the granular overburden.<br>■ Concrete for caissons would have to be placed by tremie methods below the water level.<br>■ Difficulty socketting caissons into gneiss bedrock. | ■ Cost many times higher than for steel H-piles. | ■ Risk of difficulties achieving seal and drilling large diameter socket into gneiss bedrock. |

# Highway 529 Overpass NBL
# South Approach Front Slope Stability (Full Sub-Excavation)

**FIGURE 1**

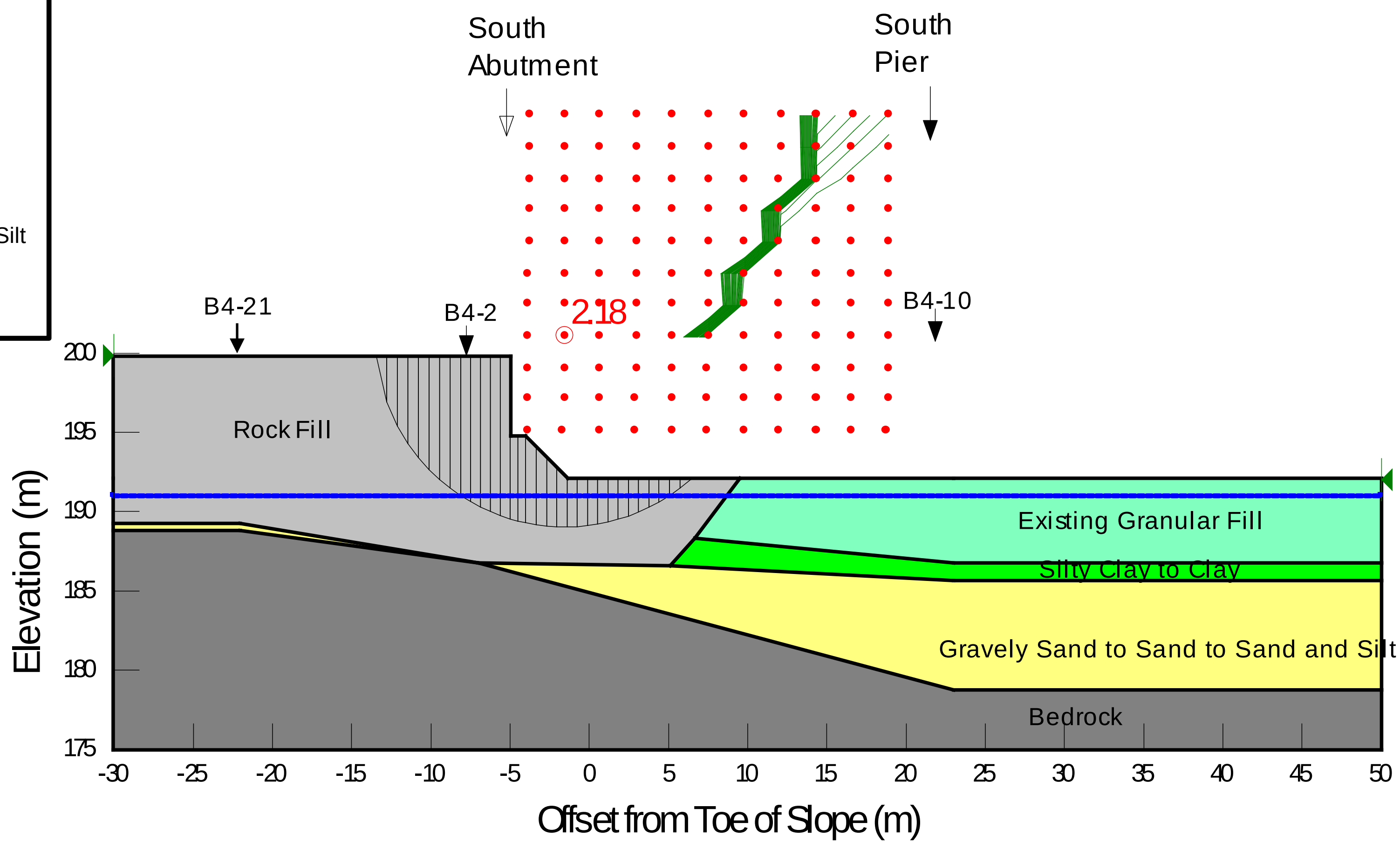
Rock Fill
Unit Weight: 19 kN/m$^3$
Phi: 40°

New Granular Fill
Unit Weight: 21 kN/m$^3$
Phi: 35°

Existing Granular Fill
Unit Weight: 20 kN/m$^3$
Phi: 29°

Silty Clay to Clay
(underlying existing fill)
Unit Weight: 16kN/m$^3$
Cohesion: 20 kPa

Gravelly Sand to Sand and Silt
Unit Weight: 20 kN/m$^3$
Phi: 30°




**Date:** March 2012

**Project No.:** 07-1191-0020-B4

**Analysis By:** AB **Reviewed By:** SEMC

Golder Associates

# Highway 529 Overpass NBL
## South Approach Side Slope Stability (Full Sub-Excavation)

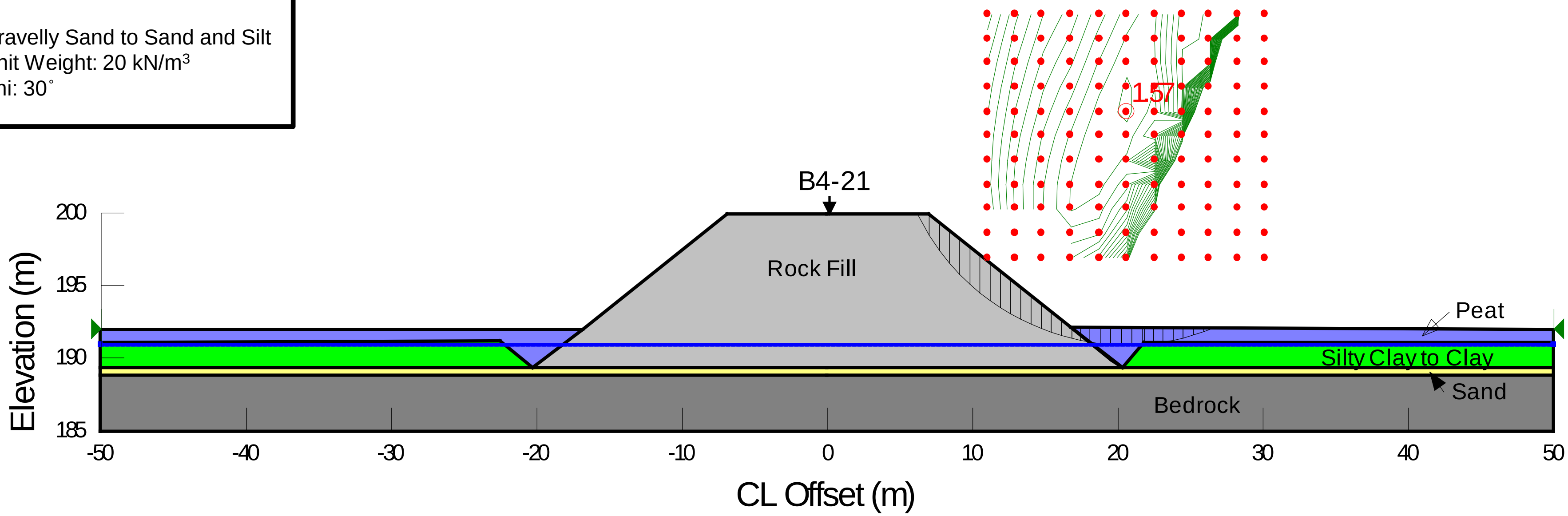
**FIGURE 2**

Rock Fill
Unit Weight: 19 kN/m³
Phi: 40˚

Peat
Unit Weight: 12 kN/m³
Phi: 27˚

Silty Clay to Clay
(new approach)
Unit Weight: 16kN/m³
Cohesion: 6 kPa

Gravelly Sand to Sand and Silt
Unit Weight: 20 kN/m³
Phi: 30˚

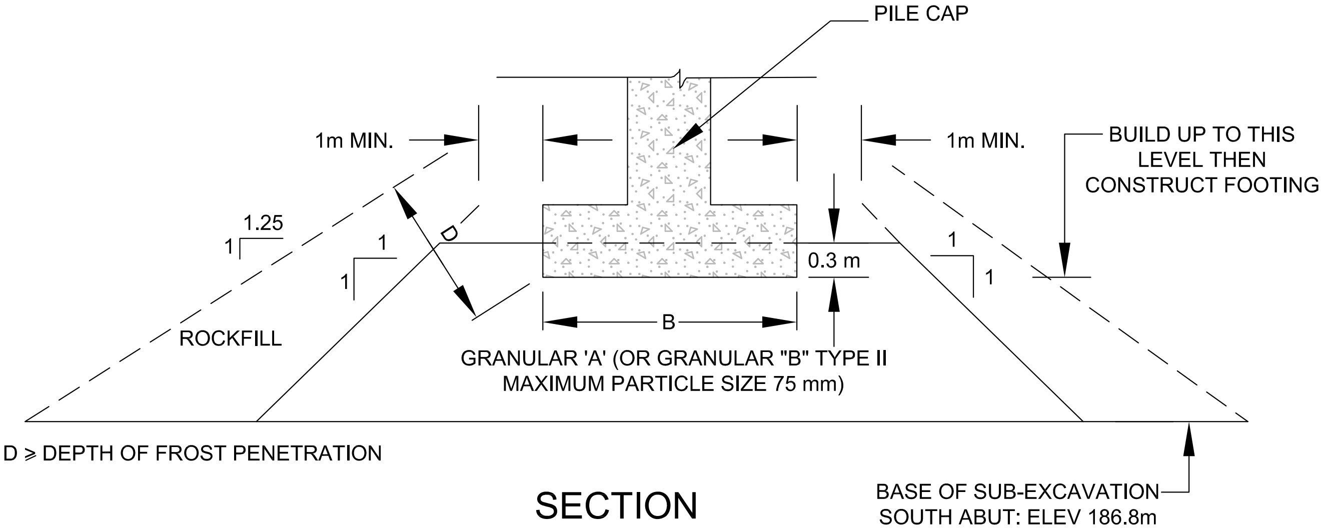



**Date:** March 2012

**Project No.:** 07-1191-0020-B4

**Analysis By:** AB **Reviewed By:** SEMC

Golder Associates




SECTION

**CONSTRUCTION SEQUENCE:**

1. REFER TO ACCOMPANYING FOUNDATION DESIGN REPORT, SECTION 6.2.

2. REMOVE SUBSOILS UNDER FOOTPRINT OF COMPACTED GRANULAR CORE TO ELEVATION SPECIFIED.

3. PLACE AND COMPACT GRANULAR 'A' ( OR GRANULAR 'B' TYPE II MAXIMUM PARTICLE SIZE 75 mm) IN ACCORDANCE WITH SP206S03 TO UNDERSIDE OF ABUTMENT. CONSTRUCT SURROUNDING ROCK FILL CONCURRENTLY WITH GRANULAR 'A' / 'B' TYPE II.

4. INSTALL PILES.

5. CONSTRUCT CONCRETE PIER CAP (OR SPREAD FOOTING IF APPLICABLE).

5. PLACE REMAINDER OF GRANULAR 'A' AND BACKFILL AS REQUIRED.

6. SOURCE M.T.C 1982.

PROJECT

GWP 5005-08-00
HIGHWAY 529 OVERPASS NBL

TITLE

**TYPICAL COMPACTED FILL CORE**

Golder Associates
Sudbury, Ontario, Canada

| | | | | |
|---|---|---|---|---|
| PROJECT No. | | 07-1191-0020 | FILE No. 07-1191-0020-B4 FIG3.dwg | |
| DESIGN | | | SCALE NTS | REV. |
| CAD | JJL | MAR 2012 | FIGURE No. | |
| CHECK | AB | MAR 2012 | 3 | |
| REVIEW | JMAC | MAR 2012 | | |

# APPENDIX A

## Record of Boreholes and Drillholes

# LIST OF SYMBOLS

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

## 1. GENERAL

| | |
|---|---|
| $\pi$ | 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 |
| FoS | Factor of Safety |
| V | volume |
| W | weight |

## II. STRESS AND STRAIN

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. stress: $\Delta\sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| $\nu$ | Poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma$-u) |
| $\sigma_{vo}$ | initial effective overburden stress |
| $\sigma_1$, $\sigma_2$, $\sigma_3$ | principal stress (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$ | 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 |
| $\gamma'$ | 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 $\rho$. Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density multiplied by 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 |
| $\sigma'_p$ | pre-consolidation pressure |
| OCR | over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$ |

### (d) Shear Strength

| | |
|---|---|
| $\tau_p$, $\tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | 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

# LIST OF ABBREVIATIONS

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

## I. SAMPLE TYPE

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

## II. PENETRATION RESISTANCE

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

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

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

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

## III. SOIL DESCRIPTION

**(a) Cohesionless Soils**

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

**(b) Cohesive Soils**
**Consistency**

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

## IV. SOIL TESTS

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

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

## V. MINOR SOIL CONSTITUENTS

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

## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERING STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of Major discontinuities

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock Mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

### BEDDING THICKNESS

| Description | Bedding Plane Spacing |
|---|---|
| Very thickly bedded | > 2 m |
| Thickly bedded | 0.6 m to 2 m |
| Medium bedded | 0.2 m to 0.6 m |
| Thinly bedded | 60 mm to 0.2 m |
| Very thinly bedded | 20 mm to 60 mm |
| Laminated | 6 mm to 20 mm |
| Thinly laminated | < 6 mm |

### JOINT OR FOLIATION SPACING

| Description | Spacing |
|---|---|
| Very wide | > 3 m |
| Wide | 1 – 3 m |
| Moderately close | 0.3 – 1 m |
| Close | 50 – 300 mm |
| Very close | < 50 mm |

### GRAIN SIZE

| Terms | Size* |
|---|---|
| Very Coarse Grained | > 60 mm |
| Coarse Grained | 2 – 60 mm |
| Medium Grained | 60 microns – 2 mm |
| Fine Grained | 2 – 60 microns |
| Very Fine Grained | < 2 microns |

* Note: Grains > 60 microns diameter are visible to the naked eye.

### CORE CONDITION

**Total Core Recovery (TCR)**

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

**Solid Core Recovery (SCR)**

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

**Rock Quality Designation (RQD)**

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

### DISCONTINUITY DATA

**Fracture Index**

A count of the number of discontinuities (physical separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

**Dip with Respect to (W.R.T.) Core Axis**

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

**Description and Notes**

An abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

**Abbreviations**

| | | | |
|---|---|---|---|
| B | - Bedding | ⊥ | - Perpendicular To |
| FO | - Foliation / Schistosity | ∥ | - Parallel To |
| CL | - Cleavage | P | - Polished |
| SH | - Shear Plane / Zone | K | - Slickensided |
| VN | - Vein | SM | - Smooth |
| F | - Fault | R | - Rough |
| CO | - Contact | ST | - Stepped |
| J | - Joint | PL | - Planar |
| FR | - Fracture | U | - Undulating |
| MF | - Mechanical Fracture | C | - Curved |

# RECORD OF BOREHOLE No B4- 1

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050316.5; E 237115.6 ORIGINATED BY EHS

DIST ___ HWY 69 BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers COMPILED BY MR

DATUM Geodetic DATE February 3, 2009 CHECKED BY AB

| ELEV / DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa | WATER CONTENT (%) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 191.8 | GROUND SURFACE | | | | | | | | | |
| 0.0 | Peat containing sand, silt and clay layers (FILL) Very soft Brown Moist to wet | 1 | SS | 2 | | 191 | | 58.2 | | |
| | | | | | ∇ | | | | | |
| 190.0 / 1.8 | CLAY Very soft Brown to grey Wet | 2 | SS | WH | | 190 | | | | |
| | | 3 | SS | WR | | | | 52; 74.9 | | |
| | | | | | | 189 | + 4 | | | |
| 187.8 / 4.2 | SAND and SILT, some sand Loose Grey Wet | 4 | SS | 3/0.05 | | 188 | | | | |

End of Borehole
Spoon and Auger Refusal

Note:

1. Water level at a depth of 1.5 m below ground surface (Elev. 190.3 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

Golder Associates

# RECORD OF BOREHOLE No B4- 2

 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 07-1191-0020 | | | |
| W.P. 5191-06-01 | LOCATION N 5050321.4; E 237114.8 | | ORIGINATED BY EHS |
| DIST | HWY 69 | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | COMPILED BY MR |
| DATUM Geodetic | DATE February 4, 2009 | | CHECKED BY AB |

| ELEV DEPTH (m) | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|---|
| 191.1 | SNOW SURFACE | | | | | |
| 0.0 | SNOW | | | | | |
| 190.7 / 0.4 | Peat containing sand and silt (FILL)<br>Brown<br>Wet | | | | ∇ | |
| 190.1 / 1.0 | SILTY CLAY<br>Very soft to soft<br>Brown to grey<br>Wet | 1 | SS | WH | | |
| | | 2 | SS | WH | | |
| | | 3 | SS | 17 | | |
| 186.8 / 4.3 | GNEISS (BEDROCK)<br>Bedrock cored from 4.3 m depth to 7.5 m depth.<br>For coring details refer to Record of Drillhole B4-2. | 1 | RC | REC 100% | | RQD = 98% |
| | | 2 | RC | REC 100% | | RQD = 100% |
| 183.6 / 7.5 | End of Borehole<br>Note:<br>1. Water level at a depth of 0.5 m below ground surface (Elev. 190.6 m) upon completion of drilling. | | | | | |

Elevation scale: 191, 190, 189, 188, 187, 186, 185, 184

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

Field vane results (sensitivity): 5, 5, 3, 4

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

Water content plotted at ~Elev. 188.5: 49, 75.4

UNIT WEIGHT γ kN/m³

GR SA SI CL

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

# RECORD OF DRILLHOLE: B4- 2

PROJECT: 07-1191-0020

SHEET 1 OF 1

LOCATION: N 5050321.4 ;E 237114.8

DRILLING DATE: February 4, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|
| | | Refer to Previous Page | | | | | |
| | NQ Coring 02/04/09 | GNEISS<br>Fine to medium grained<br>Fresh<br>Very strong<br>Pinkish grey | | 186.8<br>4.3 | 1 | J, I, R<br>J, I, R | |
| 6 | | | | | 2 | | UCS = 138 MPa |
| | | | | 183.6<br>7.5 | | | |
| | | End of Drillhole | | | | | |

Column headings also include: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.25 METRES; B Angle; DIP w.r.t. CORE AXIS; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY k, cm/s ($10^{-6}$, $10^{-5}$, $10^{-4}$, $10^{-3}$); Diametral Point Load Index (MPa); RMC -Q' AVG.

Depth scale: 5, 6, 7, 8, 9, 10, 11, 12, 13, 14



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: AB

Golder Associates



# RECORD OF BOREHOLE No B4- 3

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050318.1; E 237110.3
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY DA
DATUM Geodetic
DATE February 5, 2009
CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 192.3 | GROUND SURFACE | | | | | |
| 0.0 | Clayey silt to sand, containing organics (FILL) Loose Brown Moist to wet | 1 | AS | - | | |
| | | 2 | SS | 5 | ▽ (approx. Elev. 190.9) | |
| 190.8 / 1.5 | PEAT (Fibrous) Soft Black Wet | 3 | SS | 3 | | |
| 190.0 / 2.3 | CLAY Very soft Brown to grey Wet | 4 | SS | WH | | |
| | | 5 | SS | WH | | |
| 187.5 / 4.8 | Gravelly SAND, trace to some silt Very loose Grey Wet | 6A / 6B | SS | WH | | 0 9 35 56 |
| 187.0 / 5.3 | GNEISS (BEDROCK) Bedrock cored from 5.3 m depth to 8.5 m depth. For coring details refer to Record of Drillhole B4-3. | 1 | RC | REC 100% | | RQD = 100% |
| | | 2 | RC | REC 100% | | RQD = 100% |
| 183.8 / 8.5 | End of Borehole | | | | | |

Plotted data (elevation scale 192 to 184):
- Natural moisture content: sample 1 ≈ 27%; sample 3: 360.5; sample 5: 88.5, with plastic limit ≈ 22 and liquid limit 54; sample 6A: 64.2.
- Shear strength (field vane) near Elev. 188.5 and 188.1, sensitivity 5 and 3.

Notes:

1. Moved 1.0 m north and obtained Shelby tube at 3.8 m depth.

2. Water level at a depth of 1.4 m below ground surface (Elev. 190.9 m) upon completion of drilling.

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

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

PROJECT: 07-1191-0020

# RECORD OF DRILLHOLE: B4- 3



LOCATION: N 5050318.1 ;E 237110.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 5, 2009

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. DEPTH (m) | RUN No. | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|
| | | Refer to Previous Page | | | | |
| | NQ Coring 02/05/09 | GNEISS<br>FIne to medium grained<br>Fresh<br>Very strong<br>Pinkish grey | 187.0<br>5.3 | 1 | J, I, R | |
| 6 | | | | | | UCS = 154 MPa |
| 7 | | | | 2 | | |
| 8 | | | 183.8<br>8.5 | | J, I, R<br>J, I, R | |
| | | End of Drillhole | | | | |
| 9 | | | | | | |
| 10 | | | | | | |
| 11 | | | | | | |
| 12 | | | | | | |
| 13 | | | | | | |
| 14 | | | | | | |
| 15 | | | | | | |

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: AB

# RECORD OF BOREHOLE No B4- 4

 **METRIC**

PROJECT 07-1191-0020 | LOCATION N 5050314.8; E 237105.8 | ORIGINATED BY EHS

W.P. 5191-06-01 | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | COMPILED BY MR

DIST | HWY 69 | DATE February 3, 2009 | CHECKED BY AB

DATUM Geodetic

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 192.2 | SNOW SURFACE | | | | | |
| 0.0 | SNOW | | | | | |
| 191.9 / 0.3 | PEAT, trace to some sand, trace silt (Fibrous) Very soft to firm Black Moist to wet | 1 | AS | - | | |
| | | 2 | SS | 4 | ▽ | |
| 190.3 / 1.9 | CLAY Very soft Grey Wet | 3 | SS | 1 | | |
| | | 4 | TO | PH | | |
| 189.0 / 3.2 | GNEISS (BEDROCK) Bedrock cored from 3.2 m depth to 6.1 m depth. For coring details refer to Record of Drillhole B4-4. | 5 | SS | 20/0 | | |
| | | 1 | RC | REC 100% | | RQD = 95% |
| | | 2 | RC | REC 100% | | RQD = 100% |
| 186.1 / 6.1 | End of Borehole | | | | | |

Note:

1. Water level at a depth of 0.8 m below ground surface (Elev. 191.1 m) upon completion of drilling.

Elevation scale: 192, 191, 190, 189, 188, 187

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

Water content values plotted: 41.5 (sample 1); 56, 85.5 (sample 4)

UNIT WEIGHT γ kN/m³

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

PROJECT: 07-1191-0020

# RECORD OF DRILLHOLE: B4- 4

SHEET 1 OF 1

LOCATION: N 5050314.8 ;E 237105.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 3, 2009

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. DEPTH (m) | RUN No. | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|
| | | Refer to Previous Page | | | | |
| | NQ Coring 02/03/2009 | GNEISS<br>Fine to medium grained<br>Fresh<br>Very strong<br>Grey | 189.0<br>3.2 | 1 | J, I, R<br>J, I, R<br>J, I, R | UCS = 118 MPa |
| 4 | | | | | | |
| 5 | | | | 2 | J, I, R<br>J, I, R | |
| 6 | | | 186.1<br>6.1 | | | |
| | | End of Drillhole | | | | |
| 7 | | | | | | |
| 8 | | | | | | |
| 9 | | | | | | |
| 10 | | | | | | |
| 11 | | | | | | |
| 12 | | | | | | |
| 13 | | | | | | |

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: AB

Golder Associates

# RECORD OF BOREHOLE No B4- 5

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050319.8; E 237104.9 ORIGINATED BY EHS

DIST HWY 69 BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers COMPILED BY DA

DATUM Geodetic DATE January 29, 2009 CHECKED BY AB

| ELEV DEPTH | DESCRIPTION (SOIL PROFILE) | STRAT PLOT | NUMBER (SAMPLES) | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100) | PLASTIC LIMIT $W_P$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 192.1 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | PEAT (Fibrous) Very soft Black Moist | | 1 | AS | - | | 192 | | 52.6 | | |
| | | | 2 | SS | 2 | | 191 | | 52.3 | | |
| | | | 3 | SS | 1 | ▽ | | | 62.1 | | |
| 189.9 / 2.2 | CLAY Very Soft Grey to brown Wet | | 4 | SS | WR | | 190 | | | | |
| | Approximately 1.5 m of heave at 3.8 m depth. | | | | | | 189 | + 5; + 5 | | | |
| 188.0 | | | 5 | SS | 8/0.15 | | 188 | | 50; 66.2 | | |
| 4.1 | End of Borehole Spoon and Auger Refusal | | | | | | | | | | |

Note:

1. Augers dipping to the southeast upon refusal. Advanced spoon to a depth of 4.4 m, bending the spoon.

2. Water level at a depth of 1.8 m below ground surface (Elev. 190.3 m) upon completion of drilling.

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

Golder Associates

# RECORD OF BOREHOLE No B4- 6

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050342.9; E 237109.9
ORIGINATED BY ID
DIST
HWY 69
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY DA
DATUM Geodetic
DATE March 31, 2009
CHECKED BY AB

| ELEV / DEPTH (m) | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | SHEAR STRENGTH (kPa) | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|
| 193.2 | GROUND SURFACE | | | | | | | |
| 0.0 | ASPHALT | | | | | | | |
| 0.1 | Sand, trace to some silt (FILL) Very loose to compact Brown to grey Moist to wet | 1 | SS | 25 | | | | |
| | | 2 | SS | 20 | | | ○ | 0 91 (9) |
| | | | | | ∇ (Elev. 191.0) | | | |
| | | 3 | SS | 2 | | | | |
| | | 4 | SS | 1 | | | | |
| | | 5 | SS | 1 | | | ○ | 0 89 (11) |
| 188.7 / 4.5 | CLAY Soft Grey Wet | 6 | SS | WH | | | $w_P$ to ○ 58 / 72.2 | |
| | | | | | | + 3 (field vane); + 4 (field vane) | | |
| | | 7 | TO | WH | | | | |
| 186.0 / 7.2 | SAND and SILT, trace gravel, cobbles and boulders inferred / encountered at 9.1 m depth Compact to very dense Grey Wet | 8 | SS | 15 | | | | |
| | | 9 | SS | 45/0.15 | | | ○ | 2 46 52 0 |
| 183.3 / 9.9 | End of Borehole Auger Refusal | | | | | | | |

Notes:

1. Spoon refusal at a depth of 9.4 m below ground surface (Elev. 183.8 m)

2. Water level at a depth of 2.2 m below ground surface (Elev. 191.0 m) upon completion of drilling.

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

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 193–184.

Golder Associates



# RECORD OF BOREHOLE No B4-10

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050345.5; E 237098.3
ORIGINATED BY EHS
DIST ____ HWY 69
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
COMPILED BY DA
DATUM Geodetic
DATE January 30 and February 2, 2009
CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION (SOIL PROFILE) | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | SHEAR STRENGTH kPa / WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 190.9 | SNOW SURFACE | | | | | | |
| 0.0 | SNOW | | | | | | |
| 190.2 / 0.8 | Peat (FILL)<br>Sand, trace gravel, trace silt (FILL)<br>Very loose to loose<br>Brown<br>Wet | 1 | SS | 7 | 190 | | |
| | | 2 | SS | 2 | 189 | | 2 93 (5) |
| | | 3 | SS | WH | 188 | | |
| | | 4 | SS | WH | | | |
| 186.8 / 4.1 | CLAY<br>Firm<br>Brown<br>Wet | 5A / 5B | SS | 2 | 187 | 60 / 61.4 | |
| | | | | | 186 | +3, +3 | |
| 185.6 / 5.3 | SAND and SILT, some clay<br>Very loose<br>Brown<br>Wet | 6 | SS | WH | | | 0 53 34 13 |
| 184.9 / 6.0 | SAND, some gravel, some silt, cobbles and boulders inferred / encountered at 9.3 m depth<br>Loose to very dense<br>Grey<br>Wet | 7 | SS | 5 | 185 | | |
| | | | | | 184 | | |
| | Approximately 2.1 m of heave at 7.6 m depth. | 8 | SS | 11 | 183 | | 15 72 (13) |
| | | | | | 182 | | |
| | Switched to NW Casing at 9.3 m depth.<br>Approximately 1.2 m of heave at 9.7 m depth. | 9 | SS | 13/0.01 | 181 | | |
| | | | | | 180 | | |
| | Approximately 3.0 m of heave at 11.3 m depth. Bi-cone was used to clear casing. | 10 | SS | 51 | 179 | | |
| 178.7 / 12.2 | GNEISS (BEDROCK)<br>Bedrock cored from 12.2 m depth to 15.3 m depth.<br>For coring details refer to Record of Drillhole B4-10. | 1 | RC | REC 100% | 178 | | RQD = 92% |
| | | 2 | RC | REC 100% | 177 / 176 | | RQD = 38% |

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

Continued Next Page

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

# RECORD OF BOREHOLE No B4-10

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01

DIST HWY 69

DATUM Geodetic

LOCATION N 5050345.5; E 237098.3

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring

DATE January 30 and February 2, 2009

ORIGINATED BY EHS

COMPILED BY DA

CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | |
| 175.6 | | | 2 | RC | | | | | | | RQD = 38% |
| 15.3 | End of Borehole<br><br>Notes:<br><br>1. Water level at 0.1 m below ground surface (Elev. 190.1 m) upon completion of drilling.<br><br>2. Water level measured in piezometer at 0.7 m above ground surface (Elev. 190.9 m) on March 31, 2009. | | | | | | | | | | |



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

○ 3% STRAIN AT FAILURE

# RECORD OF DRILLHOLE: B4-10

PROJECT: 07-1191-0020

SHEET 1 OF 1

LOCATION: N 5050345.5 ;E 237098.3

INCLINATION: -90°  AZIMUTH: ---

DRILLING DATE: January 30 and February 2, 2009

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | NOTES / WATER LEVELS / INSTRUMENTATION |
|---|---|---|---|---|---|---|
| | | Refer to Previous page | 178.7 | | | |
| 13–15 | NQ Coring 01/30/09 and 02/02/09 | GNEISS<br>Fine to medium grained<br>Slightly weathered<br>Very strong<br>Grey | 12.2 | 1 | J, I, R<br>J, I, R<br>J, F, O, R<br>J, I, R<br>J, I, R | UCS = 112 MPa |
| | | | | 2 | J, I, R<br>J, I, R<br>J, I, R<br>J, S, T, R<br>J, I, R<br>J, P, L, R<br>J, I, R<br>J, I, R<br>J, I, R<br>J, I, R<br>J, I, R<br>J, I, R<br>J, I, R | |
| | | End of Drillhole | 175.6<br>15.3 | | | |

Depth scale: 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: AB

Golder Associates

# RECORD OF BOREHOLE No B4-11

 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 07-1191-0020 | | | |
| W.P. 5191-06-01 | LOCATION N 5050387.2; E 237101.2 | | ORIGINATED BY ID |
| DIST | HWY 69 | BOREHOLE TYPE N/A | COMPILED BY DA |
| DATUM Geodetic | DATE April 14, 2009 | | CHECKED BY AB |

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 197.6 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

Golder Associates

# RECORD OF BOREHOLE No B4-12

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050392.1; E 237100.4 ORIGINATED BY ID

DIST HWY 69 BOREHOLE TYPE N/A COMPILED BY DA

DATUM Geodetic DATE April 14, 2009 CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT $\gamma$ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 199.3 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

Golder Associates

# RECORD OF BOREHOLE No B4-13

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
DIST HWY 69
DATUM Geodetic

LOCATION N 5050388.8; E 237095.8
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
DATE April 15, 2009

ORIGINATED BY ID
COMPILED BY DA
CHECKED BY AB

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa | WATER CONTENT (%) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 192.2 | GROUND SURFACE | | | | | | | | | |
| 0.0 | Sand and gravel (FILL)<br>Compact<br>Brown<br>Moist | 1 | AS | - | | 192 | | | | |
| | | 2 | SS | 9/0.15 | | | | | | |
| 191.3<br>0.9 | GNEISS (BEDROCK)<br>Bedrock cored from 0.9 m depth to 4.1 m depth.<br>For coring details refer to Record of Drillhole B4-13. | 1 | RC | REC 100% | | 191 | | | | RQD = 70% |
| | | 2 | RC | REC 100% | | 190 | | | | RQD = 97% |
| | | 3 | RC | REC 100% | | 189 | | | | RQD = 100% |
| 188.1<br>4.1 | End of Borehole<br>Note:<br>1. Borehole dry upon completion of drilling. | | | | | | | | | |

Plot legend: SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; scales 20 40 60 80 100. PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 10 20 30.

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

PROJECT: 07-1191-0020

# RECORD OF DRILLHOLE: B4-13

SHEET 1 OF 1

LOCATION: N 5050388.8 ;E 237095.8

DRILLING DATE: April 15, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. DEPTH (m) | RUN No. | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|
| | | Refer to Previous Page | | | | |
| 1 | NQ Coring 04/05/09 | GNEISS<br>Fine to medium grained<br>Slightly weathered<br>Grey<br><br>Numerous joints from 0.9 m to 1.2 m depth. | 191.3<br>0.9 | 1 | J, I, R<br>J, I, R<br>J, I, R | |
| 2 | | | | 2 | J, I, R<br>J, I, R<br>J, I, R | |
| 3 | | | | 3 | J, I, R | |
| 4 | | End of Drillhole | 188.1<br>4.1 | | | |
| 5 | | | | | | |
| 6 | | | | | | |
| 7 | | | | | | |
| 8 | | | | | | |
| 9 | | | | | | |
| 10 | | | | | | |

SUD-RCK 07-1191-0020 B4 BH LOGS METRIC.GPJ GAL-MISS.GDT 13/03/12 DATA INPUT:

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: ID
CHECKED: AB

Golder Associates



# RECORD OF BOREHOLE No B4-14

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01

DIST ______ HWY 69

DATUM Geodetic

LOCATION N 5050385.5; E 237091.3

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring

DATE March 31, 2009

ORIGINATED BY ID

COMPILED BY DA

CHECKED BY AB

| ELEV DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|---|---|
| 192.8 | GROUND SURFACE | | | | | | |
| 0.0 | ASPHALT | | | | | | |
| 0.2 | Sand and gravel to sand (FILL)<br>Dense to very dense<br>Brown<br>Moist to wet | 1 | SS | 98 | | 192 | |
| | | 2 | SS | 46 | | 191 | |
| | | 3 | SS | 10/0.08 | ∇ | | |
| 190.2<br>2.6 | GNEISS (BEDROCK)<br>Bedrock was cored from 2.6 m to 5.7 m depth.<br>For coring details refer to Record of Drillhole B4-14. | 1 | RC | REC 100% | | 190<br>189 | RQD = 89% |
| | | 2 | RC | REC 100% | | 188 | RQD = 94% |
| 187.1<br>5.7 | End of Borehole<br>Notes:<br>1. Water level at a depth of 2.3 m below ground surface (Elev. 190.5 m) upon completion of drilling. | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

GR SA SI CL

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

# RECORD OF DRILLHOLE: B4-14

PROJECT: 07-1191-0020
SHEET 1 OF 1

LOCATION: N 5050385.5 ;E 237091.3
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 31, 2009
DRILL RIG: CME 55
DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate
BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | NOTES / WATER LEVELS / INSTRUMENTATION |
|---|---|---|---|---|---|---|
| | | Refer to Previous Page | 190.2 | | | |
| 3–5 | NQ Coring 03/31/09 | GNEISS<br>Fine to coarse grained<br>Slightly weathered<br>Strong<br>Grey | 2.6 | 1 | J, I, R<br>J, I, R | |
| | | | | 2 | J, I, R<br>J, I, R<br>J, I, R | UCS = 83 MPa |
| | | | 187.1 | | | |
| | | End of Drillhole | 5.7 | | | |

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % 80 60 40 20; FRACT. INDEX METRES 5 10 15 20; DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja; Jn); HYDRAULIC CONDUCTIVITY k, cm/s $10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.; NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 3 4 5 6 7 8 9 10 11 12

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: ID
CHECKED: AB

Golder Associates

# RECORD OF BOREHOLE No B4-15

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050390.4; E 237090.5
ORIGINATED BY ID
DIST HWY 69
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY DA
DATUM Geodetic
DATE April 14, 2009
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION (SOIL PROFILE) | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100) | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$ — WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 192.7 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | ASPHALT | | | | | | | | | | |
| 0.1 | Sand and gravel to sand (FILL)<br>Dense<br>Brown<br>Moist | | | | | | 192 | | | | |
| | | | 1 | SS | 39 | | | | | | |
| | | | 2 | SS | 15/0.08 | | | | | | |
| 191.1<br>1.6 | End of Borehole<br>Auger Refusal<br><br>Note:<br><br>1. Borehole dry upon completion of drilling. | | | | | | | | | | |

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

SUD-MTO 001 07-1191-0020 B4 BH LOGS METRIC.GPJ GAL-MISS.GDT 21/02/12 DATA INPUT:

Golder Associates

# RECORD OF BOREHOLE No B4-16

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050413.8; E 237096.6
ORIGINATED BY EHS
DIST
HWY 69
BOREHOLE TYPE N/A
COMPILED BY DA
DATUM Geodetic
DATE March 10, 2009
CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$ / WATER CONTENT (%) 10 20 30 | UNIT WEIGHT $\gamma$ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 203.8 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

SUD-MTO 001 07-1191-0020 B4 BH LOGS METRIC.GPJ GAL-MISS.GDT 21/02/12 DATA INPUT:

Golder Associates

# RECORD OF BOREHOLE No B4-17

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050418.7; E 237095.7 ORIGINATED BY EHS

DIST HWY 69 BOREHOLE TYPE N/A COMPILED BY MR

DATUM Geodetic DATE March 10, 2009 CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 205.0 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

Golder Associates



# RECORD OF BOREHOLE No B4-18

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050415.4; E 237091.2
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE NQ Coring
COMPILED BY DA
DATUM Geodetic
DATE March 9 and 10, 2009
CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | REMARKS (RQD) |
|---|---|---|---|---|---|
| 203.9 | GROUND SURFACE | | | | |
| 0.0 | ORGANICS | | | | |
| 0.2 | Brown, Moist | | | | |
| | GNEISS (BEDROCK)<br>Bedrock cored from 0.2 m to 10.7 m depth.<br>For coring details refer to Record of Drillhole B4-18. | 1 | RC | REC 100% | RQD = 95% |
| | | 2 | RC | REC 100% | RQD = 96% |
| | | 3 | RC | REC 100% | RQD = 100% |
| | | 4 | RC | REC 100% | RQD = 100% |
| | | 5 | RC | REC 100% | RQD = 100% |
| 195.5 / 8.4 | PEGMATITE (BEDROCK) | 6 | RC | REC 100% | RQD = 100% |
| | | 7 | RC | REC 100% | RQD = 100% |
| 193.2 / 10.7 | End of Borehole<br>Note:<br>1. Borehole dry upon completion of drililng. | | | | |

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (203, 202, 201, 200, 199, 198, 197, 196, 195, 194); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m$^3$; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

# RECORD OF DRILLHOLE: B4-18

PROJECT: 07-1191-0020

SHEET 1 OF 2

LOCATION: N 5050415.4 ;E 237091.2

DRILLING DATE: March 9 and 10, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

Columns: DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY (TOTAL CORE %, SOLID CORE %) | R.Q.D. % | FRACT. INDEX METRES | DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn) | HYDRAULIC CONDUCTIVITY k, cm/s ($10^{-6}$, $10^{-5}$, $10^{-4}$, $10^{-3}$) | Diametral Point Load Index (MPa) | RMC -Q' AVG. | NOTES WATER LEVELS INSTRUMENTATION

Drilling record: NQ Coring, 03/09/09 and 04/09/09

| Depth (m) | Description | Elev. / Depth (m) | Run No. | Discontinuity type and surface description | Notes |
|---|---|---|---|---|---|
| | Refer to Previous Page | 203.7 | | | |
| 0.2 | GNEISS<br>Fine to coarse grained<br>Sound<br>Very strong<br>Grey | 0.2 | 1 | J, I, R<br>J, I, R<br>J, F, O, R<br>J, F, O, R | |
| | | | 2 | J, I, R<br>J, I, R<br>J, F, O, R<br>J, F, O, R | |
| | | | 3 | J, I, R<br>J, I, R | |
| | | | 4 | | |
| | | | 5 | | |
| | | | 6 | J, I, R | |
| 8.4 | PEGMATITE<br>Fine to coarse grained<br>Sound | 195.5<br>8.4 | 6 | J, I, R<br>J, I, R | |
| | | | 7 | | UCS = 101 MPa |

*CONTINUED NEXT PAGE*

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B4 BH LOGS METRIC.GPJ GAL-MISS.GDT 13/03/12 DATA INPUT:

PROJECT: 07-1191-0020

# RECORD OF DRILLHOLE: B4-18



LOCATION: N 5050415.4 ;E 237091.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 9 and 10, 2009

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | FLUSH COLOUR % RETURN | RECOVERY TOTAL CORE % | RECOVERY SOLID CORE % | R.Q.D. % | FRACT. INDEX METRES | DISCONTINUITY DATA B Angle | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY k, cm/s | Diametral Point Load Index (MPa) | RMC -Q' AVG. | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | | | | | | | | | |
| | NQ Coring | PEGMATITE<br>Fine to coarse grained<br>Sound | | 193.2 | 7 | | | | | | | | | | | | | | | |
| | | End of Drillhole | | 10.7 | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | |
| 16 | | | | | | | | | | | | | | | | | | | | |
| 17 | | | | | | | | | | | | | | | | | | | | |
| 18 | | | | | | | | | | | | | | | | | | | | |
| 19 | | | | | | | | | | | | | | | | | | | | |
| 20 | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: AB

# RECORD OF BOREHOLE No B4-19

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01

LOCATION N 5050412.1; E 237086.7

ORIGINATED BY EHS

DIST HWY 69

BOREHOLE TYPE N/A

COMPILED BY DA

DATUM Geodetic

DATE March 10, 2009

CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100) | PLASTIC LIMIT $W_P$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 202.9 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

Golder Associates

# RECORD OF BOREHOLE No B4-20

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050417.0; E 237085.9 ORIGINATED BY EHS

DIST HWY 69 BOREHOLE TYPE N/A COMPILED BY DA

DATUM Geodetic DATE March 10, 2009 CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT $\gamma$ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 203.2 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock | | | | | | | | | | |

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

Golder Associates

# RECORD OF BOREHOLE No B4-21

 **METRIC**

PROJECT 07-1191-0020
W.P. 5191-06-01
LOCATION N 5050303.3; E 237112.8
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY DA
DATUM Geodetic
DATE January 29, 2009
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS |
|---|---|---|---|---|---|---|---|
| 192.0 | GROUND SURFACE | | | | | | |
| 0.0 | ORGANICS (FILL) | | | | | | |
| 0.2 | Brown, Moist | | | | | | |
| 191.2 | Silty sand, trace gravel, trace clay (FILL) | 1 | AS | - | ▽ | 191 | |
| 0.9 | Brown, Wet | | | | | | |
| | PEAT, Black, Wet | 2 | SS | WH | | 190 | |
| | SILTY CLAY to CLAY, clayey silt seam at top of the deposit, Very soft, Grey to brown, Wet | 3 | SS | WH | | | |
| 189.3 2.7 | SAND, trace to some gravel, trace to some silt, Very loose, Grey, Wet | 4 | SS | 40/0.08 | | 189 | |
| 188.8 3.2 | End of Borehole, Spoon and Auger Refusal | | | | | | |

Notes:

1. Water level at a depth of 1.1 m below ground surface (Elev. 190.9 m) upon completion of drilling.

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

Plotted value: 53

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

SUD-MTO 001 07-1191-0020 B4 BH LOGS METRIC.GPJ GAL-MISS.GDT 21/02/12 DATA INPUT:

# RECORD OF BOREHOLE No B4-22

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 LOCATION N 5050430.2; E 237088.7 ORIGINATED BY EHS

DIST HWY 69 BOREHOLE TYPE N/A COMPILED BY DA

DATUM Geodetic DATE March 10, 2009 CHECKED BY AB

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100) | PLASTIC LIMIT $W_P$ / NATURAL MOISTURE CONTENT $W$ / LIQUID LIMIT $W_L$; WATER CONTENT (%) (10, 20, 30) | UNIT WEIGHT $\gamma$ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 205.2 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Exposed Bedrock<br><br>Note:<br><br>1. Less than about 75 mm of organics overlying bedrock. | | | | | | | | | | |

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

# RECORD OF BOREHOLE No B4-23

 **METRIC**

PROJECT 07-1191-0020

W.P. 5191-06-01 | LOCATION N 5050293.9; E 237135.8 | ORIGINATED BY ID

DIST ___ HWY 69 | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | COMPILED BY JJL

DATUM Geodetic | DATE April 7, 2010 | CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100) | PLASTIC LIMIT $W_P$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 193.5 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Sand and gravel to sand (FILL) | | | | | | | | | | |
| 0.2 | Very loose<br>Grey<br>Moist | | 1 | SS | 2 | | 193 | | | | |
| | SAND<br>Loose<br>Brown<br>Moist | | 2 | SS | 5 | | | | ○ (approx. 10) | | 1 89 (10) |
| | | | 3 | SS | 5 | ∇ | 192 | | | | |
| 191.2<br>2.3 | End of Borehole<br>Auger Refusal | | | | | | | | | | |

Notes:

1. Water level at a depth of 2.2 m below ground surface (Elev. 191.3 m) upon completion of drilling.

2. Moved 1.5 m south and encountered auger refusal at 2.3 m depth.

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

# APPENDIX B

## Laboratory Test Results

**TABLE B-1**
**REFUSAL/BEDROCK ELEVATIONS**
**HIGHWAY 529 OVERPASS NBL**
**GWP 5005-08-00**

| Borehole | Depth to Refusal/Bedrock Surface* (m) | Refusal/Bedrock Surface Elevation (m) | Comments |
|---|---|---|---|
| B4-1 | 4.2 | 187.6 | Spoon and Auger Refusal |
| B4-2 | 3.9 | 186.8 | Bedrock Surface |
| B4-3 | 5.3 | 187.0 | Bedrock Surface |
| B4-4 | 3.2 | 189.0 | Bedrock Surface |
| B4-5 | 4.1 | 188.0 | Spoon and Auger Refusal |
| B4-6 | 9.9 | 183.3 | Auger Refusal |
| B4-10 | 12.2 | 178.7 | Bedrock Surface |
| B4-11 | G.S.** | 197.6 | Exposed Bedrock |
| B4-12 | G.S.** | 199.3 | Exposed Bedrock |
| B4-13 | 0.9 | 191.3 | Bedrock Surface |
| B4-14 | 2.6 | 190.2 | Bedrock Surface |
| B4-15 | 1.6 | 191.1 | Auger Refusal |
| B4-16 | G.S.** | 203.8 | Exposed Bedrock |
| B4-17 | G.S.** | 205.0 | Exposed Bedrock |
| B4-18 | 0.2 | 203.7 | Bedrock Surface |
| B4-19 | G.S.** | 202.9 | Exposed Bedrock |
| B4-20 | G.S.** | 203.2 | Exposed Bedrock |
| B4-21 | 3.2 | 188.8 | Spoon and Auger Refusal |
| B4-22 | G.S.** | 205.2 | Exposed Bedrock |
| B4-23 | 2.3 | 191.2 | Auger Refusal |

* Below bottom of snow where encountered.
** G.S. denotes bedrock was encountered at ground surface.

Compiled by: EC
Checked by: AB
Reviewed by: JMAC

**TABLE B-2**
**UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS**
**HIGHWAY 529 OVERPASS NBL**
**GWP 5005-08-00**

| Borehole Number | Sample Depth* (m) | Sample Elevation (m) | Rock Type | Core Diameter (mm) | Uniaxial Compressive Strength (MPa) |
|---|---|---|---|---|---|
| B4-2 | 5.4 | 184.9 | Gneiss | 48 | 138 |
| B4-3 | 6.0 | 186.3 | Gneiss | 48 | 154 |
| B4-4 | 3.1 | 188.5 | Gneiss | 48 | 118 |
| B4-10 | 11.4 | 178.1 | Gneiss | 48 | 112 |
| B4-14 | 4.7 | 188.1 | Gneiss | 48 | 83 |
| B4-18 | 10.0 | 193.9 | Gneiss | 48 | 101 |

* Below bottom of snow where encountered

Compiled by: EC
Checked by: AB
Reviewed by: JMAC

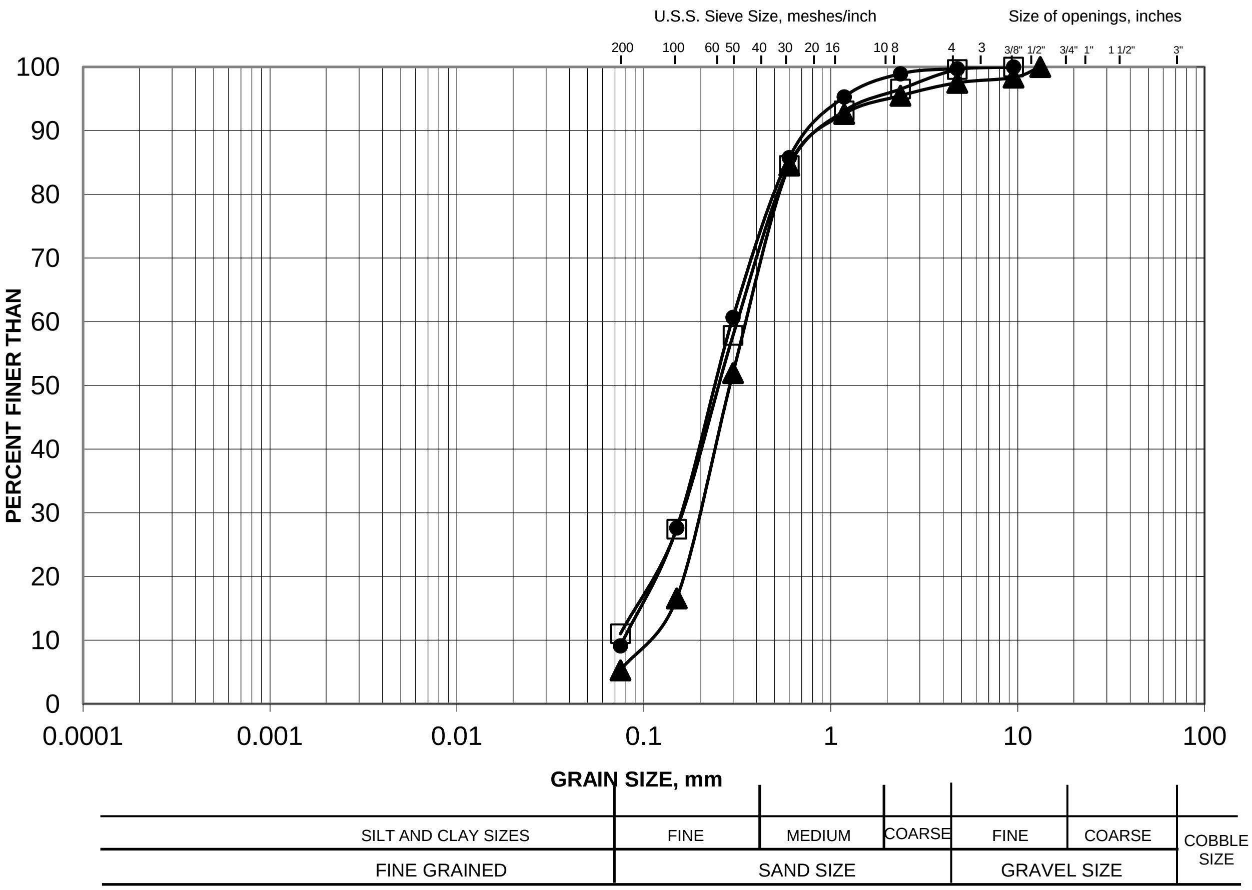
**TABLE B-3**
**POINT LOAD STRENGTH TEST RESULTS**
**HIGHWAY 529 OVERPASS NBL**
**GWP 5005-08-00**

| Borehole Number | Sample Depth [1] (m) | Sample Elevation (m) | Rock Type | Test Type [2] | Core Diameter (mm) | Ram Pressure (MPa) | Load (kN) | $I_s$ Diametral [2] (MPa) | $I_s$ 50 mm [2] (MPa) | Approximate UCS [2] (MPa) |
|---|---|---|---|---|---|---|---|---|---|---|
| B4-2 | 4.5 | 186.2 | Gneiss | D | 48 | 11.1 | 0.010 | 4.64 | 4.53 | 104 |
| B4-2 | 4.8 | 185.9 | Gneiss | D | 48 | 7.5 | 0.007 | 3.15 | 3.08 | 71 |
| B4-2 | 6.2 | 184.5 | Gneiss | D | 48 | 11.6 | 0.011 | 4.87 | 4.76 | 109 |
| B4-3 | 5.6 | 186.7 | Gneiss | D | 48 | 13.9 | 0.013 | 5.84 | 5.71 | 131 |
| B4-3 | 8.2 | 184.1 | Gneiss | D | 48 | 14.6 | 0.014 | 6.14 | 6.00 | 138 |
| B4-4 | 3.0 | 188.9 | Gneiss | D | 48 | 12.8 | 0.012 | 5.41 | 5.28 | 122 |
| B4-4 | 4.6 | 187.3 | Gneiss | D | 48 | 15.9 | 0.015 | 6.65 | 6.50 | 150 |
| B4-4 | 5.3 | 186.6 | Gneiss | D | 48 | 15.0 | 0.014 | 6.28 | 6.14 | 141 |
| B4-10 | 12.7 | 177.5 | Gneiss | D | 48 | 9.2 | 0.009 | 3.88 | 3.79 | 87 |
| B4-10 | 14.5 | 175.7 | Gneiss | D | 48 | 10.2 | 0.010 | 4.28 | 4.18 | 96 |
| B4-14 | 3.4 | 189.4 | Gneiss | D | 48 | 13.7 | 0.013 | 5.76 | 5.63 | 130 |
| B4-14 | 4.2 | 188.6 | Gneiss | D | 48 | 12.6 | 0.012 | 5.29 | 5.16 | 119 |
| B4-14 | 5.2 | 187.6 | Gneiss | D | 48 | 12.1 | 0.011 | 5.07 | 4.95 | 114 |
| B4-18 | 8.6 | 195.3 | Gneiss | D | 48 | 11.5 | 0.011 | 4.83 | 4.72 | 109 |

**NOTES:**
1. Depths are given below the ground surface at the borehole location (bottom of snow where encountered).
2. Where: D = Diametral test;
   $I_s$ Diametral = Uncorrected point load strength;
   $I_s$ 50 mm = Corrected point load strength; and
   UCS = Uniaxial compressive strength = $I_s$ 50 mm x K. A K value of 23 has been used, based on correlation with UCS for this site ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock Mech. Sci. and Geomechanical Abst., Vol 22, No. 2, 1985, pp. 53-60.
   K = Conversion factor uniaxial compressive strength and corrected point load strength.

Compiled by: EC
Checked by: AB
Reviewed by: JMAC

| GRAIN SIZE DISTRIBUTION<br>Sand (FILL) | FIGURE<br>B-1 |
|---|---|




LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|---|---|---|---|
| ●— | B4-6 | 2 | 191.4 |
| □— | B4-6 | 5 | 189.1 |
| ▲— | B4-10 | 2 | 189.1 |

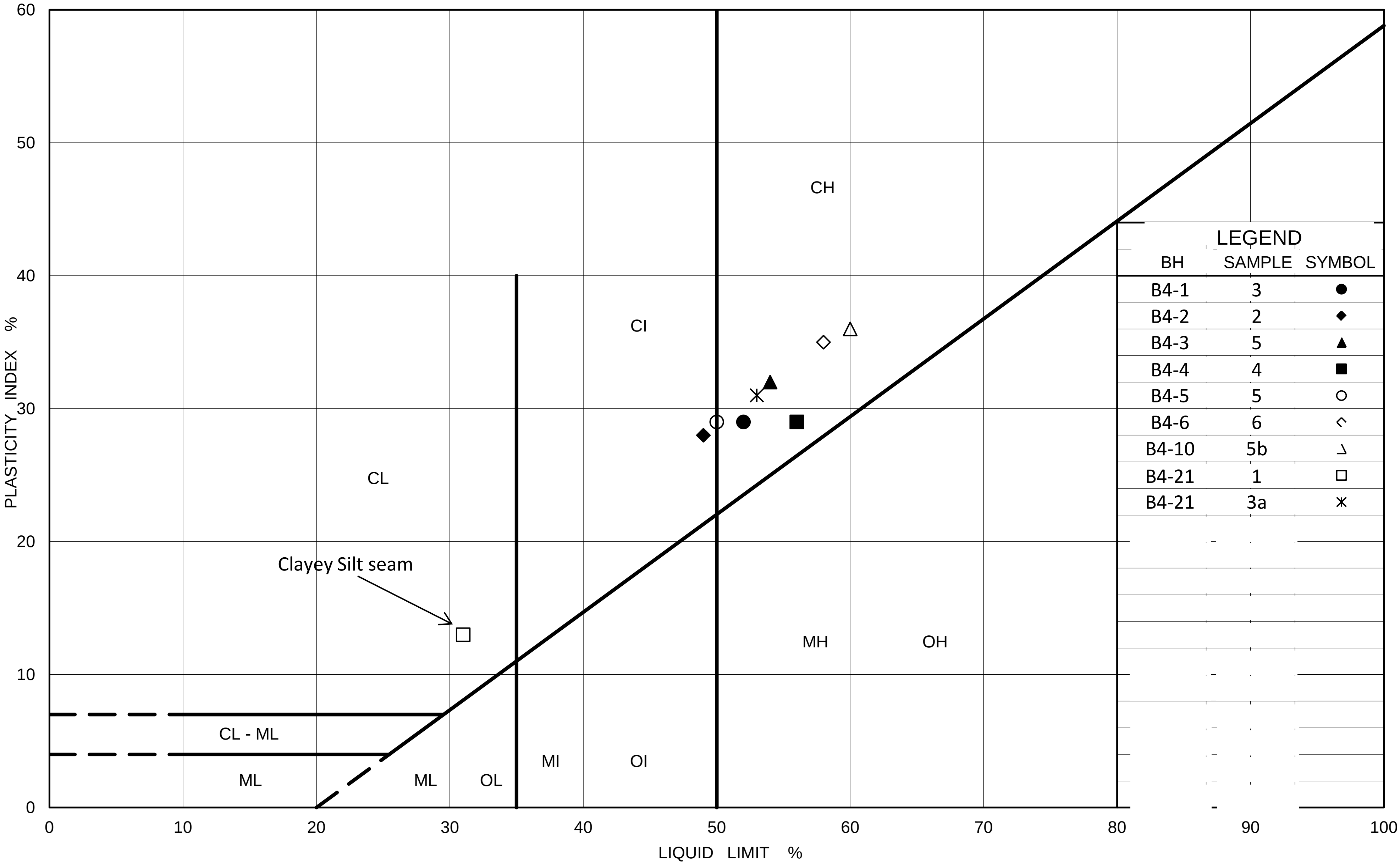
Project Number: 07-1191-0020-B4

Checked By: AB

**Golder Associates**

Date: March 2012

Oct 75, FF-S-21




Ministry of Transportation
Ontario

# PLASTICITY CHART
## Silty Clay to Clay

FIG No. B-2

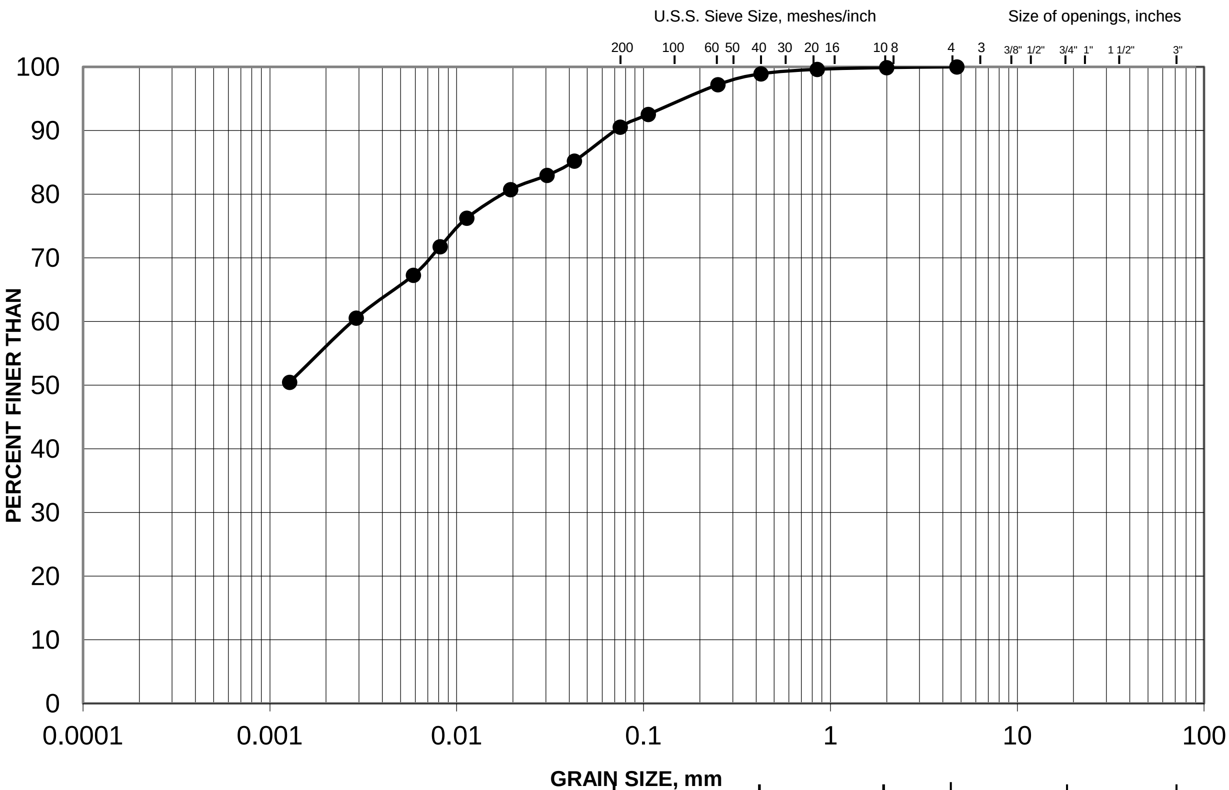
Project No. 07-1191-0020-B4

Checked By: AB

# GRAIN SIZE DISTRIBUTION

## Clay

**FIGURE B-3**




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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|---|---|---|---|
| —●— | B4-3 | 6a | 187.5 |

Project Number: 07-1191-0020-B4

Checked By: AB

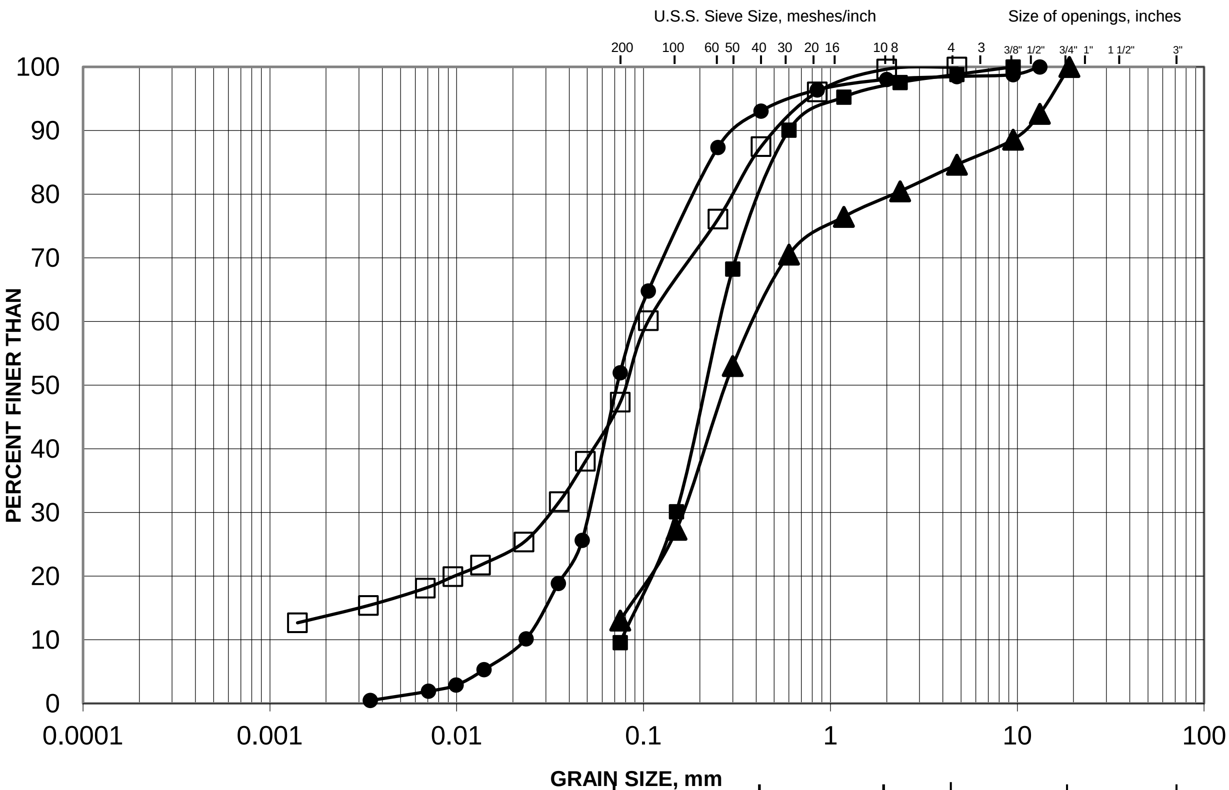
Golder Associates

Date: March 2012

# GRAIN SIZE DISTRIBUTION

## Sand to Sand and Silt

**FIGURE B-4**




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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|---|---|---|---|
| ● | B4-6 | 9 | 183.9 |
| □ | B4-10 | 6 | 185.3 |
| ▲ | B4-10 | 8 | 183.0 |
| ■ | B4-23 | 2 | 192.4 |

Project Number: 07-1191-0020-B4

Checked By: AB

**Golder Associates**

Date: March 2012

# APPENDIX C

## Non-Standard Special Provisions and Operational Constraints

**MASS CONCRETE – Item No.**

Non-Standard Special Provision

## Scope of Work

The scope of work for the above noted tender item includes the supply and placement of mass concrete under the south abutment, north pier and north abutment where applicable.

## Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

## Basis of Payment

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

**DOWELS INTO ROCK – Item No.**

---

Non-Standard Special Provision

---

**Scope of Work**

Work under this item is for the placement and field testing of dowels into rock.

**Construction**

Dowels into rock shall be constructed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS 1440 (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

For dowels into rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water; otherwise, a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

**Rock Dowel Testing**

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

**Performance Tests**

The following table summarizes the number of dowels into rock where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

| Bridge | Foundation | Number of Dowels for Performance Testing |
|---|---|---|
| Highway 529 Overpass NBL | South Abutment, North Pier, North Abutment | 2 per foundation |

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

**DOWELS INTO ROCK – Item No.**

---

Non-Standard Special Provision

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| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Cycle-Step | 1-1 | 1-2 | 1-3 | 2-1 | 2-2 | 2-3 | 2-4 |
| % Design Load | 50 | 75 | 25 | 50 | 75 | 100 | 25 |

| | | | | | |
|---|---|---|---|---|---|
| Cycle-Step | 3-1 | 3-2 | 3-3 | 3-4 | 3-5 |
| % Design Load | 50 | 75 | 100 | 110 | 25 |

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.025 mm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 3 additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

**Basis of Payment**

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

**ROCK POINTS - Item No.**

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Non-Standard Special Provision

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As part of the work under the above tender item, the Contractor shall supply Titus "Rock Injector Design" Pile Points or equivalent on HP 310x110 as applicable. Piles will be driven to bedrock.

**References**

OPSS 906 – Structural Steel
SP903S01

**Materials**

The pile points shall be of the following:

| **Product** | **Manufacturer** |
|---|---|
| HPP-R-12 | Titus Steel Company Ltd.<br>6767 Invader Crescent<br>Mississauga, Ontario<br>Tel. 905-564-2446 |

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

**Basis of Payment**

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

END OF SECTION

**H-PILES – HP310 X 110 - Item No.**

Non-Standard Special Provision

### 903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules.

The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unrealistic excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

**GRANULAR ‘B’ TYPE II – Item No.**

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Non-Standard Special Provision

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At the south abutment, the Contactor shall use SP110 S13 (Aggregates) Granular ‘B’ Type II with maximum particle size of 75 mm as sub-excavation backfill to facilitate pile driving.

## OPERATIONAL CONSTRAINT – OBSTRUCTIONS

At the Highway 529 Overpass NBL structure, cobbles and boulders were noted within the cohesionless soils at the boreholes advanced at the location of the south pier. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for piling for deep foundations through these materials.

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.


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