Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone 905-567-4444
Fax 905-567-6561

REPORT ON

DETAILED
FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 69 / HIGHWAY 559 UNDERPASS
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559
PARRY SOUND, ONTARIO
G.W.P 335-00-00
MINISTRY OF TRANSPORTATION, ONTARIO

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9

GEOCRES No. 41H-49

DISTRIBUTION

3	Copies -	Ministry of Transportation, Ontario, North Bay, Ontario (Northeastern Region)
1	Copy -	Ministry of Transportation, Ontario, Downsview, Ontario (Pavement and Foundation Section)
2	Copies -	URS Canada Inc. Markham, Ontario
2	Copies -	Golder Associates Ltd., Mississauga, Ontario

November 2005

03-1111-028-1

TABLE OF CONTENTS

SECTION **PAGE**

LIST OF TABLES

LIST OF FIGURES

LIST OF DRAWINGS

LIST OF APPENDICES

PART A

**DETAILED
FOUNDATION INVESTIGATION REPORT
HIGHWAY 69 / HIGHWAY 559 UNDERPASS
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559
PARRY SOUND, ONTARIO
G.W.P 335-00-00
MINISTRY OF TRANSPORTATION, ONTARIO**

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF DRAWINGS

LIST OF APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a detailed foundation investigation as part of the detailed design for the new Highway 69 / Highway 559 two-span underpass structure. The proposed work is part of the detailed design for the four-laning of Highway 69 and re-alignment of Highway 559 north of Nobel, Ontario including the construction of associated new highway on- and off-ramps, access and service roads, bridges and overhead truss sign structures. The general location of the Highway 69 and Highway 559 alignments are shown on the Site Location Map on Figure 1.

The terms of reference for the scope of work are outlined in Golder's proposal P31-1270 dated July 2003 that forms part of the Consultant's Agreement (Number P.O.5005-A-000320) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated October 2003. The General Arrangement (GA) Drawing for the proposed underpass structure at the new interchange of Highway 69 and Highway 559 was provided to Golder by URS on January 27, 2005.

This report addresses the investigation for the Highway 69 / Highway 559 underpass structure and the associated approach embankments only. Separate reports detail the foundation investigations for the related swamp crossings, high fill areas, other bridge structures and overhead truss sign structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure, including the associated approach embankments, by borehole drilling, rock coring, in-situ testing and laboratory testing on selected samples. The boreholes for the current investigation were located in the field by Callon Dietz Incorporated (Callon Dietz), a professional surveying company retained by URS. The investigated area is shown in plan on Drawing 1A.

2.0 SITE DESCRIPTION

The proposed Highway 69 / Highway 559 underpass structure is located immediately east of the existing Highway 69 alignment (north of Nobel, Ontario) at about 1.7 km north of existing Highway 559 (as shown on Figure 1).

In general, the topography in the area of the overall project site consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed structure and associated west approach embankment are to be situated on a moderately treed, topographic high with bedrock outcrops exposed at ground surface. The east approach embankment is to be located within a lower lying, more heavily wooded area of the site where bedrock exists at relatively shallow depth. The ground surface within the limits of the proposed structure and approach embankment areas generally lies between about Elevation 216 m (to the west) and 203 m (to the east), referenced to Geodetic Datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed Highway 69 / Highway 559 underpass structure investigation was carried out between September 21 and September 27, 2004 during which time ten (10) sampled boreholes (559-5 to 559-12, 559-15 and 559-17), five (5) shallow hand excavations (559-1, 559-2, 559-4, 559-14 and 559-16) and two (2) probe holes (559-3 and 559-13) were put down at the site. Fifteen (15) of the investigated locations were advanced at the proposed locations of the east abutment, central pier and west abutment footings (five per foundation element - including one at each corner of the foundation units and one in the central portion of each foundation unit) and one was advanced to refusal at each of the east and west approach embankments. All of the investigated locations were advanced to refusal on inferred bedrock. At each abutment and at the central pier, bedrock coring was carried out at three (3) of the investigated locations to a minimum depth of 3 m.

The field investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Where the bedrock surface was inferred to be relatively shallow, hand excavations or probe holes were advanced instead of boreholes in order to confirm the depth to bedrock while minimizing drill rig set-up time. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced to auger and/or sampler refusal (i.e. inferred bedrock) which occurred at depths ranging from ground surface (i.e. bedrock outcrop) to 2.2 m below the existing ground surface (not including rock coring). At investigated locations 559-2, 559-4, 559-6, 559-7, 559-9, 559-11, 559-12, 559-14 and 559-16, located within the footprints of the proposed foundation units, the depth of investigation was further advanced by coring into the bedrock about 3.0 m to 3.8 m. The groundwater level in the open boreholes / drillholes was observed throughout drilling operations. All boreholes were abandoned in accordance with O. Reg. 128 (amendment to O. Reg. 903).

The field work was supervised throughout by members of our engineering and technical staff, who confirmed the investigated locations, arranged for the clearance of underground service locations, supervised 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 Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing such as water content and grain size distribution were carried out on samples of the overburden soils. Strength testing such as point load index were carried out on specimens from the rock core.

All investigated locations were located in the field by Callon Dietz prior to drilling operations. The surveying of the elevations of the as-drilled boreholes, hand excavations and probe holes was carried out by members of our engineering staff, referenced to benchmark geodetic elevations provided by URS. The investigated locations and ground surface elevations are shown on Drawing 1A.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Georgian Bay Fringe. The Georgian Bay Fringe borders Georgian Bay as a broad belt characterized by shallow soil and bare bedrock knobs and ridges (The Physiography of Southern Ontario; Third Edition) however; Quaternary deposits of lacustrine and fluvial origin together with more recent swamp sediments have been accumulated between the bedrock ridges and, consequently, the overburden thickness and bedrock surface can be variable. The bedrock in the area are typically highly deformed gneisses and migmatites of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Geology of Ontario; OGS Special Volume 4). Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks covered only in a few places by the flat-lying Palaeozoic bedrock strata.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil and groundwater conditions as encountered in the boreholes, hand excavations and probe holes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets following the text of this report. The results from the laboratory testing are provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the investigated locations.

The inferred soil stratigraphy as encountered at the investigated locations of the proposed Highway 69 / Highway 559 underpass structure are shown on Drawings 1A and 1B.

In general, the subsoils at the structure site consist of a surficial layer of topsoil and/or leaf litter underlain by a thin deposit of silty sand to sandy silt subsequently underlain by bedrock. The total overburden thickness ranges from no cover (i.e. bedrock outcrops present at ground surface) to about 2.2 m below ground surface. All of the boreholes, hand excavations and probe holes were terminated at the inferred bedrock surface; with the exception of nine (9) investigated locations at foundation areas which were cored at least three metres into the bedrock.

In the area of the west approach embankment (adjacent to the existing Highway 69), west abutment and central pier, the depth of the bedrock ranges from about 0.0 m (exposed at ground surface) to 2.2 m below the existing ground surface, but typically less than 1.0 m depth. The overburden is thickest in the area of the central pier.

In the area of the east abutment and the east approach embankment, the depth to bedrock ranges from about 0.0 m (exposed at the ground surface) to 0.9 m below the existing ground surface.

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

4.2.1 Topsoil

A layer of topsoil was encountered at ground surface at all investigated locations where bedrock was not outcropping. The surface of the topsoil (i.e. ground surface) ranged between Elevations 215.5 m and 205.9 m and the thickness ranged between 0.1 m and 0.2 m.

4.2.2 Silty Sand to Sandy Silt

A silty sand to sandy silt deposit containing trace to some gravel, trace clay, organics and rootlets was encountered below the topsoil at the majority of the investigated locations where bedrock was not outcropping at ground surface. The top of this deposit ranged from Elevation 215.3 m to 205.7 m, typically following the contours of the existing ground surface, and the thickness ranged from about 0.1 m to 2.0 m. The bottom of this deposit was defined by refusal to further auger advancement or sampler penetration and was confirmed by rock coring at select locations.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 1 blow to greater than 50 blows per 0.3 m of penetration. Higher blow counts are attributed to the existence of weathered rock fragments at the interface of the bottom of the deposit and the bedrock surface. The 'N' values indicate a very loose to very dense relative density within the deposit.

The natural water content measured on samples of this deposit ranged between 1 percent and 13 percent, with an average of about 7 percent

Grain size distributions for two (2) samples from this deposit are shown on Figure A-1 of Appendix A.

4.2.3 Bedrock

Bedrock was encountered and cored at investigated locations 559-2, 559-4, 559-6, 559-7, 559-9, 559-11, 559-12, 559-14 and 559-16. The presence of bedrock was confirmed by hand excavations or probe holes and was inferred from refusal to further drilling or sampler advancement at the remaining investigated locations. The surface of the bedrock varies from ground surface (i.e. bedrock outcrop) to a depth of about 2.2 m. At the investigated locations, the bedrock surface ranges between about Elevation 214.8 m at the west end of the site to Elevation 205.0 m at the east end of the site.

The bedrock samples are described as fresh, light grey to pink, medium to coarse grained, non-porous to slightly porous granitic gneiss containing near horizontal, distinct foliation. The Total Core Recovery (TCR) measured on the core samples was between 93 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples of the upper 1 m of the bedrock is highly variable, ranging from 26 percent (in borehole 559-6; attributed to a continuous vertical fracture) to 100 percent, indicating a poor to excellent quality. Below the upper 1 m, the RQD typically measured from about 81 to 100 percent, typically greater than 90 percent, indicating a rock mass of good to excellent quality.

Axial and diametral point load strength tests were performed on samples of the rock core. Diametral point load strength index values are shown on the Record of Drillhole Sheets. Axial point load strength index values ranged from 6.4 MPa to 10.1 MPa and diametral point load strength index values ranged from 1.1 MPa to 6.8 MPa, typically greater than 3.0 MPa indicating a strong to very strong rock mass. A summary of the point load index values on the rock core from the nine (9) investigated locations where coring was performed is shown in the following table. Table 1 following the text of this report presents a detailed list of all point load index testing results performed for this investigation along with the estimated Unconfined Compressive Strength (UCS) value for each test.

Borehole (Drillhole) No.	**Average Axial Point Load Index, I_{s50} (MPa)**	**Average Diametral Point Load Index, I_{s50} (MPa)**
559-2	8.2	4.8
559-4	7.5	4.6
559-6	8.1	5.0
559-7	7.3	5.0
559-9	7.3	4.7
559-11	8.1	5.0
559-12	9.5	5.7
559-14	8.9	5.4
559-16	7.8	5.5

4.2.4 Groundwater Conditions

In general, the samples taken in the overburden boreholes were noted to be dry to moist. Groundwater levels during drilling operations were noted to range from about Elevation 213.3 m at the west abutment to 206.0 m at the east abutment (2.1 m to 3.8 m below existing ground surface). Details of the groundwater conditions and water levels observed in the open boreholes / drillholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.3 CLOSURE

This Foundation Investigation Report was prepared by Mr. Chad Gilfillan and reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder Associates Ltd. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
PROVINCE OF ONTARIO

Chad M. Gilfillan, E.I.T.,
Geotechnical Group

J. Paul Dittrich, Ph.D., P.Eng.,
Associate

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

CMG/JPD/FJH/cmg/sm
n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\1 - highway 559-69 bridge\03-1111-028-1 final rpt 05nov30 hwy69-559 bridge report.doc

PART B

DETAILED
FOUNDATION DESIGN REPORT
HIGHWAY 69 / HIGHWAY 559 UNDERPASS
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559
PARRY SOUND, ONTARIO
G.W.P 335-00-00
MINISTRY OF TRANSPORTATION, ONTARIO

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed Highway 69 / Highway 559 underpass structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes, hand excavations and probe holes advanced during the subsurface investigation.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.1 General

It is understood that the Highway 69 / Highway 559 underpass structure will consist of a two-span, slab-on-girder bridge with 39 m span lengths and abutments located east and west of the proposed Highway 69 northbound and southbound alignments and a central pier located in the median.

Based on the information provided on the General Arrangement (GA) Drawing provided by URS on January 27, 2005, the grade of the proposed Highway 559 bridge deck varies between about Elevation 217.2 m and 216.6 m while the grade of the proposed Highway 69 is at about Elevation 208.5 m, with the centre ditch (in the median area) at about Elevation 207.5 m. The proposed approach embankments will be about 3 m (up to 4.5 m immediately behind the abutment) and about 11 m in height at the west and east sides of the bridge, respectively. The existing ground surface varies from about Elevation 215.5 m to 205.9 m at the borehole locations. Figure 2 shows the approximate bedrock surface elevation contours in plan.

The proposed Highway 69 in this area will be constructed in cut and it is understood that this excavation will be carried out prior to construction of the proposed structure. The recommendations given in the following sections have taken this into account as it pertains to foundation design and construction, excavation, drainage and other considerations.

5.2 Bridge Foundation Options

The native soils at the bridge site consist of topsoil overlying a thin (typically less than 2 m deep) deposit of very loose to very dense silty sand to sandy silt. The thin native overburden soils are underlain by strong to very strong granitic gneiss bedrock. The bedrock surface at the proposed foundations, as established at the borehole locations, ranges from about Elevation 214.5 m to 213.7 m at the west abutment; about Elevation 211.8 m to 211.4 m at the pier; and about Elevation 208.5 m at the east abutment. It should be noted that the average bedrock surface elevation at the west abutment area is approximately 6 m higher than the east abutment area. The granitic gneiss bedrock is suitable for the support of the proposed abutments and pier on shallow foundations.

Due to the shallow nature of the overburden deposits at the site, it is understood that integral abutments are not being considered at this location. For integral abutments, a minimum pile length of about 5 m is generally required for support of the abutments which, at this site, would require significant excavation/trenching into the very strong bedrock and would likely be cost prohibitive. Instead, the following foundation alternatives could be considered:

- Semi-integral abutments supported on shallow spread footings; or
- Perched abutments, founded on spread footings placed on well compacted granular pads within the approach embankment fill.

Recommendations for spread footings (founded on bedrock or perched on granular pads) for the bridge abutments and central pier are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the foundation alternatives is given in Table 2 following the text of this report.

It should be noted that spread footings placed on the thin, very loose to loose granular deposits is not considered as a suitable option due to the required footing elevations (as shown on the GA Drawing) and due to the low bearing value and potential for differential settlement across and between different founding elements.

5.3 Spread Footings

The bridge abutments and central pier may be supported on spread footings placed on properly prepared granitic gneiss bedrock. The details of the bedrock surface elevation as encountered in the boreholes at the different foundation elements is summarized in the following table.

Foundation Element	*Borehole Numbers*	*Depth to Bedrock*	*Bedrock Surface Elevation*
West abutment	559-2 to 559-6	0.0 m to 1.5 m	213.7 m to 214.8 m
Central pier	559-7 to 559-11	1.1 m to 2.2 m	211.4 m to 211.9 m
East abutment	559-12 to 559-16	0.0 m to 0.5 m	208.3 m to 208.9 m

Based on the GA drawing provided by URS, the west abutment and central pier are proposed to be founded at about Elevation 212.5 m and Elevation 206.5 m, respectively. This will require a removal of up to about 2.3 m of bedrock (at the west abutment) and 5.4 m of bedrock (at the pier) (not including the rock excavation required to construct the road bed for the proposed Highway 69 NBL and SBL alignments). It is anticipated that the bedrock at this depth would be of good quality assuming that proper excavation/blasting techniques are utilized for removing the excess rock (as discussed in Section 5.8).

The east abutment is proposed to be founded at about Elevation 209.0 m based on the GA Drawing provided by URS. As noted in the table above, the bedrock surface is variable within the limits of this foundation element and is typically located below the proposed founding elevation. In addition, the upper portion of the bedrock is moderately fractured in a few localized areas (RQD values as low as 43 percent as encountered in borehole 559-14) and it may be necessary to subexcavate loose or fractured rock from some areas of the foundation footprint. As such, the footing founding elevation for the east abutment may require a combination of overburden excavation and either bedrock excavation, mass concrete placement or both. One founding level could be chosen for the entire footing, or a stepped footing could be considered to enable an appropriate balance. For design, the following options for founding levels of the east abutment may be considered:

1. A founding elevation of about 209.0 m may be assumed:

   In this case, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete would be placed to raise the grade to the founding level. A Non-Standard Special Provision (NSSP) should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix B). The benefit of this approach is that excavation into the strong to very strong bedrock is avoided.

2. Alternatively, a founding elevation of about 207.5 m may be assumed:

   In this case, following the removal of the overburden, excavation of the upper portion of the bedrock will be required within the foundation footprint. Based on the borehole results, subexcavation of up to about 1.4 m of bedrock will be required. This depth of excavation is recommended so that the footing is founded below the upper moderately fractured zone that was encountered in a few localized areas. It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of 100 MPa to 250 MPa) and the level of fracturing in the upper portion of the rock is variable. This will make excavation potentially difficult, particularly in areas where only small depths and narrow zones of removal are needed (refer to Section 5.8 for bedrock excavation/blasting recommendations).

3. As a third option, an intermediate founding level to those noted above may be assumed for design. In this case, a combination of bedrock subexcavation and mass concrete placement will be required.

All bedrock excavation within and near the footing areas should be carried out using line drilling and pre-shearing techniques in order to minimize shattering and over-break. Additional recommendations on bedrock excavation are provided in Section 5.8.

The abutment footings will likely be founded on the bedrock above the adjacent Highway 69 road grade. The footings must be maintained an adequate distance away from the edge of the rock cut and the rock face adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained. In this regard, the abutment footing should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical from the toe of the rock cut. If the layout does not allow for this setback zone, provision should be made for vertical rock dowels along the crest of the cut to be installed prior to excavation in order to control and pre-support the rock face.

In all areas where mass concreting is to be employed, it will be necessary to clean, scale and remove any loose debris to ensure a proper bond to the bedrock. In addition, a check on the sliding resistance between the mass concrete and the bedrock should be carried out (in accordance with the recommendations provided in Section 5.3.2).

As an alternative to founding the east abutment footing on bedrock (or mass concrete), consideration could be given to the use of an abutment footing perched within the east approach embankment. This option would require that the spread footing be founded on a well compacted granular fill pad (i.e. not founded on rock fill).

The simplest option for the abutment and pier footings, from a foundation perspective, is spread footings placed on the bedrock surface or on mass concrete placed on the bedrock surface which should minimize bedrock excavation difficulties. If a perched abutment was to be used, a longer bridge span may be required as the footprint of the granular fill may encroach on the proposed rock cut for the NBL alignment. In addition, it may not be desirable to use a combination of different foundation types at the same structure. The cost effectiveness of each of the foundation alternatives should be considered in the design.

It should be noted that rock cuts (for the NBL and SBL) and footing excavations to expose the founding bedrock surfaces may, in some places, extend below suspected 'perched' groundwater levels encountered in some boreholes (generally 2 m to 4 m below the existing ground surface). Groundwater control measures (as discussed in Section 5.7.2) may be locally required to maintain dry and stable excavations especially during periods of high groundwater levels.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the surface of the properly prepared granitic gneiss bedrock may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 10,000 kPa. For footings placed on a mass concrete pad, the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the granitic gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with concrete. A NSSP should be included in the Contract Documents to address the requirements for field inspection (an example is provided in Appendix B). In order to carry out this inspection, the excavation should be dry.

For spread footings placed (or perched) within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS (for 25 mm of settlement) will depend on the thickness of the Granular 'A' pad and the consistency and thickness of any underlying soils; a value of 350 kPa may be assumed for preliminary design. If this "perched" abutment option is adopted for the design of the foundation at the east abutment, these resistances would have to be confirmed once the elevation and location of the abutment footing is known.

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

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the granitic gneiss bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. In the case of mass concrete placed on the bedrock surface, the design must also check the sliding resistance between the base of the mass concrete and the bedrock The coefficient of friction, tan

δ, may be taken as 0.70 between the base of the concrete footings and/or mass concrete and the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling 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 in the rock may be handled in the same way as the dowel embedment 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 minimum embedded length within the bedrock of 1 m, 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, a NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (an example is provided in Appendix B).

If "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared granitic gneiss bedrock at this site, frost susceptibility is not an issue. For "perched" abutments, all footings should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.4 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, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 3501.00 and 3504.00.

- 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 Northeastern Region Directive for backfill to structures adjacent to rock embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3505.00.

- 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 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC).*

- For Case I, 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 Select Subgrade Material (SSM) or rock fill:

	SSM	Rock Fill
Soil unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.35	0.24
At rest, K_o	0.50	0.38

- For Case II, 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	0.31
At rest, K_o	0.43	0.47

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

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

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

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for Parry Sound is 0.05. Based on experience, for the thin overburden soils at this site, a 10 to 20 percent amplification factor of the ground motion could occur (particularly in the area of the east approach), resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

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

- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	**Granular B Type II**
Yielding wall	0.32	0.26	0.30
Non-yielding wall	0.37	0.30	0.34

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

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

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

Where K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 19 kN/m^3 for the native materials

d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.5 Approach Embankment Design

The construction of the Highway 69 / Highway 559 underpass structure will require placement of up to about 3 m of fill (up to 4.5 m immediately behind the abutment) within the limits of the west approach embankment and up to about 11 m of fill within the limits of the east approach embankment.

Based on the investigated locations at this site, the approach embankments will be founded on either bedrock (which is exposed at a number of locations in both the east and west approach footprints) or a thin deposit (typically less than 1 m deep) of very loose to very dense silty sand to sandy silt underlain by bedrock at shallow depth. All topsoil and organic matter should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement.

The results of stability and settlement analysis for the new approach embankments are presented in the following sections. It should be noted that the proposed Highway 559 interchange embankments and associated on- and off- ramp alignments located west and immediately east of the proposed structure have been investigated as swamp crossings and/or high fill embankment areas and are reported under separate cover.

5.5.1 Stability

Analyses were performed on the critical (i.e. highest) sections of the proposed new approach embankments to assess stability and liquefaction potential.

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.20), 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 were 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 used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this sites 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 proposed embankment heights and geometries.

At the west approach area, bedrock is either outcropping or at very shallow depth. The thin overburden soils present near the west abutment area will largely be removed as part of the excavation required to found the west abutment footing at about Elevation 212.5 m. As such, the west approach embankment has been assumed to be founded on bedrock for the purposes of stability analysis.

At the east approach area, the very loose to compact subsoils are composed of cohesionless soils up to about 1 m thick in some areas, underlain by bedrock. For these soils, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

At all areas, the analyses assume that organic soils (encountered at or below the ground surface during field investigation operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this area. In general, the soils within the approach embankment areas were not saturated and groundwater was not observed in the open boreholes in the overburden.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. It is understood that consideration is being given to the use of earth fill or rock fill for the construction of the approach embankments, and as indicated in the table below, both fill types were considered in the analysis. Rock fill is assumed to have side slopes at 1.25H:1V and the earth fill is assumed to have side slopes at 2H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 5.5.1.1.

East and West Approach Embankments

Soil Type	Unit Weight (kN/m3)	Strength Parameters
Rock Fill	19	$c' = 0$ kPa, $\phi' = 38°$
Earth Fill (Sand and Gravel)	21	$c' = 0$ kPa, $\phi' = 35°$
Very loose to compact Silty Sand to Sandy Silt	20	$c' = 0$ kPa, $\phi' = 30°$

The results of the stability analyses for the two embankment fill options are summarized in the following table. At each area, the highest (i.e. most critical) embankment section has been analyzed. The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

Location	*Embankment Height at Critical Section (m)*	*Earth Fill Option*		*Rock Fill Option*	
		Recommended Side Slope Profile	*Minimum Factor of Safety*	*Recommended Side Slope Profile*	*Minimum Factor of Safety*
East Approach	11	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
West Approach	4.5				

The incorporation of a 2 m wide bench (or berm) into the uniform side slope profile is required at certain sections of the proposed fill embankments as per OPSD – 202.010 and MTO Northeastern Region guidelines. The presence of a berm will increase the internal and surficial stability of the embankment and aid in surface water control on the slope. The presence of this berm has been incorporated in the stability analysis, where required. Additional details on the berm requirements are described in the following section.

5.5.1.1 Embankment Fill Types and Berm Requirements

The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability.

5.5.1.1.1 Earth Fill

The main advantage of using earth fill (i.e. sand and gravel) is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

For the earth fill option, the incorporation of a 2 m wide mid-height bench (or berm) into the uniform side slope profile is required wherever the embankment will exceed a height of 8 m.

5.5.1.1.2 Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, rock fill will likely be available from the rock cuts proposed for the underpass, thus providing an advantage in cost. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

For the rock fill option, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile is required wherever the embankment will exceed a height of 6 m such that the uninterrupted rock fill slope never exceeds a height of 6 m (as per MTO Northeastern Region guidelines). We understand that the Northeastern Region requirements for berms have recently changed from 6 m to 10 m height. However, we have been instructed to maintain the original guidelines for this project.

5.5.2 Liquefaction Potential

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.06 g, a factor of safety of less than 1.0 against liquefaction is obtained for magnitude 7.0 earthquake events at localized areas under the east approach embankment toes (i.e. areas of sandy silt, low SPT N-values representative of a very loose state of compaction, and low confining stresses under less than about 1.5 m of embankment fill). Total seismic settlements are calculated to be less than 10 mm based on analysis performed in accordance with Tokimatsu and Seed (1987). Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.15 g results in a factor of safety against side slope instability of

1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur. Since deep-seated global instability is not anticipated under the design earthquake event, localized toe failures would be mainly a maintenance issue. This should be considered in the life-cycle costing when assessing the relative costs of the works. Alternatively, consideration could be given to sub-excavation and removal of these silty subsoils prior to construction of the approach embankments in order to eliminate the potential for seismically induced liquefaction at the embankment toes.

5.5.3 Settlement

Settlement analyses were performed on the critical sections of the proposed approach embankments. For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights, approximately 3 m (locally 4.5 m) and 11 m at the west and east approaches, respectively. The unit weights and slope profiles for the embankment fill described in Section 5.5.1 were employed in the analyses. The analyses performed assume that the organic soils/topsoil have been removed prior to construction.

As noted previously, within the west approach embankment area, bedrock is either outcropping or at very shallow depth and the thin overburden soils present near the west abutment will largely be removed as part of the excavation required for the abutment footing construction. As such, the west approach embankment will be founded primarily on bedrock. At the east approach area, the very loose to compact cohesionless subsoils are up to about 1 m thick in some areas, underlain by bedrock. Surficial deposits of topsoil were encountered at some of the investigated locations.

Provided that the surficial topsoil is removed prior to the new embankment fill placement (as discussed in Section 5.6), settlements of the new approach embankments, due to compression of the thin foundation soils, are expected to be minimal. For embankment fills constructed with rock fill, the majority of the settlement of the approach embankments is expected due to compression of the rock fill itself. Estimated post-construction settlements are summarized in Table 3.

The following sections describe the estimated settlement of the foundation soils and the estimated settlements of the embankment fill due to the loading imposed by the new approach embankments

5.5.3.1 Settlement of Cohesionless Foundation Soils

The immediate compression of the very loose to compact silty sand to sandy silt native subsoils encountered in the boreholes in the area of the east approach were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and correlations proposed by Bowles (1984) and Kulhway and Mayne (1990).

The following table presents the results of the estimated settlements of the foundation soils as a results of the new embankment construction in the area of the approaches.

Location of Embankment	*Approximate Chainage*	*Maximum New Embankment Height* (m)*	*Estimated Settlement of Foundation Soils (mm)*
West Approach	9+940 to 9+960	4.5	---**
East Approach	10+040 to 10+060	11+0.2 = 11.2	50

Notes : *includes additional fill required after removal of maximum depth of organics/topsoil
**no foundation soils in this area after organics/topsoil removed

These settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to the filling based on the estimated relatively high permeability of the native soils as indicated by the results of the grain size distributions.

5.5.3.2 Settlement of Rock Fill

If rock fill is used for the construction of the embankments, in addition to the settlement due to compression of the foundation soils described above, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision, SP 206S03 dated January 2004, the settlement of the newly placed rock fill is expected to be minor. In general, it is estimated that for the granitic gneiss rock fill likely to be used at this site, for the up to 11 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill.

Location of Embankment	*Approximate Chainage*	*Maximum New Embankment Height* (m)*	*Estimated Settlement of Embankment Soils (mm)*
West Approach	9+940 to 9+960	4.5	45
East Approach	10+040 to 10+060	11+0.2 = 11.2	110

Note : *includes additional fill required after removal of maximum depth of organics/topsoil

It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction. If rock fill is used, consideration should be given to delaying the final paving for about 1 year to allow the majority of the settlement to take place.

5.6 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be appropriate subbase for the proposed approach embankments; however, prior to the placement of any fill, all surface and near surface layers of topsoil/organic deposits and any softened or loosened soils should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled.

Table 3 summarizes the recommended fill type to be placed for the widenings, the location and depth of organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements, platform widenings (in accordance with NRE 98-200) and the recommended method of removal of organics. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

5.6.1 Removal of Organics

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil and leaf litter) of up to about 0.2 m deep can be expected in some areas of the new approach embankments. These organic layers should be stripped from the plan limits of the approach areas prior to fill placement.

5.6.2 Embankment Fill Placement

If earth fill (granular) is to be used for construction of the new embankments, placement of all granular fill material should be carried out in accordance with OPSS 206.07.07, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for earth fill embankments should be no steeper than 2H:1V.

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision SP 206S03 dated January 2004. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion.

5.7 Design and Construction Considerations

5.7.1 Excavation

As noted in Section 5.3, excavations for the construction of spread footings for the bridge abutments and pier and/or as part of the cutting required to establish the grade for the NBL and SBL of the new Highway 69 will extend up to 2 m through the very loose to very dense silty sand to sandy silt native soils and up to about 7 m into the underlying bedrock. In addition, as noted in Section 5.6, excavation within the plan limits of the approach embankments will be required in order to remove topsoil / organic deposits up to about 0.2 m deep prior to fill placement.

If space permits, temporary excavations (i.e. those that are open only for a relatively short period) through the native soils above the groundwater table may be made with side slopes no steeper than about 1.5H:1V. Below the groundwater table, shallower side slopes (no steeper than about 2H:1V) will be required unless prior dewatering is carried out.

If space and/or staging restrict the use of open cuts, a temporary support system could be constructed to support the excavations in the area of the bridge structure foundations. The temporary excavation support system should be in accordance with MTO Special Provision 539S01. The temporary support system should be designed to Performance Level 3 as defined in SP 539S01. Roadway protection should be as per current MTO Special Provision 539S01.

It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of 100 MPa to 250 MPa). This will make excavation potentially difficult particularly in areas where only small depths and narrow zones of removal are needed. Bedrock excavation in the vicinity of the proposed structure foundations should be carried out using line drilling and pre-shearing techniques (as discussed in Section 5.8). This method would provide better control over the configuration of the founding surface, and this procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

5.7.2 Groundwater and Surface Water Control

The groundwater level at the site is generally at about 2 m to 4 m below the existing ground surface and may be locally 'perched' in the west abutment and central pier areas. At the east abutment location, excavations to expose the founding bedrock surface for spread footings may require groundwater control. At the west abutment and central pier locations, the bedrock surface is at a higher elevation comparatively and, depending on the groundwater level at the time of construction, groundwater control may not be required. However, in all cases a dry and stable excavation will be required to permit placement of mass concrete and construction of footings in the dry.

It is likely that open-cut excavations with sufficient pumping and/or controlled drainage from the topographically high area will adequately manage the groundwater. Surface water should be directed away from the excavations at all times.

It should be noted that the base of the excavation within the bedrock for the NBL and SBL of the new Highway 69 is below the water level in the ponds and swamps located to the west of the existing Highway 69 in this area of the site. Although the potential exists for seepage to migrate along fractures in the bedrock and into the rock cuts for the new Highway 69, it is anticipated that the seepage volumes will be low.

5.8 Blasting Recommendations for Rock Excavations

5.8.1 Excavation Considerations

For excavations into the bedrock, the overall slope to the cut face may be formed vertical or at a steep slope (i.e. 0.25H:1V). The use of controlled blasting techniques (such as pre-shearing or cushion blasting) are recommended, particularly along footing areas, in order to provide a neat excavation line and minimize face instabilities resulting from blast damage to the rock mass.

5.8.2 Special Provisions

5.8.2.1 Blasting

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. It is recommended that the Contractor retain a blast engineer and submit proposed blast plans for review at least 3 weeks in advance of rock excavation.

The use of explosives shall follow the general specifications outlined in OPSS 120 and the *Guidelines for Safe Blasting in Ontario Highway Construction Operations, ORBA October 2001* should be followed. It is recommended that a separate NSSP for the control of all blasting operations be prepared (refer to SP 299F06). The NSSP should include, but not be limited to, the following:

- Outlining the requirements, procedure and extent of a pre-blast survey. This would include all structures within a radius of about 100 m of the blasting operations, as well as notification to all individuals working or living within 500 m.
- Submission of a blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.

We recommend limiting ground vibration levels to 50 mm/s for adjacent services and buildings. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the adjacent structures.

It is recommended that the specification for the blasting require a minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling. It is also recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a Quality Verification Engineer to assess if the blasting operations have effected the integrity of the rock mass that will ultimately be supporting the new footings. A NSSP for rock bolting, if necessary, should be included in the Contract Specifications in the event that additional support is required in these areas.

5.9 CLOSURE

This Foundation Design Report was prepared by Mr. Chad Gilfillan, E.I.T. and technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder Associates Ltd. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted an independent audit review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
PROVINCE OF ONTARIO

Chad M. Gilfillan, E.I.T.,
Geotechnical Group

J. Paul Dittrich, Ph.D., P.Eng.,
Associate

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

CMG/JPD/FJH/jpd/cmg/sm
n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\1 - highway 559-69 bridge\03-1111-028-1 final rpt 05nov30 hwy69-559 bridge report.doc

REFERENCES

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, 2nd Edition. McGraw-Hill Book Company, New York.

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-00. 2001. CSA Special Publication, S6.1-00. Canadian Standards Association.

Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, 3rd Edition (Ontario Geological Survey, Special Volume 2). Ontario Ministry of Natural Resources.

Guidelines for Safe Blasting in Ontario Highway Construction Operations, Ontario Road Builder's Association (ORBA), October 2001

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.

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Makdisis, F.I., and Seed, H.B. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. ASCE Journal of the Geotechnical Engineering Division, V. 104, GT7, pp 849-867.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Schmertmann, J.H. 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

Tokimatsu, K., and Seed, H. 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking. ASCE Journal of Geotechnical Engineering, V.113, N.8.

U.S. Navy. 1971. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

RECORD OF BOREHOLE LOGS

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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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 2m
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

Term	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

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 separations) 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 separations such as fractures, bedding planes and foliation planes or mechanically induced features 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	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
‖ - Parallel To	
⊥ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No 559-1

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033556.7 ;E 255317.4	ORIGINATED BY EHS	
DIST 52 HWY 69	BOREHOLE TYPE Hand excavated	COMPILED BY KG	
DATUM Geodetic	DATE September 23, 2004	CHECKED BY CG	

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.8	GROUND SURFACE										
0.0	Topsoil, moss										
0.2	Silty Sand, trace gravel, trace rootlets Light brown										
	End of Borehole Hand excavated to bedrock						214				

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

Golder Associates

RECORD OF BOREHOLE No 559-2

METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5033572.8 ;E 255320.0
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Hand excavated
COMPILED BY KG
DATUM Geodetic
DATE September 24, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.7	GROUND SURFACE										
0.0	Topsoil										
0.2	Bedrock Refer to Record of Drillhole 559-2 for details						214				
							213				
						∇	212				
211.3 3.4	End of Borehole										

Notes:

1. Hand excavated to bedrock.
2. Water level in open borehole at 2.4 m depth upon completion of drilling.

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

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

RECORD OF DRILLHOLE: 559-2

PROJECT: 03-1111-028

LOCATION: N 5033572.8 ;E 255320.0

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Sept. 24, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

SHEET 1 OF 1

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	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	214.50			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	0.20	1	MB,, JN,IR, JN,IR,	
1				JN,IR, JN,IR,	
2			2	JN,IR, JN,IR,	∇
3			3	JN,IR, MB,, MB,,	
	END OF DRILLHOLE Note: 1. Water level in open borehole at 2.4 m depth upon completion of drilling	211.35 3.35		MB,,	
4					
5					
6					
7					
8					
9					
10					

MISS-ROCK-2 031111028AARCK.GPJ GAL-CANADA.GDT 27/1/05 JFC

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-3

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033575.3 ;E 255321.6		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Probe hole	COMPILED BY KG
DATUM Geodetic	DATE September 24, 2004		CHECKED BY CG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
214.7	GROUND SURFACE										
0.0	Bedrock outcrop										
							214				

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

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

Golder Associates

RECORD OF BOREHOLE No 559-4

 METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033570.5 ;E 255326.4		ORIGINATED BY EHS
DIST 52 HWY 69	BOREHOLE TYPE Hand excavated		COMPILED BY KG
DATUM Geodetic	DATE September 23, 2004		CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.1	GROUND SURFACE										
0.0	Topsoil						215				
	Silty Sand, trace organics, trace gravel										
0.4	Weathered Bedrock										
	Bedrock										
	Refer to Record of Drillhole 559-4 for details						214				
						▽	213				
							212				
211.7 / 3.5	End of Borehole										

Notes:

1. Hand excavated to bedrock.

2. Water level in open borehole at 2.2 m depth upon completion of drilling.

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

MISS_MTO_03111028AAMTO.GPJ ON_MOT.GDT 8/11/05

RECORD OF DRILLHOLE: 559-4

PROJECT: 03-1111-028

SHEET 1 OF 1

LOCATION: N 5033570.5 ;E 255326.4

DRILLING DATE: Sept. 23, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	214.74			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	0.36			
				MB,,	
1			1		
				JN,IR, JN,IR, MB,,	
2				JN,IR, JN,IR,	∇
			2	MB,,	
3				MB,, JN,IR, MB,, MB,, MB,,	
		211.65	3		
	END OF DRILLHOLE	3.45		MB,,	
4	Note: 1. Water level in open borehole at 2.2 m upon completion of drilling				
5					
6					
7					
8					
9					
10					

Other column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER .3m; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-5

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033565.7 ;E 255331.0		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic	DATE September 24, 2004		CHECKED BY CG

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.5	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace gravel Loose Light brown Moist		1	SS	6		215		o		
214.8 / 0.7	End of Borehole Auger refusal										

Note:

1. Open borehole dry upon completion of drilling.

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 26/1/05

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

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

Golder Associates

RECORD OF BOREHOLE No 559-6

METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5033568.2 ;E 255332.6
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE NW Casing
COMPILED BY KG
DATUM Geodetic
DATE September 23, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
215.2	GROUND SURFACE											
0.0	Topsoil					215						
0.2	Silty Sand, some gravel, trace organics Very loose to very dense Light brown and oxidized Dry	1	SS	2								
		2	SS	38			o		24	53	(23)	
213.8		3	SS	76/.22		214						
1.5	Bedrock Refer to Record of Drillhole 559-6 for details				∇	213						
						212						
						211						
209.9						210						
5.3	End of Borehole											

Notes:

1. Spoon refusal at 1.5 m depth.

2. Water level in open borehole at 2.1 m depth upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: 559-6

SHEET 1 OF 1

LOCATION: N 5033568.2 ;E 255332.6

DRILLING DATE: Sept. 23, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	213.70			
	Weathered Bedrock	1.50			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	1.63			
2			1	JN,Ro, JN,IR, JN,Ro, JN,Ro, JN,Ro,	∇
3			2	MB,, MB,, JN,IR,	
4				MB,, JN,IR, MB,,	
5			3	JN,Ro, JN,Ro, MB,,	
	END OF DRILLHOLE	209.93 5.27			
6					
7					
8					
9					
10					
11					

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 28/1/05 JFC

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-7

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033606.2 ;E 255341.8		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic	DATE September 21, 2004		CHECKED BY CG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
212.9	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace gravel Loose to dense Light brown and oxidized Moist		1	SS	5						
			2	SS	50		212		o		
211.8 / 1.1	Bedrock Refer to Record of Drillhole 559-7 for details						211				
						∇	210				
							209				
208.4 / 4.5	End of Borehole Notes: 1. Auger refusal at 1.1 m depth. 2. Water level in open borehole at 2.7 m depth upon completion of drilling.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: 559-7

SHEET 1 OF 1

LOCATION: N 5033606.2 ;E 255341.8

DRILLING DATE: Sept. 24 and 25, 2004

DATUM: Geodetic

DRILL RIG: TRACK CME 55

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	211.80			
	Weathered Bedrock	1.10			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	1.22	1	MB,,	
2			2	JN,ST,Ro	
				MB,,	
				JN,IR,	∇
3				MB,, JN,,	
				JN,IR,	
			3	MB,,	
4				JN,IR,	
		208.40 4.50		MB,,	
	END OF DRILLHOLE				
5					
6					
7					
8					
9					
10					
11					

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 26/1/05 JFC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

PROJECT 03-1111-028	RECORD OF BOREHOLE No 559-8	1 OF 1	METRIC
W.P. 335-00-00	LOCATION N 5033608.8 ;E 255343.5		ORIGINATED BY EHS
DIST 52 HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY KG
DATUM Geodetic	DATE September 26, 2004		CHECKED BY CG

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: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.0	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace gravel, trace organics Loose to very dense Light brown with oxidized layers Moist		1	SS	5						
211.9			2	SS	114		212				
1.1	End of Borehole Auger refusal Note: 1. Open borehole dry upon completion of drilling.										

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 26/1/05

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

RECORD OF BOREHOLE No 559-9

METRIC

PROJECT	03-1111-028				
W.P.	335-00-00	LOCATION	N 5033604.0 ;E 255348.2	ORIGINATED BY	EHS
DIST 52	HWY 69	BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY	KG
DATUM	Geodetic	DATE	September 25, 2004	CHECKED BY	CG

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLE 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
213.7	GROUND SURFACE										
0.0	Topsoil										
0.2	Sandy Silt, trace gravel, trace organics Very loose Layered light and dark brown Moist		1	SS	1		213				
			2	SS	2				○		
212.3 1.4	Silty Sand, trace to some gravel, trace organics Very dense Layered oxidized and light brown Moist		3	SS	72		212		○		
211.8 1.9	Bedrock Refer to Record of Drillhole 559-9 for details						211				
							210				
						∇	209				
208.3 5.4	End of Borehole										

Notes:

1. Auger refusal at 1.9 m depth.

2. Water level in open borehole at 4.8 m depth upon completion of drilling.

Legend: SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: 559-9

LOCATION: N 5033604.0 ;E 255348.2

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Sept. 25 and 26, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH / COLOUR % RETURN	RECOVERY: TOTAL CORE % (20 40 60 80) / SOLID CORE % (20 40 60 80)	R.Q.D. % (20 40 60 80)	FRACT. INDEX PER .3m (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	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		211.80												
2		Weathered Bedrock		1.90												
		Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink		211.51 2.19	1							JN,IR,				
3					2							MB,, MB,, JN,IR, JN,IR, JN,IR, JN,IR, MB,,				∇
4												JN,IR, JN,IR, MB,,				
					3							FO,IR, MB,, JN,IR, JN,IR, JN,IR,				
5												FO,IR, MB,,				
		END OF DRILLHOLE		208.26 5.44												
6																
7																
8																
9																
10																
11																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-10 — 1 OF 1 — METRIC

PROJECT	03-1111-028				
W.P.	335-00-00	LOCATION	N 5033599.2 ;E 255352.9	ORIGINATED BY	EHS
DIST 52	HWY 69	BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY	KG
DATUM	Geodetic	DATE	September 25, 2004	CHECKED BY	CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.6	GROUND SURFACE										
0.0	Topsoil										
0.2	Sandy Silt, trace gravel, trace organics Very loose to loose Light brown and oxidized Moist		1	SS	1						
							213				
			2	SS	3				o		
211.9			3	SS	3/.15		212				
1.7	End of Borehole Auger refusal Spoon refusal Note: 1. Open borehole dry upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No 559-11

METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5033601.7 ;E 255354.5
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY KG
DATUM Geodetic
DATE September 25, 2004
CHECKED BY CG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.6	GROUND SURFACE							
0.0	Topsoil							
0.2	Silty Sand to Sandy Silt, trace gravel, trace clay; Very loose to very dense; Light brown and oxidized; Moist	1	SS	2		213		
		2	SS	3			o	
		3	SS	15		212		12 31 49 8
211.4		4	SS	100			o	
2.2	Bedrock; Refer to Record of Drillhole 559-11 for details				∇	211, 210, 209		
208.2 / 5.4	End of Borehole							

Notes:

1. Auger refusal at 2.2 m depth.
2. Water level in open borehole at 3.1 m depth upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
UNIT WEIGHT γ kN/m³

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

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 26/1/05

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: 559-11

SHEET 1 OF 1

LOCATION: N 5033601.7 ;E 255354.5

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Sept. 25, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	211.40			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	2.20	1	MB,, JN,Ro, JN,Ro, MB,, JN,Ro, MB,,	
3			2	MB,, MB,, JN,Ro, MB,, JN,Ro,	∇
4			3	MB,, MB,, MB,, JN,Ro, MB,,	
5		208.21			
	END OF DRILLHOLE	5.39			
6					
7					
8					
9					
10					
11					
12					

Column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER .3m; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 26/1/05 JFC

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-12

METRIC

PROJECT 03-1111-028

W.P. 335-00-00 LOCATION N 5033639.7 ;E 255363.7 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Advanced by split-spoon sampler COMPILED BY KG

DATUM Geodetic DATE September 26, 2004 CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	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
209.0	GROUND SURFACE										
0.0	Topsoil										
208.5 / 0.5	Silty Sand, trace gravel, trace organics Very loose Brown Moist		1	SS	3						
	Bedrock Refer to Record of Drillhole 559-12 for details						208				
						∇	207				
							206				
205.3 / 3.7	End of Borehole										

Notes:

1. Spoon refusal at 0.5 m depth.

2. Water level in open borehole at 2.1 m depth upon completion of drilling.

Legend: ○ UNCONFINED; ● QUICK TRIAXIAL; + FIELD VANE; × REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

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

RECORD OF DRILLHOLE: 559-12

PROJECT: 03-1111-028
LOCATION: N 5033639.7 ;E 255363.7
INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Sept. 26, 2004
DRILL RIG: TRACK CME 55
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		206.50			
		Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink		0.50		MB,,	
1					1	MB,, FO,UN,Ro FO,UN,Ro MB,,	
2					2	SHR,IR, MB,, MB,,	▽
3					3	MB,, MB,, MB,, MB,,	
		END OF DRILLHOLE		205.29 3.71			
4							
5							
6							
7							
8							
9							
10							

Other columns (not filled with values): PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3m; B Angle; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec; Diametral Point Load Index (MPa); RMC -Q' AVG.

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 26/1/05 JFC

Golder Associates

RECORD OF BOREHOLE No 559-13

METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5033642.2 ;E 255365.3
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Probe hole
COMPILED BY KG
DATUM Geodetic
DATE September 26, 2004
CHECKED BY CG

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_p, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.9	GROUND SURFACE										
0.0	Bedrock outcrop										
							208				

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

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

Golder Associates

RECORD OF BOREHOLE No 559-14

METRIC

PROJECT 03-1111-028

W.P. 335-00-00 LOCATION N 5033637.5 ;E 255370.1 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Hand excavated COMPILED BY KG

DATUM Geodetic DATE September 26, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.7	GROUND SURFACE										
0.0	Topsoil										
	Silty sand, some organics										
0.4	Very loose Reddish brown Moist						208				
	Bedrock Refer to Record of Drillhole 559-14 for details						207				
						∇	206				
205.1 3.6	End of Borehole Notes: 1. Hand excavated to bedrock. 2. Water level in open borehole at 2.4 m depth upon completion of drilling.										

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 27/1/05

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

RECORD OF DRILLHOLE: 559-14

PROJECT: 03-1111-028

SHEET 1 OF 1

LOCATION: N 5033637.5 ;E 255370.1

DRILLING DATE: Sept. 26, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	208.34			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	0.36		JN,ST,Ro	
				JN,ST,Ro	
1			1	JN,UN,Ro	
				MB,, MB,,	
				JN,UN,Ro	
2			2		
				JN,ST,Ro	∇
				MB,,	
3			3	FO,UN,Ro	
				FO,UN,Ro	
		205.07			
	END OF DRILLHOLE	3.63		MB,,	
	Note: 1. Water level in open borehole at 2.4 m depth upon completion of drilling.				
4					
5					
6					
7					
8					
9					
10					

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

RECORD OF BOREHOLE No 559-15

METRIC

PROJECT	03-1111-028			ORIGINATED BY	EHS
W.P.	335-00-00	LOCATION	N 5033632.7 ;E 255374.7	COMPILED BY	KG
DIST	52 HWY 69	BOREHOLE TYPE	Advanced by split-spoon sampler	CHECKED BY	CG
DATUM	Geodetic	DATE	September 27, 2004		

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES 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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.7	GROUND SURFACE										
208.4 0.0	Topsoil		1	SS	2						
0.3	Sandy Silt, trace gravel, trace organics Loose Reddish brown Moist						208				
	End of Borehole Spoon refusal										

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

Golder Associates

RECORD OF BOREHOLE No 559-16

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5033635.2 ;E 255376.4	ORIGINATED BY EHS	
DIST 52	HWY 69	BOREHOLE TYPE Hand excavated	COMPILED BY KG
DATUM Geodetic	DATE September 27, 2004	CHECKED BY CG	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.6	GROUND SURFACE										
0.0	Topsoil										
0.1	Bedrock Refer to Record of Drillhole 559-16 for details						208				
							207				
						∇	206				
205.4 3.2	End of Borehole Notes: 1. Hand excavated to bedrock. 2. Water level in open borehole at 2.7 m depth upon completion of drilling.										

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

RECORD OF DRILLHOLE: 559-16

PROJECT: 03-1111-028

LOCATION: N 5033635.2 ;E 255376.4

INCLINATION: -90°   AZIMUTH: ---

DRILLING DATE: Sept. 27, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	208.52			
	Granitic Gneiss Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink	0.08	1	MB,,	
1				JN,IR, JN,IR, JN,IR, JN,IR, MB,,	
			2	MB,, JN,UN,Ro MB,,	
2			3	MB,, JN,,Ro	∇
3		205.39			
	END OF DRILLHOLE Note: 1. Water level in open borehole at 2.7 m depth upon completion of drilling.	3.21			
4					
5					
6					
7					
8					
9					
10					

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 26/1/05 JFC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No 559-17

METRIC

PROJECT 03-1111-028

W.P. 335-00-00

LOCATION N 5033651.3 ;E 255379.1

ORIGINATED BY EHS

DIST 52 HWY 69

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY KG

DATUM Geodetic

DATE September 27, 2004

CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.9	GROUND SURFACE										
0.0	Topsoil										
0.2	Sandy Silt, trace gravel, trace organics Very loose Oxidized		1	SS	1						
205.3											
205.0 0.9	Sand, trace silt, trace gravel Compact Light brown and oxidized		2	SS	5/.08		205				
	End of Borehole Auger refusal Spoon refusal										

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

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

TABLES

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.: 03-1111-028

LOCATION: Proposed Highway 69 / Highway 559 Underpass

DATE: December 24, 2004

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type (D=Diametral, A=Axial)	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
559-2	1	Granitic Gneiss	1.2	0.4	D	5.177	119
559-2	2	Granitic Gneiss	1.9	0.6	A	7.825	180
559-2	3	Granitic Gneiss	3.6	1.1	D	4.905	113
559-2	4	Granitic Gneiss	4.8	1.5	A	8.512	196
559-2	5	Granitic Gneiss	6.1	1.9	D	5.263	121
559-2	6	Granitic Gneiss	8.1	2.5	D	3.985	92
559-2	7	Granitic Gneiss	10.4	3.2	D	4.769	110
559-4	1	Granitic Gneiss	2.2	0.7	D	5.476	126
559-4	2	Granitic Gneiss	1.9	0.6	A	6.399	147
559-4	3	Granitic Gneiss	5.2	1.6	D	4.854	112
559-4	4	Granitic Gneiss	7.5	2.3	D	4.820	111
559-4	5	Granitic Gneiss	7.2	2.2	A	8.645	199
559-4	6	Granitic Gneiss	9.5	2.9	D	3.125	72
559-6	1	Granitic Gneiss	6.4	2.0	D	3.253	75
559-6	2	Granitic Gneiss	6.8	2.1	A	8.739	201
559-6	3	Granitic Gneiss	10.9	3.3	D	5.637	130
559-6	4	Granitic Gneiss	11.9	3.6	A	7.401	170
559-6	5	Granitic Gneiss	12.6	3.8	D	4.198	97
559-6	6	Granitic Gneiss	15.4	4.7	D	6.829	157
559-7	1	Granitic Gneiss	4.2	1.3	D	5.544	128
559-7	2	Granitic Gneiss	4.3	1.3	A	7.607	175
559-7	3	Granitic Gneiss	7.3	2.2	D	5.544	128
559-7	4	Granitic Gneiss	8.9	2.7	A	7.062	162
559-7	5	Granitic Gneiss	12.2	3.7	D	4.564	105
559-7	6	Granitic Gneiss	14.1	4.3	D	4.207	97
559-9	1	Granitic Gneiss	7.6	2.3	D	5.731	132
559-9	2	Granitic Gneiss	8.2	2.5	A	7.328	169
559-9	3	Granitic Gneiss	9.0	2.8	D	6.250	144
559-9	4	Granitic Gneiss	12.6	3.8	D	1.150	26
559-9	5	Granitic Gneiss	12.8	3.9	D	5.288	122
559-9	6	Granitic Gneiss	11.8	3.6	A	7.253	167
559-9	7	Granitic Gneiss	16.3	5.0	D	4.939	114
559-11	1	Granitic Gneiss	7.7	2.3	D	5.160	119
559-11	2	Granitic Gneiss	8.3	2.5	A	7.544	174
559-11	3	Granitic Gneiss	10.3	3.1	D	5.467	126
559-11	4	Granitic Gneiss	12.6	3.8	A	8.736	201
559-11	5	Granitic Gneiss	14.6	4.5	D	3.611	83
559-11	6	Granitic Gneiss	16.8	5.1	D	5.961	137

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.: 03-1111-028
LOCATION: Proposed Highway 69 / Highway 559 Underpass
DATE: December 24, 2004

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type (D=Diametral, A=Axial)	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
559-12	1	Granitic Gneiss	2.0	0.6	D	5.714	131
559-12	2	Granitic Gneiss	3.3	1.0	A	10.050	231
559-12	3	Granitic Gneiss	4.5	1.4	D	5.424	125
559-12	4	Granitic Gneiss	6.8	2.1	A	8.912	205
559-12	5	Granitic Gneiss	8.6	2.6	D	5.671	130
559-12	6	Granitic Gneiss	10.3	3.1	D	5.995	138
559-14	1	Granitic Gneiss	3.5	1.1	D	5.033	116
559-14	2	Granitic Gneiss	3.9	1.2	A	8.354	192
559-14	3	Granitic Gneiss	4.9	1.5	D	4.598	106
559-14	4	Granitic Gneiss	6.2	1.9	A	9.518	219
559-14	5	Granitic Gneiss	8.3	2.5	D	6.216	143
559-14	6	Granitic Gneiss	10.6	3.2	D	5.561	128
559-16	1	Granitic Gneiss	1.3	0.4	A	7.188	165
559-16	2	Granitic Gneiss	1.6	0.5	D	6.157	142
559-16	3	Granitic Gneiss	2.7	0.8	D	5.169	119
559-16	4	Granitic Gneiss	4.3	1.3	A	8.370	193
559-16	5	Granitic Gneiss	7.3	2.2	D	4.828	111
559-16	6	Granitic Gneiss	9.6	2.9	D	5.671	130
SUMMARY[2]					**Average Axial**	**8.030**	**185**
					Average Diametral	**5.129**	**118**
					St. Dev. Axial	**0.637**	**15**
					St. Dev. Diametral	**0.698**	**16**
					Number of Axial Tests	**18**	
					Number of DiametralTests	**38**	

[1] UCS = Is x 23 is based on previous experience and would require UCS testing to further validate this relationship.
[2]Statistical summary based on the removal of the 2 highest and 2 lowest values.
Note: Specimens tend to be anisotropic in nature (ie. stronger axial than diametral).

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES
Highway 69 / Highway 559 Underpass
G.W.P. 335-00-00

Footing Option	*NF*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread Footings on bedrock or mass concrete pad		Can minimize bedrock excavation depending on design footing level.	Variable bedrock surface will require bedrock and soil excavation with mass concrete placement to achieve level footing. Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break.	Much lower relative costs than piled foundations since less bedrock excavation required.	If bedrock is higher than anticipated, bedrock removal is required. Variability in bedrock surface will impact mass concrete quantities and excavation depths.
Spread Footings perched within embankment fill		Can eliminate bedrock removal and/or mass concrete placement at east abutment.	Not practical at west abutment and central pier where bedrock excavation required to accommodate geometry of bridge. Potential for differential settlement between east abutment (due to compression of embankment fill) and central pier/west abutment (founded on unyielding bedrock).	Lower relative costs than piled foundations since less bedrock excavation required. Possible higher relative costs then spread footing on bedrock since lower allowable bearing capacity will require larger footing size.	Different footing design required at east abutment and central pier/west abutment.
Steel H Piles			Due to shallow depth of bedrock, bedrock excavation to form trench will be required to achieve minimum required piles lengths.	Significant bedrock trench for H-piles will increase costs for blasting and backfilling as compared to costs for bedrock excavation for spread footing alternative.	Not recommended due to significant depth of excavation required in strong bedrock.

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\draft\highway 559-69 bridge\tables\table2_evaluation foundation alternatives.doc

NF: Indicates that the founding option is considered not feasible.

TABLE 3
Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Highway 69 / Highway 559 Underpass
G.W.P. 335-00-00

Highway	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered along alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction Settlement (δ)* and Platform Widening (w)** (mm)	Swamp Excavation / Organic Removal OPSD
Highway 69 / Highway 559 Structure	9+940 to 9+960	West Approach (fill typically up to 3 m high; up to 4.5 m high immediately behind abutment)	Bedrock at or near ground surface	Rock fill	Yes. Up to 0.2 m below ground surface.	1.25H : 1V	No.	$\delta = 45$ $w = 1000$	Remove all organics within footprint of embankment.
	10+040 to 10+060	East Approach High Fill (fill up to 11 m high)	Shallow silty sand overburden	Rock fill	Yes. Up to 0.3 m below ground surface.	1.25H: 1V	Yes. 6m high x 2 m wide (where embankment height exceeds 6 m).	$\delta = 110$ $w = 1000$	Remove all organics within footprint of embankment.

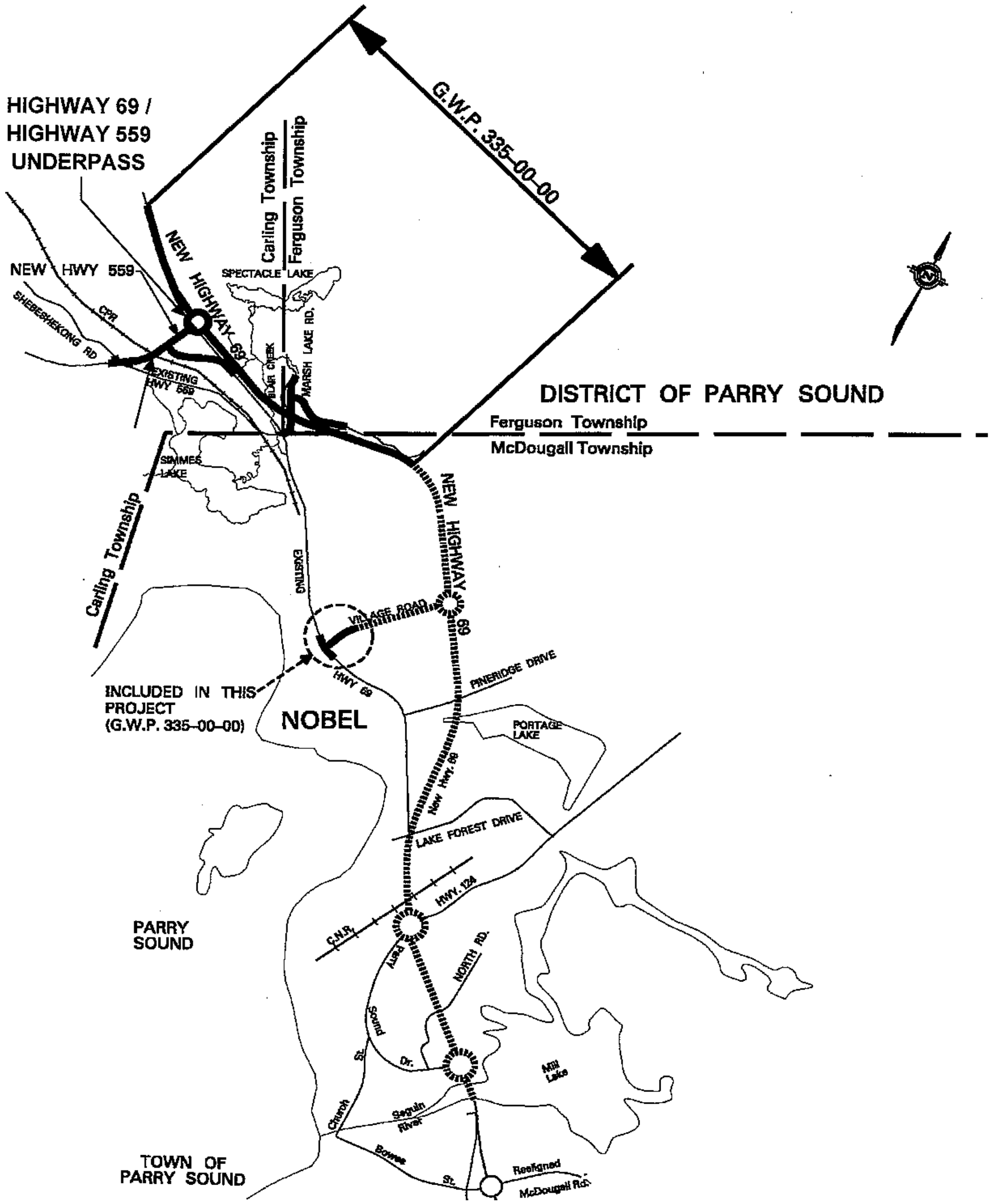
n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\draft\highway 559-69 bridge\tables\table3_summaryapproachembankmentrecommendations (incl nre 98-200).doc

Note : * **Settlements include compression of rockfill plus compression of cohesive layers below embankment (where encountered).**
** **Recommended embankment platform widening (per embankment side) based on guidelines in NRE 98-200.**

FIGURES

SITE LOCATION MAP
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559

FIGURE 1



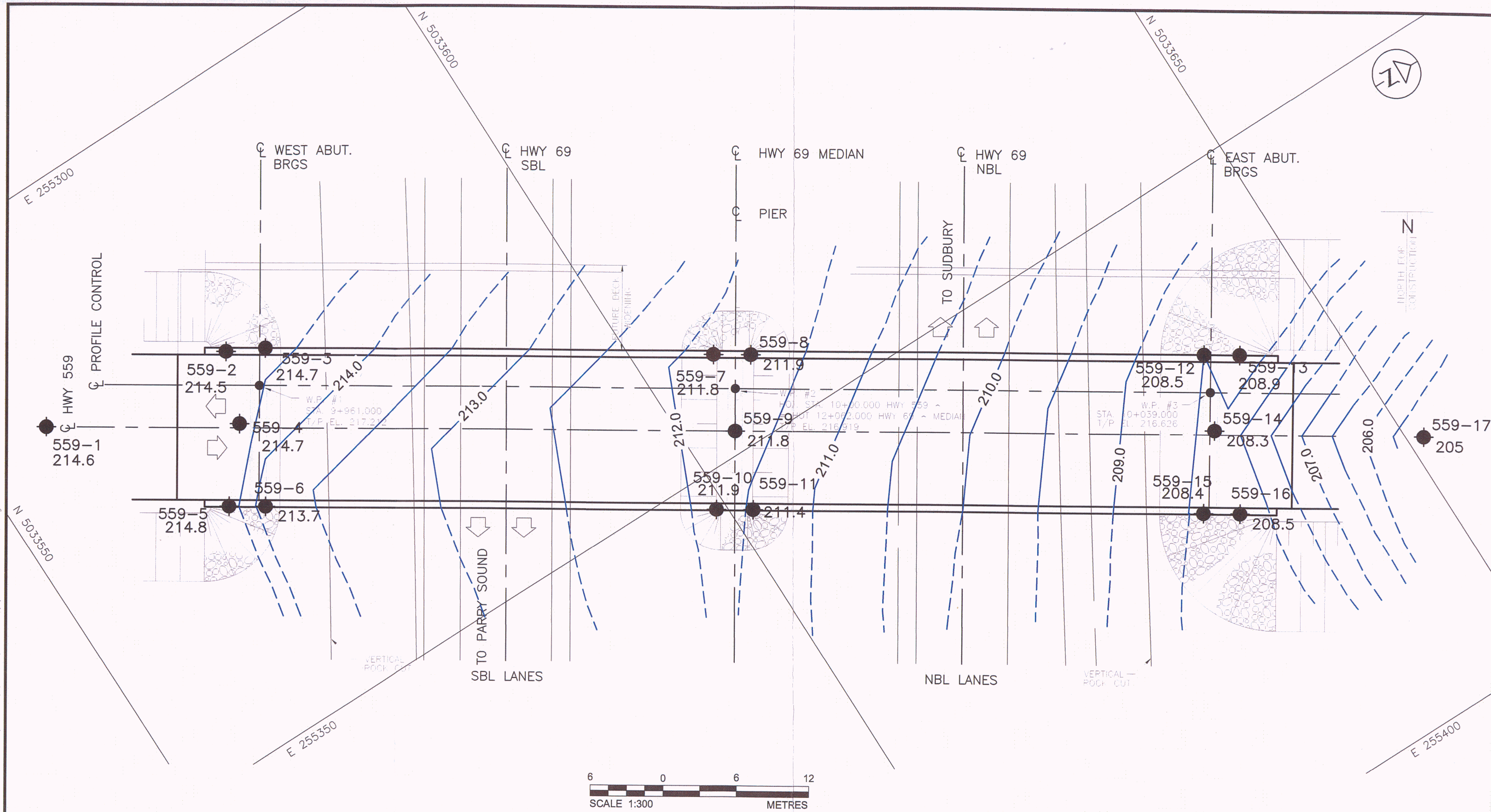
N:\Active\2003\1111\03-1111-028 URS Hwy 69 Parry Sound\Reporting\Final\1 - Highway 559-69 Bridge\Figures\Figure 1.xls

G.W.P. No. 335-00-00
Date: November 2005
Project: 03-1111-028-1

Golder Associates

Drawn by: CMG
Checked by: JPD

Provided in digital format by URS on January 7, 2005



LEGEND:

- APPROXIMATE BOREHOLE LOCATION
- 210.0 BEDROCK SURFACE ELEVATION (m)
- —210.0— APPROXIMATE BEDROCK CONTOUR ELEVATION (m) CONTOUR INTERVAL IS 0.5 METRES
- — — — ESTIMATED EXTENDED BEDROCK CONTOURS (m)

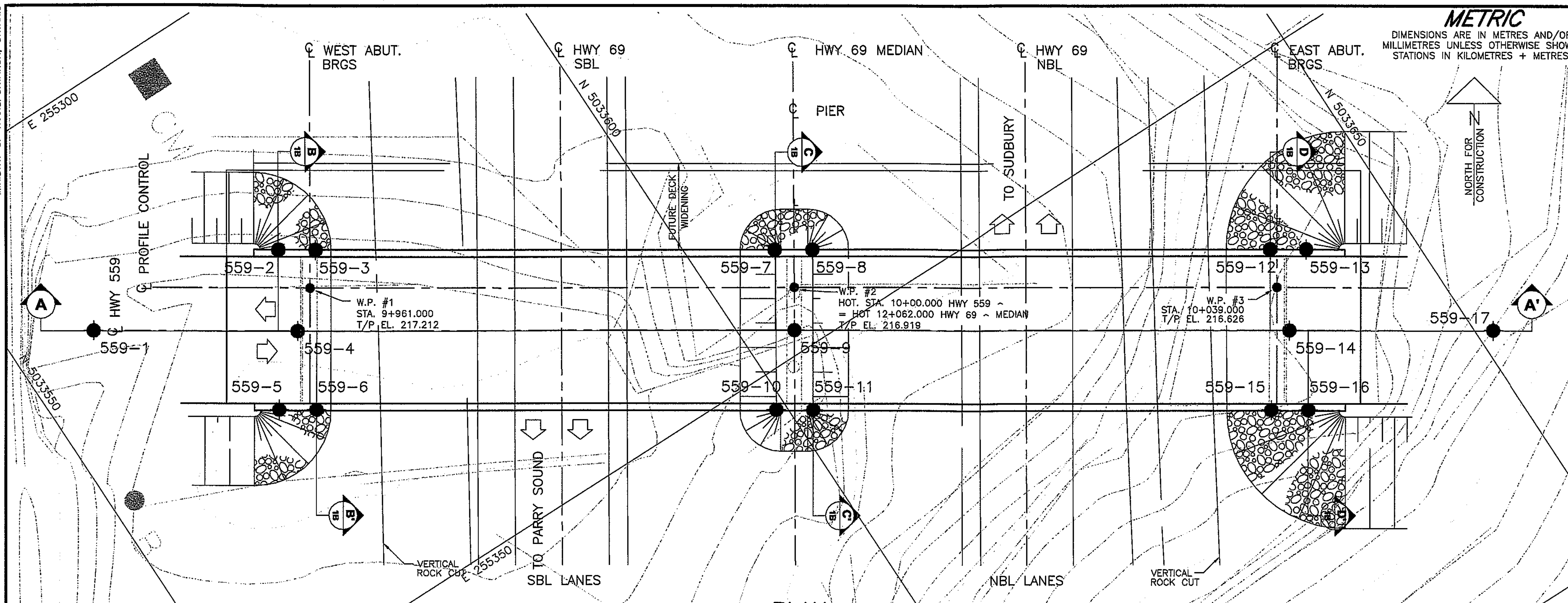
Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	TITLE	APPROXIMATE BEDROCK SURFACE CONTOURS
DATE	NOV. 2005		
DESIGN			
CAD	JFC		
CHECK	CMG		HIGHWAY 69/HIGHWAY 559 UNDERPASS
REVIEW	JPD	FIGURE	2

FILE No. 031111028EC002.dwg
PROJECT No. 03-1111-028-1 REV. A

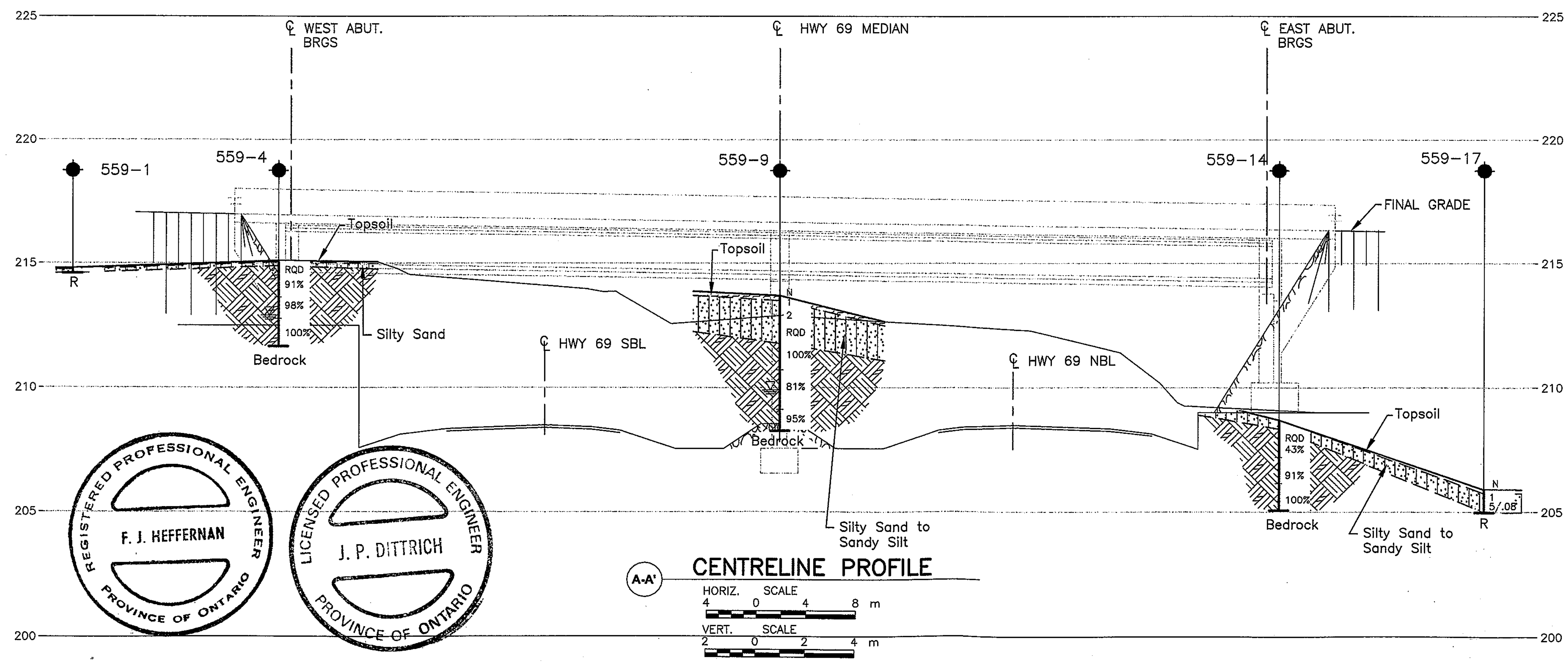
PLOT DATE: November 08, 2005
FILENAME: T:\Projects\2003\03-1111-028 (URSCI, Parry Sound)\-EC- (559 FINAL)\031111028EC002.dwg

DRAWINGS



PLAN

SCALE 4 0 4 8 m



A-A' CENTRELINE PROFILE

HORIZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m

CONT No.
WP No. 335-00-00

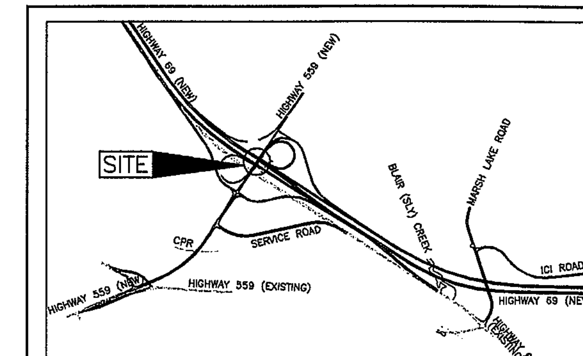
HIGHWAY 559
PROPOSED HIGHWAY 69 UNDERPASS

BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
500 0 500 1000m

LEGEND

●	Borehole – Current Investigation
R	Refusal
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
559-1	214.8	5033556.7	255317.4
559-2	214.7	5033572.8	255320.0
559-3	214.7	5033575.3	255321.6
559-4	215.1	5033570.5	255326.4
559-5	215.5	5033565.7	255331.0
559-6	215.2	5033568.2	255332.6
559-7	212.9	5033606.2	255341.8
559-8	213.0	5033608.8	255343.5
559-9	213.7	5033604.0	255348.2
559-10	213.6	5033599.2	255352.9
559-11	213.6	5033601.7	255354.5
559-12	209.0	5033639.7	255363.7
559-13	208.9	5033642.2	255365.3
559-14	208.7	5033637.5	255370.1
559-15	208.7	5033632.7	255374.7
559-16	208.6	5033635.2	255376.4
559-17	205.9	5033651.3	255379.1

NOTES

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

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

For subsurface information only.

REFERENCE

Base plans provided in digital format by URS, drawing file no. HWY 69_559_ga_jan27.dwg, dated Jan. 2005 received Jan. 27, 2005.

NO.	DATE	BY	REVISION

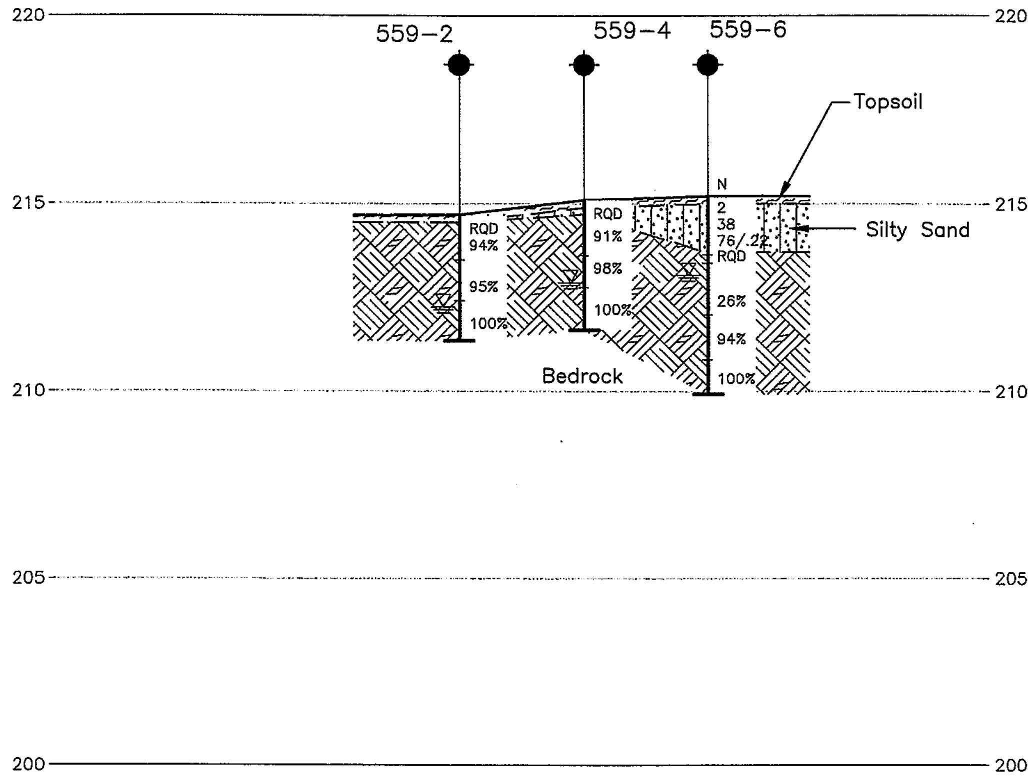
Geocres No. 41H-49

HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: NOV. 2005	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1A

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

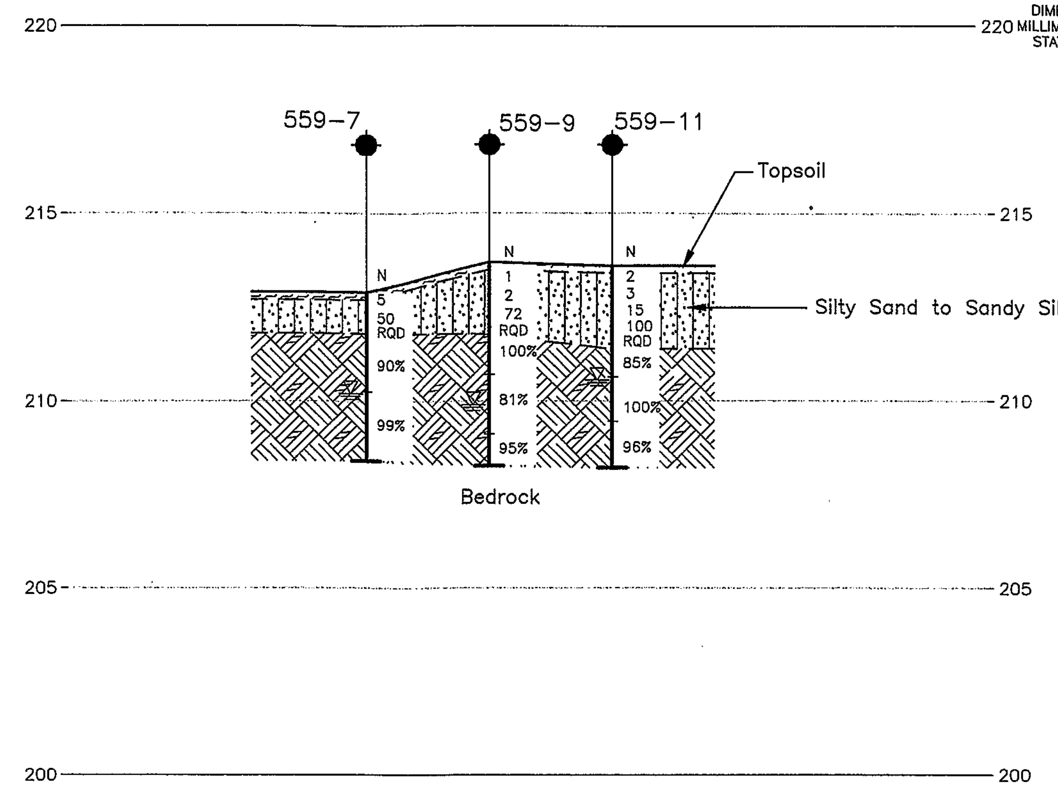
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



B-B' 1A WEST ABUTMENT

HORZ. SCALE 4 0 4 8 m

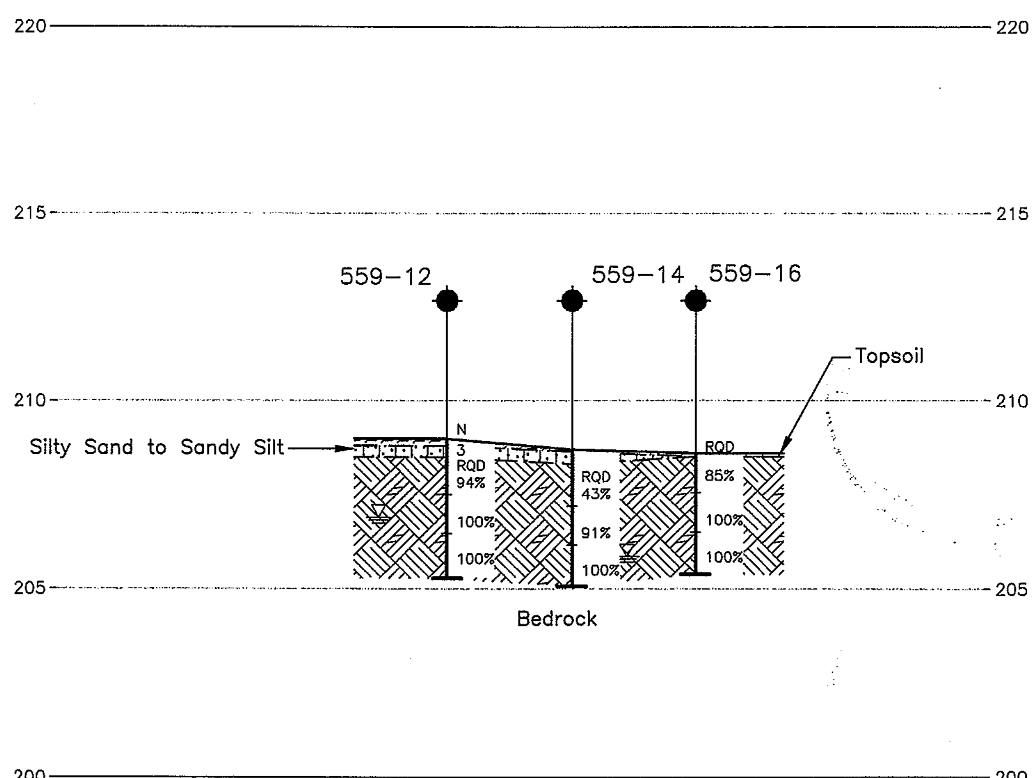
VERT. SCALE 2 0 2 4 m



C-C' 1A PIER

HORZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m



D-D' 1A EAST ABUTMENT

HORZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
PROVINCE OF ONTARIO

CONT No.
WP No. 335-00-00

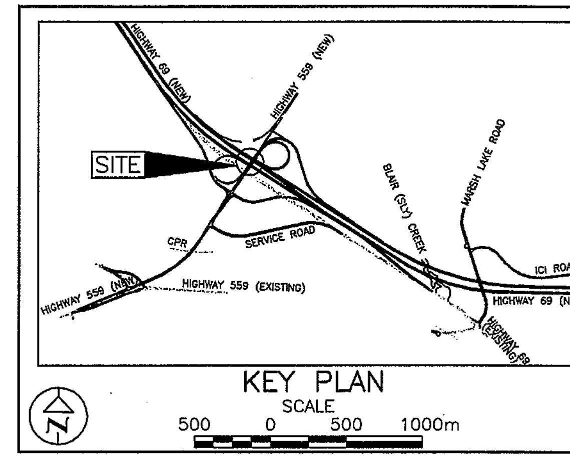
HIGHWAY 559
PROPOSED HIGHWAY 69 UNDERPASS

CROSS SECTIONS

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE 500 0 500 1000m

LEGEND

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

No.	ELEVATION	NORTHING	EASTING
559–1	214.8	5033556.7	255317.4
559–2	214.7	5033572.8	255320.0
559–3	214.7	5033575.3	255321.6
559–4	215.1	5033570.5	255326.4
559–5	215.5	5033565.7	255331.0
559–6	215.2	5033568.2	255332.6
559–7	212.9	5033606.2	255341.8
559–8	213.0	5033608.8	255343.5
559–9	213.7	5033604.0	255348.2
559–10	213.6	5033599.2	255352.9
559–11	213.6	5033601.7	255354.5
559–12	209.0	5033639.7	255363.7
559–13	208.9	5033642.2	255365.3
559–14	208.7	5033637.5	255370.1
559–15	208.7	5033632.7	255374.7
559–16	208.6	5033635.2	255376.4
559–17	205.9	5033651.3	255379.1

NOTES

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

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

For subsurface information only.

REFERENCE

Base plans provided in digital format by URS, drawing file no. HWY 69_559_ga_jan27.dwg, dated Jan. 2005 received Jan. 27, 2005.

NO.	DATE	BY	REVISION

Geocres No. 41H-49

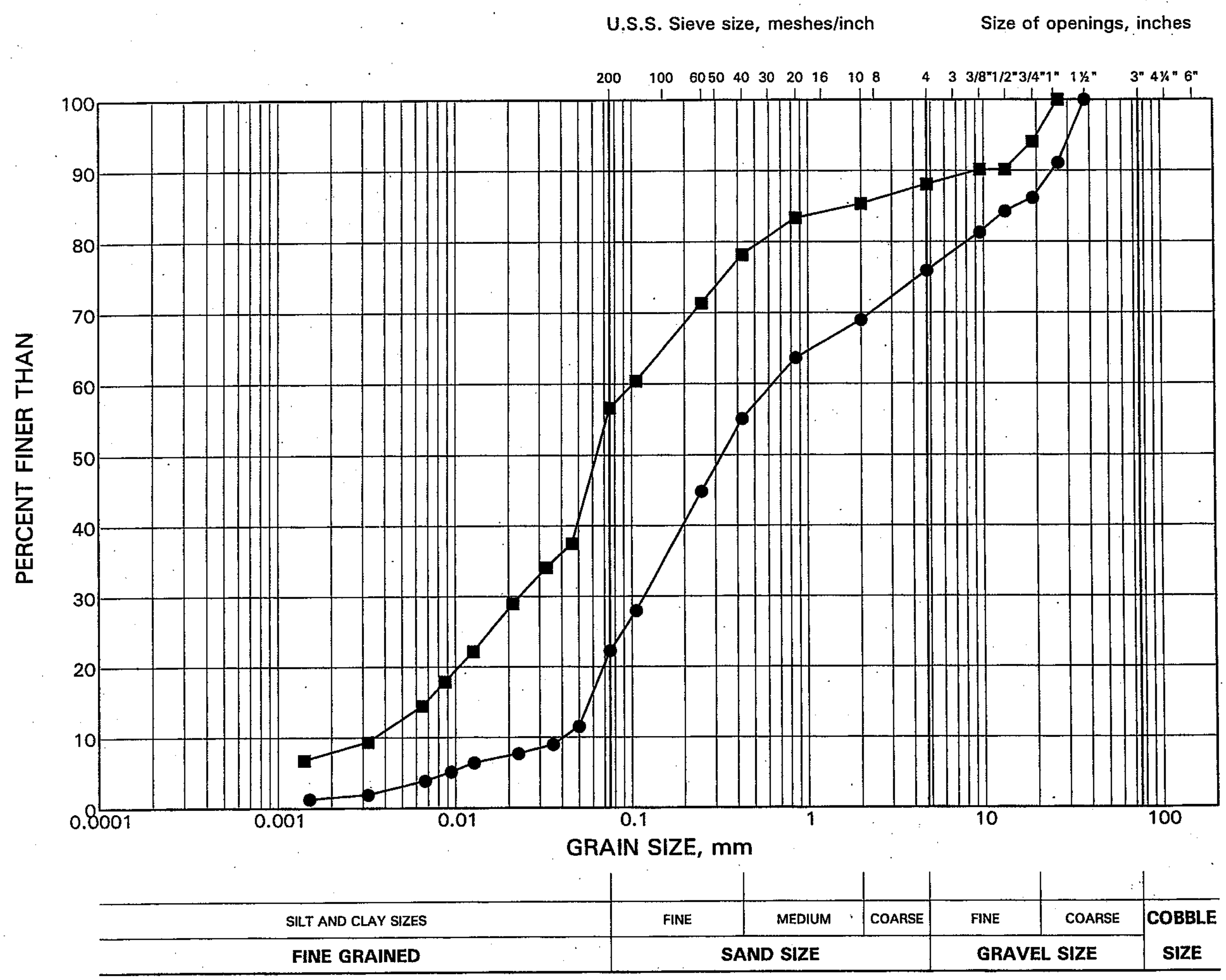
HWY. 69		PROJECT NO. 03-1111-028	DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: NOV. 2005	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1B

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Silty Sand to Sandy Silt, some gravel

FIGURE A-1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	559-6	2	214.1
■	559-11	3	212.1

Date November, 2005
Project 03-1111-028

Golder Associates

Prepared by LG
Checked by ________

APPENDIX B

SAMPLE NON-STANDARD SPECIAL PROVISIONS

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the East and/or West abutment footings and the Central column footing.

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.

<u>EARTH EXCAVATION FOR STRUCTURE – Item No.</u>
<u>ROCK EXCAVATION FOR STRUCTURE – Item No.</u>
<u>UNWATERING STRUCTURE EXCAVATION – Item No.</u>

Special Provision No. 902S01M

Excavation and Backfilling-Structures

902.02 REFERENCES

Section 902.02 of OPSS 902, December, 1983, is amended by the addition of the following:

OPSS 510

902.03 DEFINITIONS

Section 902.03 of OPSS 902, December, 1983, is amended by the addition of the following:

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

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section 902.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

902.04.02 Working Drawings

Working Drawings for protection systems shall be according to OPSS 539.

Where unwatering is required, the Contractor shall be responsible for the design of the unwatering scheme for the intended purpose. The design of temporary structures or protection system for unwatering shall be according to OPSS 539.

902.04.03 Submission of Certificate of Conformance

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

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

- Excavation for Foundation
- Excavation for Backfill and Frost Tapers
- Use of Excavation Material
- Backfilling

The Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

The Contractor shall be responsible for ensuring the quality of the material used for backfill. The quality of the material shall be verified by test results from a qualified and recognized testing laboratory. The frequency of sampling and testing shall be according to ASTM D75-87 and D3665.

902.05.04 Protection System

Section 902.05 of OPSS 902, December, 1983, is amended by the addition of the following:

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

Subsection 902.07.01 of OPSS 902, December, 1983, is amended by replacing the word "Engineer" in the last paragraph with the words "Contract Administrator".

902.07.02 Excavation

Subsection 902.07.02 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.07.02.01 General

For excavation, the Contractor shall be responsible for preventing any deterioration of the foundation soil or rock, surface water from entering and eroding the face of the excavation, and build up of hydrostatic pressures which may have harmful effects upon the temporary or permanent structures.

902.07.02.02 Excavation for Foundation

The excavation for foundation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract administrator.

The Contractor shall be responsible for maintaining the stability of the excavation if any excavation below a stream or channel bed is carried out.

902.08 Measurement for Payment

902.09.01 Structures

Subsection 902.09.01 of OPSS 902, is amended by deleting the first five paragraphs and replacing them with the following:

"Earth Excavation for Structure" and "Rock Excavation for Structure" applies to the specific structure(s) designated, i.e., Bridge, Retaining Wall or Culvert, and is measured by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the volume in cubic metres below the designated payment surface.

The above measurement also includes, where applicable, the excavation quantities, below the designated payment surface, for placing granular backfill and for placing the granular frost tapers.

For open footing culverts, the above measurement also includes the excavation quantities below the designated payment surface but between the plan areas of the footings and above a stream bed or the top of the footings, whichever is higher.

Where the structure excavation overlaps excavation required for other work, deductions will not be made to the structure excavation measurement.

902.10 Basis of Payment

902.10.01 Excavation and Backfill

Subsection 902.10.01 of OPSS 902 is amended by deleting the first paragraphs and replacing it with the following:

Payment at the contract price(s) for the tender item(s) "Earth Excavation for Structure" and "Rock Excavation for Structure" shall be full compensation for all labour, equipment and material for all excavation required, for removal of pavement, curb and gutter and sidewalk except where there is a separate item for removal of pavement, curb and gutter and sidewalk which overlaps pavement, curb and gutter and sidewalk removal required for structure excavation, protection of adjacent works, unwatering backfilling and compacting around the footing according to subsection 902.07.04, placing and compacting of suitable material infill in accordance with OPSS 206 and management of any surplus or unsuitable excavated material, including the cost of disposal areas, all according to the requirements of this specification.

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

The Contractor shall be responsible for all additional costs due to excavation beyond the required tolerance limits, including but not limited to additional structure design, granular materials, concrete, reinforcing steel and retention of the services of a blasting consultant.

The Contractor shall be responsible for restoring the over excavated area to its original conditions. For over excavation in earth, the backfill materials shall be granular material such as Granular A or B compacted according to OPSS 501. For over excavation in rock, concrete shall be placed to achieve the original excavation limits. Te concrete shall be of the same class concrete as the element it supports.

902.07.02.03 Excavation for Backfill and Frost Tapers

Excavation for backfill and frost tapers shall be carried out according to the specifications and details shown on the contract drawings. The Contractor shall be responsible for restoring the over excavated portion with backfill and shall be compacted to OPSS 501.

The excavation for backfill and frost tapers shall be inspected and approved by the Quality Verification Engineer prior to placement of fill material. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator. .

902-07.02.04 Preservation of Channel

Where applicable, the Contractor shall be responsible for restoring a channel back to its original conditions unless other wise specified in the contract.

902.07.02.04 Removals

Where applicable, removal of pavement, curb and gutter, and sidewalks shall be according to OPSS 510.

902.07.03 Unwatering Structure Excavation

Subsection 902.07.03 of OPSS 902, December, 1983, is amended by replacing the first paragraph with the follows:

The Contractor shall carry out all work necessary to prevent disturbance to the founding material. Concrete shall be placed in the dry, unless otherwise specified in the contract.

After the unwatering, the excavation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

902.07.04 Backfilling

Subsection 902.07.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

The Contractor shall ensure that the concrete has reached at least 70 percent of its design strength before placing the backfill against an abutment, wingwall, retaining wall or concrete culvert.

Backfilling shall be according to OPSS 501.

The backfilling shall be according to OPSS 501.

The backfilling operation shall be inspected and approved by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

DOWELS Into Rock – Item No.

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).

Where dowels are to be placed in 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 rock dowels 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 69/Highway 559 Underpass	West Abutment	2
Highway 69/Highway 559 Underpass	Central Pier	1
Highway 69/Highway 559 Underpass	East Abutment	2

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.

Special Provision

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 0.0025 cm. 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.