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REPORT ON

DETAILED FOUNDATION INVESTIGATION AND DESIGN HIGHWAY 559 / CANADIAN PACIFIC RAILWAY OVERHEAD 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:

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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 foundation investigation as part of the detailed design for the new Highway 559 / Canadian Pacific Railway (CPR) single-span overhead 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 terms of reference for the 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 overhead structure at the intersection of new Highway 559 and the Canadian Pacific Railway (approximate track mileage 31.95 of the Parry Sound Subdivision) was provided to Golder by URS on November 5, 2004.

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

The purpose of this investigation is to establish the subsurface conditions in the area of the proposed structure 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.

2.0 SITE DESCRIPTION

The site is located approximately 650 m west of existing Highway 69 and north of existing Highway 559 (north of Nobel, Ontario) at the intersection of the proposed new Highway 559 alignment and the Canadian Pacific Railway (approximate track mileage 31.95 of the Parry Sound Subdivision). The general location of the Highway 69 and Highway 559 study area is shown on the Site Location Map on Figure 1.

In general, the topography in the overall project study area consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed overhead structure site is located within an open grassy area where bedrock is at relatively shallow depth or outcrops at ground surface. The ground surface in the area of the proposed bridge is generally between about Elevation 213 m (to the east of the existing railway alignment) and Elevation 207 m (to the west of the existing railway alignment), referenced to Geodetic Datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed Highway 559 / CPR overhead structure investigation was carried out between November 21 and December 1, 2004 during which time thirteen (13) sampled boreholes (CPR-1 to CPR-6, CPR-8, CPR-9 and CPR-12 to CPR-16), three (3) shallow hand excavations (CPR-7, CPR-11 and CPR-18) and two (2) probe holes (CPR-10 and CPR-17) were put down at the site. Ten (10) of the explorations were advanced at the proposed east and west abutment footing locations (five per foundation element - including one at each corner of the foundation units and one in the central portion of each foundation unit), six (6) were advanced at the proposed retaining wall locations (three per retaining wall) and one (1) was advanced at each of the east and west approach embankments.

The boreholes were drilled using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario and 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 was inferred to be relatively shallow, hand excavations or probe holes were advanced instead of boreholes in order to confirm the depth to bedrock. All of the explorations were advanced to refusal on inferred bedrock. At each abutment, bedrock coring was carried out at three (3) of the investigated locations to a minimum depth of 3 m obtaining samples of the bedrock 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 3.8 m below the existing ground surface (not including rock coring). At investigated locations CPR-2, CPR-4, CPR-6, CPR-8, CPR-9 and CPR-10, located within the footprints of the proposed foundation units, the depth of investigation was further advanced into the bedrock by coring about 3.4 m to 4.2 m. The groundwater level in the open boreholes / drillholes was observed throughout drilling operations and a piezometer was installed in CPR-6 to permit monitoring of the groundwater level at this location. The piezometer consists of 38 mm O.D. threaded PVC tubing with a slotted screen at depth and was backfilled with a sand filter and sealed with bentonite within the borehole. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. All boreholes were abandoned, and the piezometer will be 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, coordinated with CPR flagmen, 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 selected samples of the overburden soils. Point load index strength testing was carried out on specimens from the rock core.

All investigated locations were staked 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 established by Callon Dietz at the staked locations. 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 559 / CPR overhead structure is shown on Drawings 1A and 1B.

In general, the subsoils at the structure site consist of a surficial layer of topsoil underlain by a relatively thin deposit of silty sand to gravelly sand containing cobbles and boulders, subsequently underlain by bedrock. The total overburden thickness ranges from 0 m (i.e. no cover with bedrock outcropping at ground surface) to about 3.8 m. All of the boreholes, hand excavations and probe holes were extended to the inferred bedrock surface; six (6) investigated locations at foundation areas were cored at least 3 m into the bedrock.

In the area of the west approach embankment, retaining wall and abutment, the depth of the bedrock ranges from about 2.4 m to 3.8 m below the existing ground surface.

In the area of the east approach embankment, retaining wall and abutment, the depth to bedrock ranges from 0 m (exposed at the ground surface) to 1.2 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 at ground surface. The surface of the topsoil (i.e. ground surface) ranged between Elevations 212.0 m and 207.8 m and the thickness ranged between 0.1 m and 0.5 m.

4.2.2 Silty Sand to Gravelly Sand

A brown to brown and grey to grey silty sand to gravelly sand deposit containing cobbles and boulders was encountered below the topsoil at the majority of the investigated locations where bedrock was not outcropping at ground surface. Layering was evident within this deposit with respect to colour and the material typically became coarser with depth. The presence of cobbles and boulders within the deposit was inferred by the grinding and resistance to augering through some portions of the deposit. The top of this deposit ranged from Elevations 211.7 m to 207.3 m and thickness ranged from about 0.1 m to 3.6 m. The bottom of this deposit was defined by refusal to further auger or sampler advancement 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 52 blows per 0.3 m of penetration. Lower blow counts were generally measured near ground surface; below a depth of about 0.5 m ‘N’ values are typically greater than 15 bows per 0.3 m of penetration. The ‘N’ values indicate a very loose to very dense relative density within the deposit; typically compact to very dense.

The natural water content measured on samples of this deposit ranged between 8 percent and 24 percent, with an average of about 17 percent.

Grain size distributions for five (5) samples from this deposit are shown on Figure A-1 of Appendix A.

4.2.3 Bedrock

Bedrock was cored at the locations of CPR-2, CPR-4, CPR-6, CPR-8, CPR-9 and CPR-10. The presence of bedrock was confirmed by hand excavations and was inferred from refusal to further drilling or sampler advancement or probe holes at the remaining investigated locations. The surface of the bedrock varies from ground surface (i.e. bedrock outcrop) to a depth of about 3.8 m. At the investigated locations, the bedrock surface ranges between about Elevation 211.6 m (to the east of the existing railway) and Elevation 204.3 m (to the west of the existing railway).

The bedrock samples are described as fresh to slightly weathered, light grey to pink, fine to coarse grained, non-porous to faintly porous granitic gneiss containing near horizontal, distinct foliation. The Total Core Recovery measured on the core samples was between 98 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples ranged from 38 percent to 100 percent, typically between 60 percent and 90 percent, indicating a rock mass of fair to good 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 1.8 MPa to 9.6 MPa, typically greater than 6 MPa, and diametral point load strength index values ranged from 2.1 MPa to 7.9 MPa, typically greater than 4 MPa, indicating a strong to very strong rock mass with medium strong zones. A summary of the point load index values on the rock core from the six (6) investigated locations where coring was carried out 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 associated approximate Unconfined Compressive Strength (UCS) value for each test.

Borehole (Drillhole) No.	**Average Axial Point Load Index (MPa)**	**Average Diametral Point Load Index (MPa)**
CPR-2	9.4	6.9
CPR-4	7.8	5.8
CPR-6	5.5	4.7
CPR-8	7.4	6.1
CPR-9	7.2	5.4
CPR-10	8.2	5.5

4.2.4 Groundwater Conditions

The water level in the piezometer sealed into the bedrock in borehole CPR-6 was measured at Elevation 207.7 m (1.2 m depth) on December 21, 2004. Water levels noted in the boreholes during drilling operations ranged from about Elevation 211.5 m to Elevation 207.0 m (0.4 m to 2.1 m depth). Details of the piezometer installation, groundwater conditions and water levels observed in the open boreholes 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 Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder and Ms. Anne Poschmann, P.Eng., a Principal with Golder. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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PART B

DETAILED
FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 559 / CANADIAN PACIFIC RAILWAY OVERHEAD
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
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Golder Associates

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed Highway 559 / CPR overhead 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 proposed Highway 559 / CPR overhead structure will consist of a single span, slab-on-girder bridge with a 40 m span length. Retaining walls are proposed adjacent to/north of the west abutment and adjacent to/south of the east abutment to retain the approach embankments from the proposed future CPR track.

Based on the information provided on the General Arrangement (GA) Drawing provided by URS on November 5, 2004, the grade of the proposed Highway 559 bridge deck varies between about Elevations 220.0 m and 219.2 m while the grade of the CPR track is at about Elevation 210.5 m. The proposed approach embankments will be about 12 m and 9 m in height at the west and east sides of the bridge, respectively. The existing ground surface varies from about Elevation 212.0 m to Elevation 207.8 m at the borehole locations.

The following sections discuss foundation design and construction options for the proposed structure and associated retaining walls and approach embankments.

5.2 Bridge Foundation Options

The subsoils at the bridge site generally consist of topsoil overlying very loose to very dense silty sand to gravelly sand containing cobbles and boulders up to about 3.8 m depth. The overburden soils are underlain by typically strong to very strong granitic gneiss bedrock (though medium strong zones exist). The bedrock surface at the proposed foundations, as established at the investigated locations, ranges between about Elevations 211.6 m and 210.5 m at the east abutment and about Elevations 207.5 m and 205.0 m at the west abutment (i.e. the average bedrock surface elevation at the east abutment area is approximately 5 m higher than at the west abutment area).

Given the shallow depth of the bedrock at the east abutment and the variable but limited thickness of the overburden at the west abutment, it is considered that the most appropriate foundation option is to support the bridge abutments on the granitic gneiss bedrock. Given the variability of the bedrock surface at the site, and the proposed road grade, the following founding options are considered feasible:

- Shallow spread footings on the bedrock – suitable for the east abutment but use at the west abutment would require excavations up to about 4 m deep to reach the bedrock surface;
- Perched abutments, founded on spread footings placed on compacted Granular A pads within the approach embankment fill; and
- Driven steel H-piles – driven through the approach embankment fill to found on the bedrock.

For integral abutments with the use of steel H-piles, a minimum pile length of 5 m is typically required to impart sufficient flexibility of the piles to accommodate bridge deck deflections. Given the proposed underside of pile cap at about Elevation 214 m, there is sufficient space for the 5 m pile length at the west abutment. With the bedrock surface elevation at the east abutment, however, up to about 2 m of bedrock subexcavation would be required to achieve the 5 m pile length.

Recommendations for spread footings (founded on bedrock and perched on granular pads) and for driven steel H-piles 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.

5.3 Spread Footings

The following table summarizes the range in bedrock surface elevation as encountered in the boreholes at the east and west abutments.

Foundation Element	*Borehole Numbers*	*Depth to Bedrock*	*Bedrock Surface Elevation*
East abutment	CPR-7 to CPR-11	0.0 m to 1.1 m	210.5 m to 211.6 m
West abutment	CPR-2 to CPR-6	2.4 m to 3.8 m	205.0 m to 207.5 m

As indicated above, there is some variability in the bedrock surface within the limits of each foundation element. In addition, the upper portion of the bedrock is, in a few local areas, moderately fractured (RQD values as low as 38 percent as encountered in borehole CPR-4) and it may be necessary to subexcavate loose or fractured rock from within some areas of the foundation footprints. For design, the following options may be considered:

1. Founding level based on the upper limit of the bedrock surface as encountered in the boreholes; the following design founding elevations may be assumed:

East Abutment:	Elevation 211.8 m
West Abutment:	Elevation 207.7 m

   In this case, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete would be placed as required 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. Founding level based on the lower limit of bedrock surface as encountered in the boreholes; the following design founding elevations may be assumed:

East Abutment:	Elevation 210.0 m
West Abutment:	Elevation 204.5 m

   In this case, following the removal of the overburden, excavation of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 3.0 m may be required at the west abutment. It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths typically in the range of 50 MPa to 220 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 5.11 for bedrock excavation/blasting recommendations).

3. As a third option, an intermediate founding level 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 5.11.

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. It should be noted that the groundwater level is relatively high (generally 0.4 m to 2.1 m below the existing ground surface) and groundwater control measures may be required to permit concrete placement in the dry, especially during periods of high groundwater levels.

As an alternative to spread footings on the bedrock or on mass concrete, consideration could be given to the use of spread footings perched within the approach embankments. This option would require construction of a compacted Granular A fill pad for support of the footing (i.e. as opposed to rock fill which may normally be used for embankment construction).

5.3.1 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 provision should be included in the Contract Documents to address the requirements for field inspection. Groundwater control measures may be required in order to carry out this inspection in the 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 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. Based on the site conditions, a value of 350 kPa may be assumed for design.

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 Steel H-Pile Foundations

As noted in Section 5.2, if steel H-piles are to be considered for support of an integral abutment at this site, bedrock excavation would be required at the east abutment to provide for adequate pile length. The granitic gneiss bedrock is typically strong to very strong and therefore boring large diameter (i.e. approximately 1 m diameter) sockets, although possible, may not be cost effective. Consideration should be given to trenching/excavating by drilling and blasting into the bedrock to provide a preformed slot into which backfill could be placed and the piles could be driven. It should be noted that groundwater control measures may be required in order to complete such excavation.

5.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles founded on the granitic gneiss bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of the driven H-piles, this value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at 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.

5.4.2 Resistance to Lateral Loads

Given the proposed grades and bedrock elevation at this site, the resistance to lateral loading will be mainly governed by the embankment fill and the backfill placed within the bedrock trench through which the piles are to be driven or placed. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equation for granular soils (assuming the bedrock excavation is backfilled with a granular material):

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

where
n_h is the constant of horizontal subgrade reaction, as given below;
z is the depth (m); and
B is the pile diameter (m).

As noted above, much of the lateral resistance of the piles will be provided solely by the unconfined Granular A pad built up above the existing ground surface. In this regard, the following recommendations are provided:

- The Granular A should be compacted to 100% of the Standard Proctor Maximum Dry Density for the material.
- The top of the Granular A pad should extend a horizontal distance of at least 5 times the pile diameter away from the edges of the pile in any direction.
- The Granular A pad should be constructed with side slopes no steeper than 1.5(H):1(V).

For these conditions,

- The upper portion of the Granular A pad (1.5 times the pile diameter down from the top of the pad) should be ignored for the calculation of lateral pile resistance.
- The depth 'z' in the equation k(h)=n(h) z/B should be calculated relative to the top of the Granular A pad adjacent to the pile caps.

The following range for the value of n_h may be assumed in the structural analysis:

Soil Unit	n_h
Native sand deposits and backfill around piles in bedrock trench excavation (assumed to be compacted granular fill below the groundwater level)	5 to 10 MPa/m
Granular A pad above existing ground surface	4.4

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

Pile Spacing in Direction of Loading d = Pile Diameter	***Reduction Factor***
8d	1.0
6d	0.7
4d	0.4
3d	0.25

5.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.5 Retaining Wall Foundations

The retaining wall adjacent to the east abutment (the SE wall) may be founded on the bedrock surface which is exposed at ground surface or is at shallow depth. Provision should be made for some bedrock excavation and/or mass concrete placement in the event that the bedrock surface varies along the length of the wall.

The retaining wall adjacent to the west abutment (the NW wall) could be founded at about Elevation 207.8 m, the upper limit of the bedrock surface encountered along the length of the wall. Following the removal of the overburden, to as low as about Elevation 206.7 m, the bedrock surface would have to be cleaned and then mass concrete would be placed as required to raise the grade to the founding level of Elevation 207.8 m. Alternatively, consideration could be given to supporting the NW wall on the native compact to dense sand deposit to minimize the extent of excavation required adjacent to the rail tracks. The footing would have to be placed below the upper very loose/loose portion of the deposit. It is understood that a founding level at Elevation 209 m is under consideration for this alternative. An analysis was carried out to assess the behaviour of a 4 m wide footing placed on compact to dense sand for the 25 m long NW retaining wall. The numerical method employed considered the effect of the relatively thin (on average 1.5 m) layer of sand under the footing which is underlain by bedrock.

It should be noted that, due to the proximity of the proposed NW wall to the existing railway tracks, both these foundation options will require railway protection (as per Section 5.10.1) to accommodate the subexcavation required to expose the founding level and/or to install a length of rigid insulation required for adequate frost protection as described in Section 5.5.1.

A summary of the advantages, disadvantages, relative costs and risks/consequences for the foundation alternatives of the NW wall is given in Table 3 following the text of this report.

The following design values may be assumed for the SE and NW retaining wall foundations:

Retaining Wall	***Foundation Option***	***Geotechnical Resistance at ULS***	***Geotechnical Resistance at SLS (for 25 mm of settlement)***
SE Wall	Founded on bedrock	10,000 kPa	- *
NW Wall	Founded on bedrock / mass concrete	10,000 kPa	- *
NW Wall	Founded on native sand	1,000 kPa	350 kPa

* The geotechnical resistance at 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.

It should be noted that the groundwater level during drilling was as much as 1.8 m above the proposed founding levels. Although the water level measured in the piezometer was at Elevation 207.7 m, it is expected that the groundwater table will be variable. Groundwater control may be required prior to excavating for the footing construction so that there is no disturbance/loosening of the founding stratum. This can be achieved by either groundwater lowering by pumping from wells or installation of interlocking sheetpiling to cut off the water inflow. Note that, due to the presence of cobbles/boulders in the subsoils in the area (as inferred from grinding of the augers during the foundation investigation), driving sheetpiling may be difficult at this location.

5.5.1 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 spread footings founded on the native compact to dense sand deposit, footings should be provided with a minimum of 1.8 m of soil cover for frost protection. At the NW wall, where the full 1.8 m of soil cover may not be possible due to the grade of a proposed drainage ditch adjacent to the wall, rigid insulation could be installed to compensate for the substandard conventional soil cover. It is estimated that 25 mm of rigid insulation is the equivalent of about 0.6 m of soil cover. Therefore, depending on the invert of the drainage ditch, it is estimated that between 50 mm and 75 mm of rigid insulation would be required. The insulation should be installed beneath the footing and extend to a distance of 1.8 m beyond the perimeter of the foundation unit (on all sides where there is inadequate soil cover). As noted above, railway protection may be required to accommodate subexcavation for the installation of the insulation in this area.

5.6 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.2, consistent with Soil Profile Type II.

5.7 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 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 relatively 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 west 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).

5.8 Approach Embankment Design

The construction of the Highway 559 / CPR overhead structure will require placement of up to about 12 m of fill within the limits of the west approach embankment and up to about 9 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 the east approach footprint) or a deposit of very loose to very dense silty sand to gravelly sand (typically less than 1.1 m deep for the east approach and less than 3.8 m deep for the west approach) underlain by bedrock. 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 alignment located east and immediately west of the proposed structure have been investigated as high fill embankment areas and are reported under separate cover.

5.8.1 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 5.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces 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 east approach area, bedrock is either outcropping or at shallow depth. At the west approach area, compact to dense cohesionless subsoils were encountered to a depth of about 3.5 m in the borehole advanced within the proposed embankment footprint. 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 and measured in a piezometer installed in CPR-6. In general, the groundwater level is located between 0.4 m and 2.1 m below the existing ground surface.

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 5.8.1.1.

East and West Approach Embankments

Soil Type	Unit Weight (kN/m3)	Strength Parameters
Rock Fill	19	$c' = 0$ kPa, $\phi' = 38^\circ$
Earth Fill (Sand and Gravel)	21	$c' = 0$ kPa, $\phi' = 35^\circ$
Compact to Dense* Silty Sand to Gravelly Sand	20	$c' = 0$ kPa, $\phi' = 33^\circ$

Note: *as encountered in borehole CPR-1

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*
West Approach	12	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
East Approach	9				

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.

5.8.1.1 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.

5.8.1.1.1 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.

5.8.1.1.2 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 overall project site, 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.

5.8.2 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 and west approach embankment toes (i.e. with the zones of silty sand with 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 which will occur at the toe of the embankment slopes). 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 the upper loose subsoils (typically less than 1 m thick) prior to construction of the approach embankments in order to eliminate the potential for seismically induced liquefaction at the embankment toes.

5.8.3 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 12 m and 9 m at the west and east approaches, respectively. The unit weights and slope profiles for the embankment fill described in Section 5.8.1 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 east approach embankment area, bedrock is either outcropping or at very shallow depth and the thin overburden soils present near the east abutment will largely be removed as part of the excavation required for the abutment footing construction. As such, the east approach embankment will be founded primarily on bedrock. At the investigated location for the west approach area, the compact to dense cohesionless subsoils are up to about 3.5 m thick. Surficial deposits of topsoil were encountered at the majority of the investigated locations.

Provided that the surficial topsoil is removed prior to the new embankment fill placement, settlements of the new approach embankments, due to compression of the 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.

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

5.8.3.1 Settlement of Cohesionless Foundation Soils

The immediate compression of the compact to dense silty sand to gravelly sand native subsoils encountered in the borehole advanced for the west approach was 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)*
East Approach	9+310	9	---**
West Approach	9+265	12+0.5 = 12.5	100

Notes : *includes additional fill required after removal of maximum depth of organics/topsoil
**minimal 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.

5.8.3.2 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, for the up to 12 m high approach embankments, will be about 1% of the new effective height of rock fill as shown below.

Location of Embankment	*Approximate Chainage*	*Maximum New Embankment Height* (m)*	*Estimated Settlement of Embankment Soils (mm)*
East Approach	9+310	9	90
West Approach	9+265	12+0.5 = 12.5	125

Notes : *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.

5.9 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.

5.9.1 Removal of Organics

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil) of up to about 0.5 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.

5.9.2 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. Blading, dozing and 'chinking' the rock to form a dense, compact mass will be required to minimize voids and bridging. 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.

5.10 Design and Construction Considerations

5.10.1 Excavation

Depending on the foundation option adopted, excavations for the bridge foundations will extend to depths of up to 3.8 m and will be made through typically compact to very dense sands and may be extended into the underlying bedrock.

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 1H:1V. Below the groundwater table, shallower side slopes (no steeper than 2H:1V) will be required unless prior dewatering is carried out. In this regard, excavations at this site will typically be below the groundwater table and as such 2H:1V side slopes will be required.

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 and/or retaining wall foundations. All temporary excavation support system(s) should be in accordance with OPSS 539. Roadway protection, where required, should be in accordance with OPSS 539 Performance Level 3. Railway protection should be designed in accordance with the guidelines of the latest AREMA Manual and the proposed installation procedures must be acceptable to the railway owner.

It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of 50 MPa to 220 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 carried out using line drilling and pre-shearing techniques (as discussed in Section 5.11). 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.

5.10.2 Groundwater and Surface Water Control

The groundwater level at the site is generally at about 0.4 m to 2.1 m below the existing ground surface. At the west abutment location, excavations carried down to the bedrock surface will require groundwater control to permit placement of mass concrete and construction of footings in the dry. At the east abutment location, the formation of a trench within the bedrock to allow installation of piles and placement of Granular A will require groundwater control.

As mentioned previously, dewatering and shoring or sheetpile cut-off is required prior to excavating for the NW retaining wall footing, founded within the sand deposit or on the bedrock, to permit placement of mass concrete and construction of footings in the dry. It is likely that pumping from within trenches/ditching with adequately sized and properly filtered pumps will be sufficient to control the groundwater inflow. For the case of footings founded on the sand deposit, dewatering (to lower the groundwater table) will be required prior to the start of the footing excavation. The dewatering system should be designed and installed by a qualified dewatering specialist. Surface water should be directed away from the excavations at all times.

5.10.3 Obstructions

The native subsoils at the site are expected to contain cobbles and boulders as was inferred from grinding of the augers during borehole advancement.

Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils. However, the presence of the boulders may interfere with or slow the progress of stripping and excavation. The presence of such obstructions may also affect the excavation works and the installation of piles. Ultimately, provision will have to be made in the Contract Drawings to ensure that the Contractor is equipped to handle such obstructions.

5.11 Blasting Recommendations for Rock Excavations

5.11.1 Excavation Considerations

For excavations into the bedrock, the overall slope to the cut face may be formed vertical or near vertical (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.

5.11.2 Special Provisions

5.11.2.1 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). In addition to any specific requirements that CPR may impose, 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.

5.12 CLOSURE

This report was prepared by Mr. Chad Gilfillan, E.I.T. and technical aspects were reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder and Ms. Anne S. Poschmann, P.Eng., a Principal with Golder. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted an independent audit review of the report.

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

CMG/JPD/ASP/FJH/cmg/sm
n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\5 - highway 559-cp bridge\03-1111-028-5 final rpt dec05 hwy559-cpr bridgereport.doc

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RECORD OF BOREHOLE SHEETS

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.

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan\phi'$

2 Shear strength = (Compressive strength)/2

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
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
|| - Parallel To
⊥ - Perpendicular To

P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

RECORD OF BOREHOLE No CPR-1

1 OF 1 **METRIC**

PROJECT	03-1111-028		
G.W.P.	335-00-00	LOCATION	N 5032987.2 ;E 254928.0
DIST	52 HWY 69	BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger
DATUM	Geodetic	DATE	December 1, 2004

ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

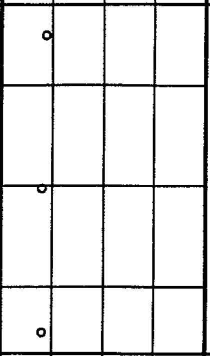
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
207.8	GROUND SURFACE									
0.0	Topsoil, some sand, containing cobbles	1	SS	1						
207.3 0.5	Sand, some gravel, trace silt Dense to compact Brown to grey, oxidized Wet				∇	207				
		2	SS	41						
		3	SS	32		206	20	72		(8)
		4	SS	21		205				
204.3		5	SS	21						
3.5	END OF BOREHOLE AUGER REFUSAL SPOON REFUSAL									

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 (%)

Note:

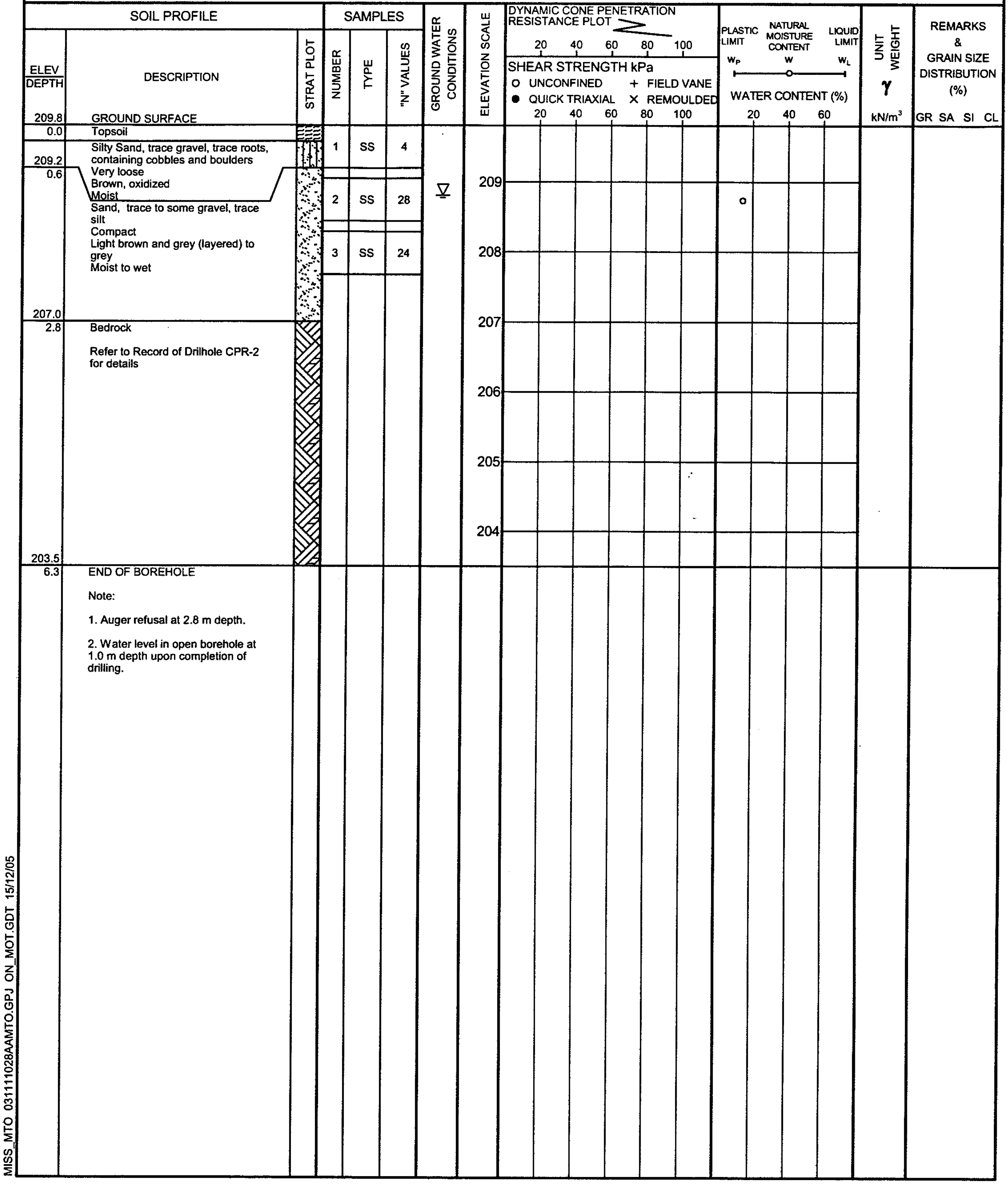
1. Water level in open borehole at 0.8 m depth upon completion of drilling.

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

RECORD OF BOREHOLE No CPR-2

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033001.3 ;E 254932.9
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 29, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
209.8	GROUND SURFACE			
0.0	Topsoil			
	Silty Sand, trace gravel, trace roots, containing cobbles and boulders Very loose Brown, oxidized Moist	1	SS	4
209.2 0.6	Sand, trace to some gravel, trace silt Compact Light brown and grey (layered) to grey Moist to wet	2	SS	28
		3	SS	24
207.0 2.8	Bedrock Refer to Record of Drillhole CPR-2 for details			
203.5 6.3	END OF BOREHOLE			

Note:

1. Auger refusal at 2.8 m depth.

2. Water level in open borehole at 1.0 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: CPR-2

SHEET 1 OF 1

LOCATION: N 5033001.0 ;E 254932.9

DRILLING DATE: November 29, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

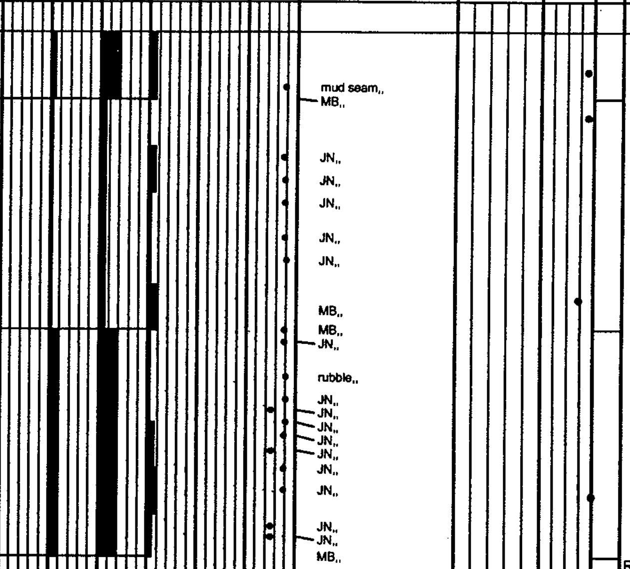
DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		206.96					
3	NQ CORING	Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation Appx. 40 % mafic minerals, 30 % feldspar, 30 % quartz		2.84	1				
4					2				
5					3				
6				203.50 6.30					
		END OF DRILLHOLE							Refer to Record of Borehole CPR-2
7									
8									
9									
10									
11									
12									

RECOVERY: TOTAL CORE % / SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT INDEX PER 0.3 m (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.



DEPTH SCALE
1 : 50

LOGGED: EHS
CHECKED: CG

RECORD OF BOREHOLE No CPR-3

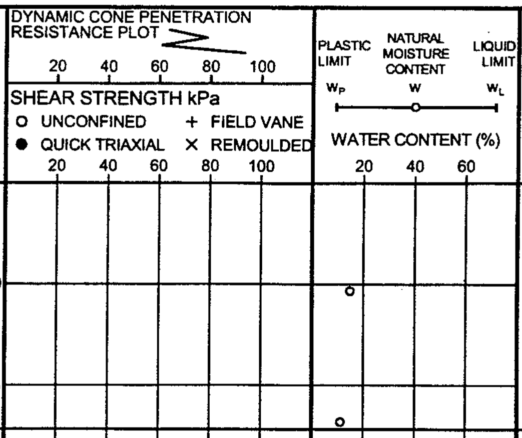
1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033002.6 ;E 254934.1
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 29, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.0	GROUND SURFACE							
0.0	Topsoil							
0.2	Silty Sand, trace gravel, cobbles and boulders, inferred by auger resistance Loose Oxidized Moist	1	SS	4				
209.4 0.6	Sand, trace gravel, trace silt, cobbles and boulders, inferred by auger resistance Compact to dense Light brown and grey (layered)	2	SS	32		209		
		3	SS	5	∇	208		
207.6		4	SS	29/.07				
2.4	END OF BOREHOLE AUGER REFUSAL SPOON REFUSAL							

Note:

1. Water level in open borehole at 1.8 m depth upon completion of drilling.



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

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 15/12/05

RECORD OF BOREHOLE No CPR-4

1 OF 1 **METRIC**

PROJECT 03-1111-028

G.W.P. 335-00-00

DIST 52 HWY 69

DATUM Geodetic

LOCATION N 5032998.1 ;E 254938.1

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

DATE November 30, 2004

ORIGINATED BY EHS

COMPILED BY DD

CHECKED BY CG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
209.3	GROUND SURFACE												
0.0	Topsoil												
	Silty sand, some gravel, trace organics, roots Loose Oxidized Moist	1	SS	3		209							
208.7 / 0.6	Sand, some gravel to Gravelly Sand, trace to some silt, cobbles and boulders, inferred by auger resistance Compact to very dense Light brown and grey, oxidized (layered) Wet	2	SS	52		208		○					
		3	SS	21	∇	207							
		4	SS	48				○		25	47	26	2
206.0	Rock fragment in tip of spoon	58	SS	100/.10									
3.3	Bedrock Refer to Record of Drilhole CPR-4 for details					206							
						205							
						204							
						203							
						202							
201.8 / 7.5	END OF BOREHOLE												

Note:

1. Auger and spoon refusal at 3.3 m depth.
2. Water level in open borehole at 2.1 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: CPR-4

SHEET 1 OF 1

LOCATION: N 5032998.1 ;E 254938.1

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 30, 2004

DRILL RIG: TRACK CME 55

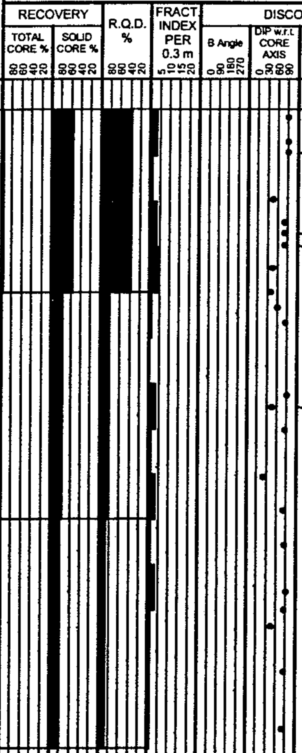
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

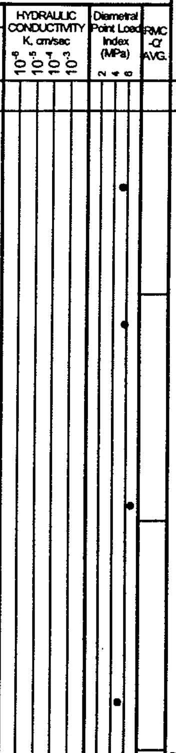
DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	206.00			
4	NQ CORING	Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation	3.30	1	rubble,, rubble,, JN,, JN,, JN,, JN,, JN,, JN,, MB,,	
5				2	JN,, JN,, JN,, JN,, JN,, JN,, MB,,	
6 7		Moderately weathered zone from 6.1 m to 6.7 m depth		3	JN,, JN,, JN,, JN,, JN,, MB,,	
		END OF DRILLHOLE	201.78 7.52			Refer to Record of Borehole CPR-4
8 9 10 11 12 13						





MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 15/12/05 JFC

DEPTH SCALE
1 : 50

Golder Associates

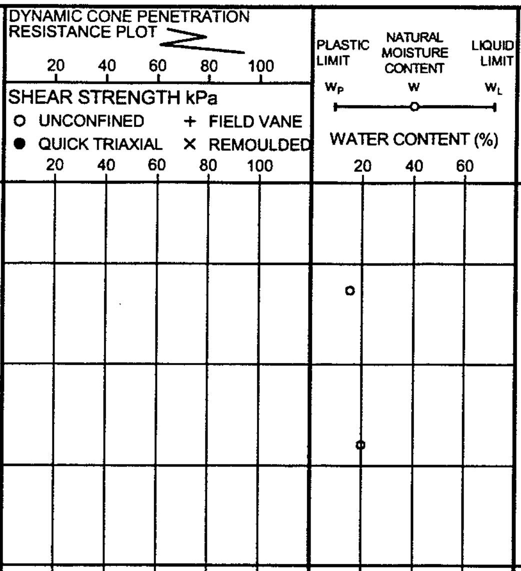
LOGGED: EHS
CHECKED: CG

RECORD OF BOREHOLE No CPR-5

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5032993.8 ;E 254942.3
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 30, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
208.8	GROUND SURFACE					
0.0	Topsoil	1	SS	3		
208.4	Silty sand, trace gravel Very loose Oxidized Moist					
208.0 0.8	Silty Sand to Sand, trace gravel, trace silt, cobbles, inferred by auger resistance Dense to compact Light brown and grey (layered), oxidized Wet	2	SS	33		208
					∇	
		3	SS	36		207
		4	SS	16		206
		5	SS	11		
205.0 3.8	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level in open borehole at 1.5 m depth upon completion of drilling.					



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

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

RECORD OF BOREHOLE No CPR-6

1 OF 1 **METRIC**

PROJECT 03-1111-028

G.W.P. 335-00-00

DIST 52 HWY 69

DATUM Geodetic

LOCATION N 5032994.8 ;E 254943.2

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

DATE November 30, 2004

ORIGINATED BY EHS

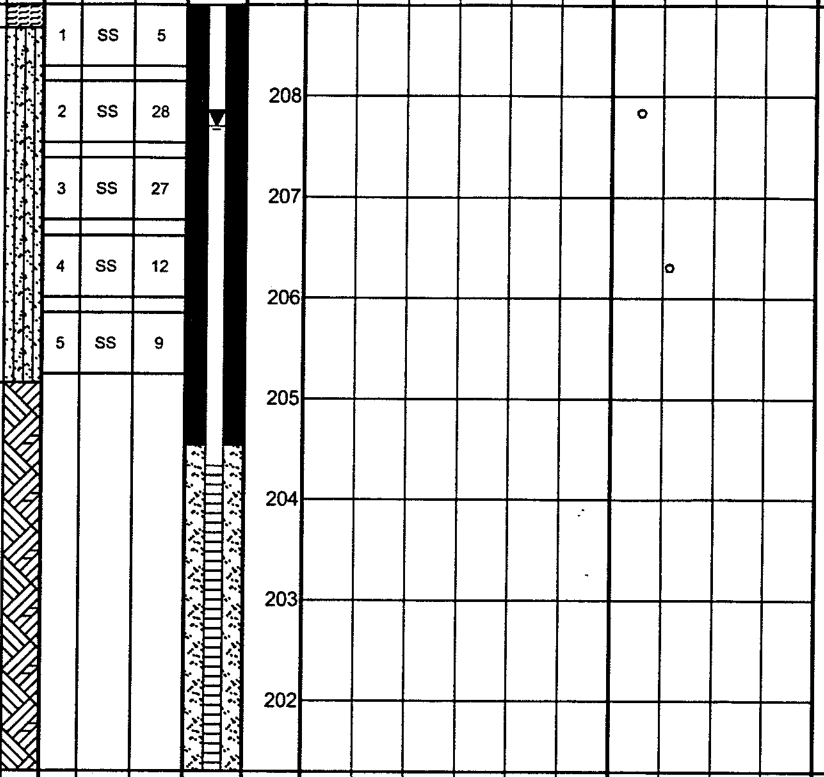
COMPILED BY DD

CHECKED BY CG

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
208.9	GROUND SURFACE							
0.0	Topsoil							
0.2	Silty Sand, some gravel to Sand, trace to some gravel, trace to some silt, cobbles inferred from auger resistance Compact to loose Light brown and grey (layered), oxidized	1	SS	5				
		2	SS	28	18	52	25	2
		3	SS	27				
		4	SS	12				
205.2		5	SS	9				
3.8	Bedrock Refer to Record of Drilhole CPR-6 for details							
201.3 7.6	END OF BOREHOLE							

Note:

1. Auger refusal at 3.8 m depth.
2. Water level in piezometer measured at 1.2 m depth on December 21, 2004.



+³, X³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: CPR-6

SHEET 1 OF 1

LOCATION: N 5032994.8 ;E 254943.2

INCLINATION: -90° AZIMUTH: —

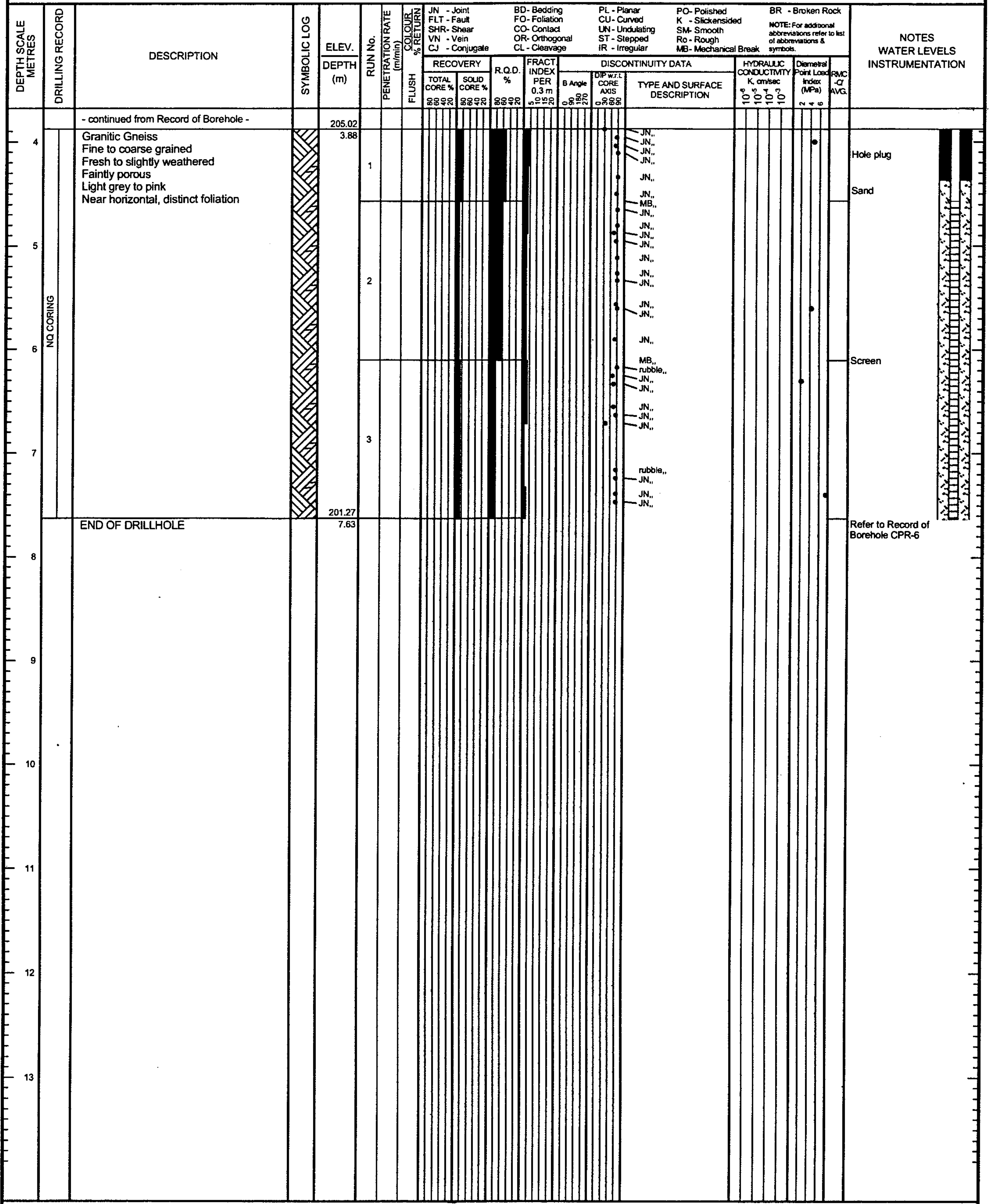
DRILLING DATE: December 1, 2004

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	



DESCRIPTION	ELEV. DEPTH (m)	NOTES WATER LEVELS INSTRUMENTATION
- continued from Record of Borehole -	205.02	
Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation	3.88	Hole plug Sand Screen
	201.27	
END OF DRILLHOLE	7.63	Refer to Record of Borehole CPR-6

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No CPR-7

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033031.9 ;E 254959.0
BOREHOLE TYPE Hand excavated
DATE November 21, 2004
ORIGINATED BY PH
COMPILED BY DD
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 × 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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
211.9	GROUND SURFACE													
0.0	Topsoil													
	Sand													
0.3	Hand excavated to bedrock													
							211							

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

RECORD OF BOREHOLE No CPR-8

1 OF 1 **METRIC**

PROJECT 03-1111-028

G.W.P. 335-00-00

LOCATION N 5033033.1 ;E 254960.0

ORIGINATED BY PH

DIST 52 HWY 69

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

COMPILED BY DD

DATUM Geodetic

DATE November 21, 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 × 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				
211.9	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, some gravel, containing cobbles Reddish brown to brown		1	AS		∇					
211.3 0.7	Bedrock Refer to Record of Drilhole CPR-8 for details						211 210 209 208				
207.4 4.5	END OF BOREHOLE Note: 1. Auger refusal at 0.7 m depth. 2. Water level in open borehole at 0.4 m depth upon completion of drilling.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: CPR-8

SHEET 1 OF 1

LOCATION: N 5033033.1 ;E 254960.0

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 21, 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

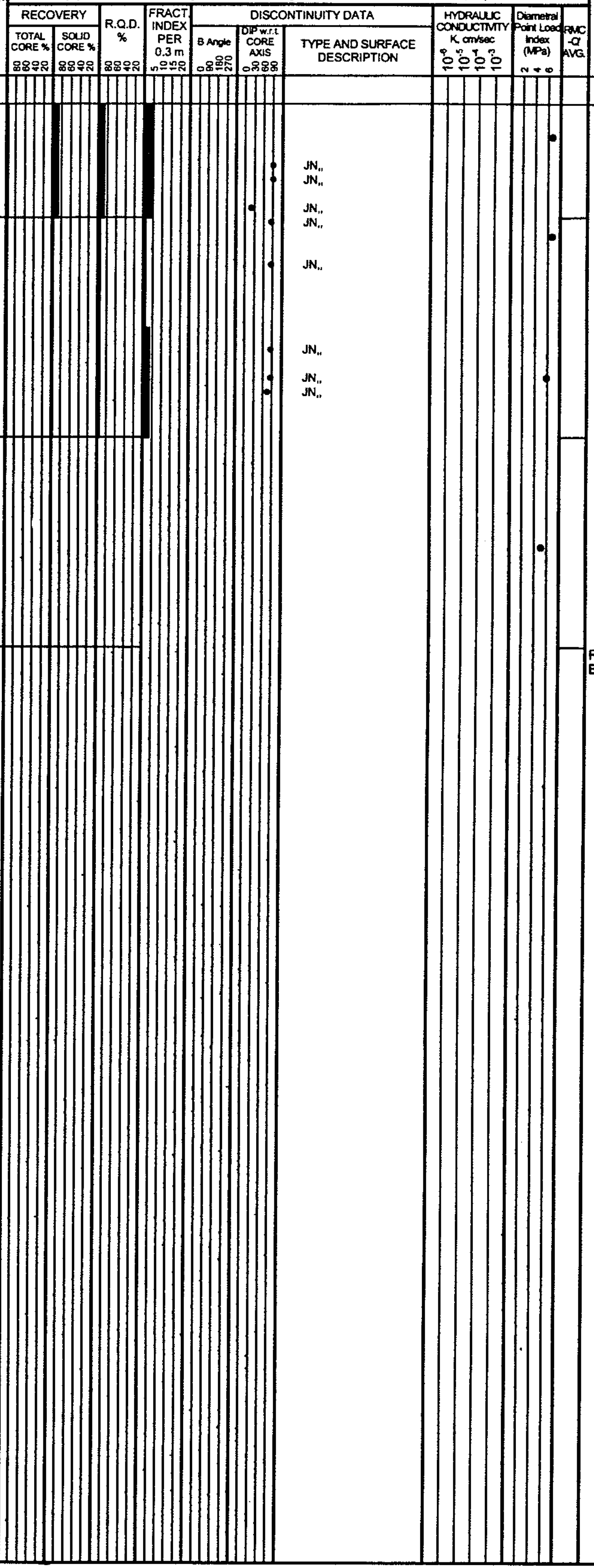
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular

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.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		211.23					
1 2 3 4	NQ CORING	Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation Appx. 40 % quartz, 30 % mafic minerals, 30 % feldspar (coarse grains)		0.67	1 2 3				
5 6 7 8 9 10		END OF DRILLHOLE		207.39 4.51					Refer to Record of Borehole CPR-8



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

RECORD OF BOREHOLE No CPR-9

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033028.6 ;E 2544964.0
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 21, 2004
ORIGINATED BY PH
COMPILED BY DD
CHECKED BY CG

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.6	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace to some gravel, containing cobbles and boulders Reddish brown Compact Moist to wet		1	SS	17	∇	211				
210.5 1.1	Bedrock Refer to Record of Drilhole CPR-9 for details						210 209 208				
207.1 4.5	END OF BOREHOLE Note: 1. Auger refusal at 1.1 m depth. 2. Water level in open borehole at 0.8 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: CPR-9

SHEET 1 OF 1

LOCATION: N 5033028.6 ;E 254964.0

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 21, 2004

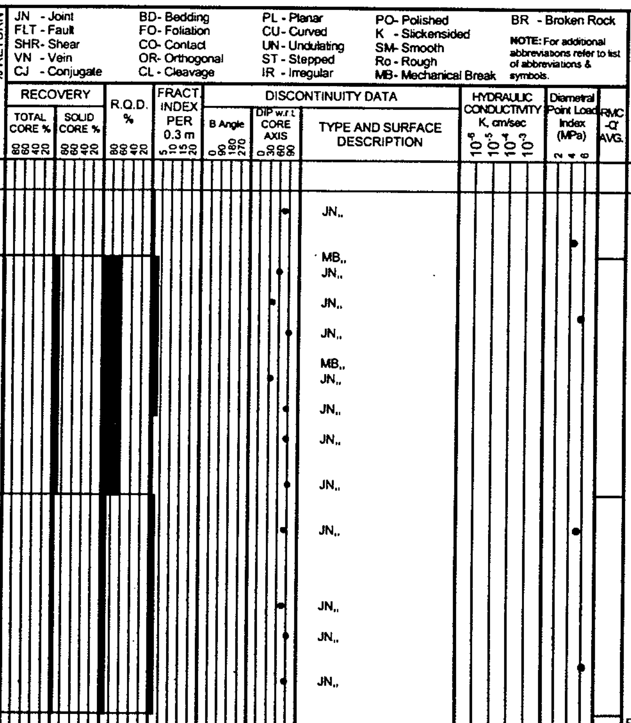
DRILL RIG: TRACK CME 55

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	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		210.53					
	NQ CORING	Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation Appx. 70 % quartz, 20 % mafic minerals, 10 % feldspar		1.07	1 2 3			JN,, MB,, JN,, JN,, JN,, MB,, JN,, JN,, JN,, JN,, JN,, JN,, JN,, JN,,	
		END OF DRILLHOLE		207.08 4.52					Refer to Record of Borehole CPR-9



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No CPR-10

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033024.2 ;E 254968.2
BOREHOLE TYPE Probe hole
DATE November 21, 2004
ORIGINATED BY PH
COMPILED BY DD
CHECKED BY CG

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

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

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: CPR-10

SHEET 1 OF 1

LOCATION: N 5033024.2 ;E 254968.2

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 21, 2004
DRILL RIG: TRACK CME 55
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

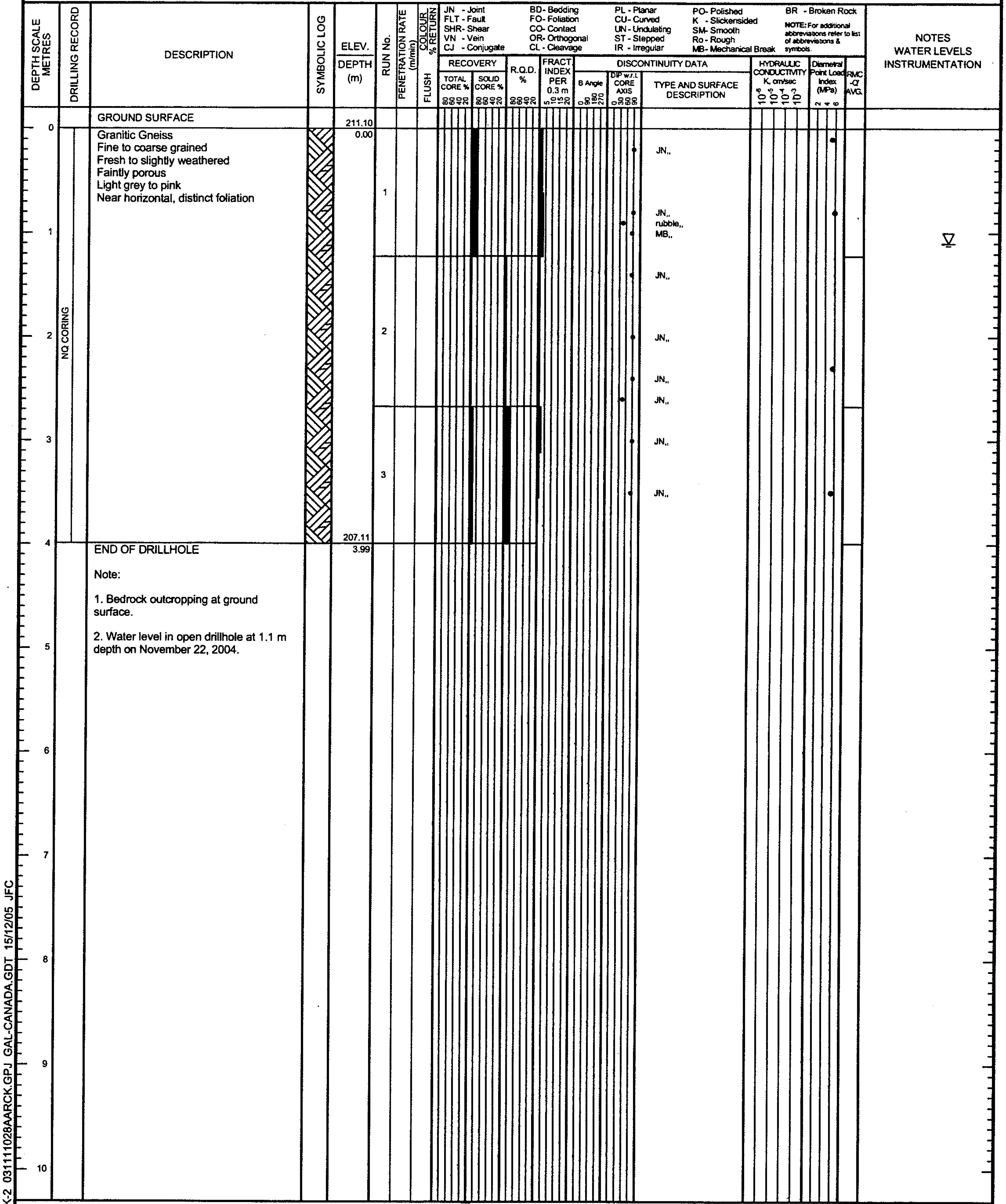
JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	NOTE: For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN - Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ -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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		GROUND SURFACE	211.10	
0	NQ CORING	Granitic Gneiss Fine to coarse grained Fresh to slightly weathered Faintly porous Light grey to pink Near horizontal, distinct foliation	0.00	1
				2
				3
4		END OF DRILLHOLE	207.11 3.99	

Note:

1. Bedrock outcropping at ground surface.

2. Water level in open drillhole at 1.1 m depth on November 22, 2004.



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No CPR-11

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033025.4 ;E 254969.2
BOREHOLE TYPE Hand excavated
DATE November 21, 2004
ORIGINATED BY PH
COMPILED BY DD
CHECKED BY CG

SOIL PROFILE			SAMPLES					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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
211.3	GROUND SURFACE										
0.0	Topsoil to Silty Sand, some organics	▨					211				
210.7 0.6	Hand excavated to bedrock										

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

RECORD OF BOREHOLE No CPR-12

1 OF 1 **METRIC**

PROJECT 03-1111-028

G.W.P. 335-00-00

DIST 52 HWY 69

DATUM Geodetic

LOCATION N 5033040.6 ;E 254973.4

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

DATE November 22, 2004

ORIGINATED BY PH

COMPILED BY DD

CHECKED BY CG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 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						
212.0	GROUND SURFACE										
0.0	Topsoil										
211.7 0.3	Silty Sand, some gravel, containing cobbles and boulders Reddish brown to greyish brown		1	SS	17						
211.2 0.8	END OF BOREHOLE SPOON REFUSAL AUGER REFUSAL						211				

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

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

Golder Associates

RECORD OF BOREHOLE No CPR-13

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033006.9 ;E 254920.4
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 29, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
210.4	GROUND SURFACE									
0.0	Topsoil									
209.8 0.6	Fine to medium Sand, trace silt, trace gravel Very loose Oxidized Wet	1	SS	2	∇	210				
	Fine to medium Sand, trace gravel, trace silt Compact Light brown (layered), oxidized Wet	2	SS	20			88	10	2	
		3	SS	18		209				
207.8		4	SS	9/0.23		208				
2.6	END OF BOREHOLE AUGER REFUSAL SPOON REFUSAL									

Note:

1. Water level in open borehole at 0.8 m depth upon completion of drilling.

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

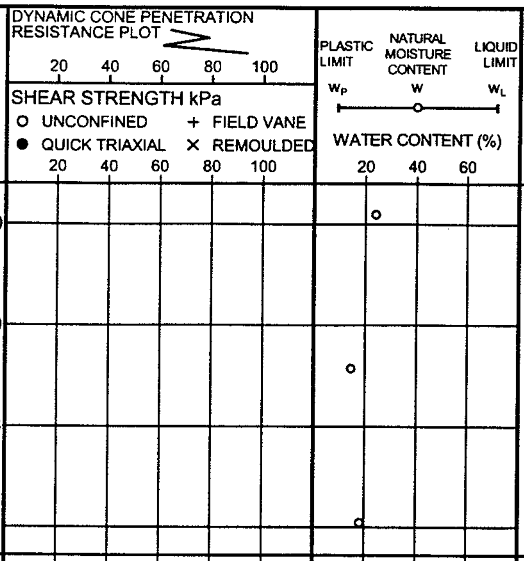
$+^3, \times^3$: Numbers refer to Sensitivity $○^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No CPR-14

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033006.2 ;E 254931.3
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 29, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
210.4	GROUND SURFACE									
0.0	Topsoil									
0.2	Sand, trace to some gravel, trace to some silt to Gravelly Sand, trace Silt Compact to dense Light brown and grey (layered) to grey Moist to wet	1	SS	4		210				
		2	SS	21	∇	209				
		3	SS	42			29	45	25	1
207.4		4	SS	36		208				
207.1	Silty Sand, some gravel Dense Grey Wet	5	SS	33		207				
206.7 3.7	Moderately to highly weathered Bedrock END OF BOREHOLE AUGER REFUSAL									



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Note:

1. Water level in open borehole at 1.3 m depth upon completion of drilling.

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

RECORD OF BOREHOLE No CPR-15

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
DIST 52 HWY 69
DATUM Geodetic
LOCATION N 5033005.5 ;E 254941.9
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE November 29, 2004
ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

SOIL PROFILE			SAMPLES					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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
210.5	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace gravel, containing cobbles Loose Oxidized Moist		1	SS	6		210				
209.9 0.6	Sand, trace to some gravel, trace silt, cobbles and boulders, inferred by auger resistance Dense Grey and light brown (layered) Wet		2	SS	72/0.18				○		
			3	SS	34		209				
208.2 2.3	Silty Sand, trace gravel Dense Grey and oxidized Wet		4	SS	43	∇	208		○		
207.5 3.1	END OF BOREHOLE AUGER REFUSAL										

Note:

1. Water level in open borehole at 2.3 m depth upon completion of drilling.

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

RECORD OF BOREHOLE No CPR-16

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
LOCATION N 5033021.0 ;E 254962.7
ORIGINATED BY PH
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE November 21, 2004
CHECKED BY CG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 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						
211.0	GROUND SURFACE										
0.0	Topsoil										
0.1 210.4	Sand, trace organics, containing cobbles/boulders Loose Reddish brown										
0.6 209.8	Silty Sand, some gravel Compact Grey-brown (layered) Wet		1	SS	19	∇	210		o		
1.2	END OF BOREHOLE SPOON REFUSAL AUGER REFUSAL										

Note:

1. Water level in open borehole at 0.9 m depth upon completion of drilling.

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

RECORD OF BOREHOLE No CPR-17

1 OF 1 **METRIC**

PROJECT 03-1111-028			
G.W.P. 335-00-00	LOCATION N 5033020.8 ;E 254972.3		ORIGINATED BY PH
DIST 52	HWY 69	BOREHOLE TYPE Probe hole	COMPILED BY DD
DATUM Geodetic	DATE November 21, 2004		CHECKED BY CG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 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						
210.5	GROUND SURFACE										
0.0	Bedrock outcrop										

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

Golder Associates

RECORD OF BOREHOLE No CPR-18

1 OF 1 **METRIC**

PROJECT 03-1111-028
G.W.P. 335-00-00
LOCATION N 5033020.3 ;E 254982.0
ORIGINATED BY PH
DIST 52 HWY 69
BOREHOLE TYPE Hand excavated
COMPILED BY DD
DATUM Geodetic
DATE November 21, 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 × 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 (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
209.9	GROUND SURFACE													
0.0	Topsoil								○					
0.3	Sand, some gravel, trace to some silt Reddish brown Hand excavated to bedrock						209							

MISS_MTO 03111028AAMTO.GPJ ON_MOT.GDT 15/12/05

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

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

TABLES

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

PROJECT NO.: 03-1111-028
LOCATION: Proposed Highway 559 / CPR Overpass
DATE: January 05, 2005

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
CPR-2	1	Granitic Gneiss	10.3	3.1	D	6.974	160
CPR-2	2	Granitic Gneiss	11.3	3.4	D	7.119	164
CPR-2	3	Granitic Gneiss	11.9	3.6	A	9.254	213
CPR-2	4	Granitic Gneiss	15.2	4.6	D	5.782	133
CPR-2	5	Granitic Gneiss	16.4	5.0	A	9.556	220
CPR-2	6	Granitic Gneiss	19.3	5.9	D	7.860	181
CPR-4	1	Granitic Gneiss	12.3	3.8	D	5.339	123
CPR-4	2	Granitic Gneiss	14.5	4.4	A	8.937	206
CPR-4	3	Granitic Gneiss	15.4	4.7	D	5.808	134
CPR-4	4	Granitic Gneiss	16.2	4.9	A	6.678	154
CPR-4	5	Granitic Gneiss	19.5	5.9	D	6.838	157
CPR-4	6	Granitic Gneiss	23.7	7.2	D	5.024	116
CPR-6	1	Granitic Gneiss	13.3	4.0	D	4.803	110
CPR-6	2	Granitic Gneiss	14.4	4.4	A	9.132	210
CPR-6	4	Granitic Gneiss	18.3	5.6	D	4.309	99
CPR-6	5	Granitic Gneiss	19.4	5.9	A	1.770	41
CPR-6	6	Granitic Gneiss	20.7	6.3	D	2.120	49
CPR-6	7	Granitic Gneiss	24.3	7.4	D	7.494	172
CPR-8	1	Granitic Gneiss	3.1	0.9	D	6.676	154
CPR-8	2	Granitic Gneiss	3.8	1.1	A	6.992	161
CPR-8	3	Granitic Gneiss	5.3	1.6	D	6.651	153
CPR-8	4	Granitic Gneiss	6.3	1.9	A	6.109	140
CPR-8	5	Granitic Gneiss	8.5	2.6	D	5.901	136
CPR-8	6	Granitic Gneiss	10.3	3.1	A	9.116	210
CPR-8	7	Granitic Gneiss	12.4	3.8	D	5.126	118
CPR-9	1	Granitic Gneiss	4.6	1.4	D	4.547	105
CPR-9	2	Granitic Gneiss	4.1	1.3	A	7.395	170
CPR-9	3	Granitic Gneiss	6.2	1.9	D	5.714	131
CPR-9	4	Granitic Gneiss	6.3	1.9	A	7.054	162
CPR-9	5	Granitic Gneiss	6.5	2.0	A	6.718	155
CPR-9	6	Granitic Gneiss	10.8	3.3	D	5.160	119
CPR-9	7	Granitic Gneiss	13.8	4.2	D	6.233	143
CPR-9	8	Granitic Gneiss	13.3	4.1	A	7.532	173

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

PROJECT NO.: 03-1111-028
LOCATION: Proposed Highway 559 / CPR Overpass
DATE: January 05, 2005

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
CPR-10	1	Granitic Gneiss	0.3	0.1	D	5.322	122
CPR-10	2	Granitic Gneiss	1.4	0.4	A	8.103	186
CPR-10	3	Granitic Gneiss	2.6	0.8	D	5.986	138
CPR-10	4	Granitic Gneiss	5.3	1.6	A	8.200	189
CPR-10	5	Granitic Gneiss	7.6	2.3	D	5.561	128
CPR-10	6	Granitic Gneiss	11.3	3.5	D	5.271	121
SUMMARY[2]					**Average Axial**	7.786	179
					Average Diametral	5.802	133
					St. Dev. Axial	1.076	25
					St. Dev. Diametral	0.868	20
					Number of Axial Tests	15	
					Number of Diametral Tests	24	

[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 559 / CPR Overhead
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. Cohesionless soils overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose bedrock. Groundwater control and/or temporary shoring system may have to be incorporated.	Much lower relative costs than piled foundations since less bedrock excavation required. Additional costs may be required for installation of temporary shoring and dewatering.	If bedrock is higher than anticipated, bedrock removal is required. Variability in bedrock surface will impact mass concrete quantities and excavation depths. Dewatering system may be required if seepage volumes are greater than those anticipated.
Spread Footings perched within embankment fill		Can eliminate bedrock removal and/or mass concrete placement at west abutment.	Not practical at east abutment where bedrock excavation required to accommodate geometry of bridge. Potential for differential settlement between west abutment (due to compression of embankment fill) and east 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. Cost savings as installation of temporary shoring and dewatering likely not required.	Different footing design required at east abutment and 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\final\5 - highway 559-cp bridge\tables\table2_evaluation foundation alternatives.doc

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

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES
NW Retaining Wall
Highway 559 / CPR Overhead
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 soil excavation and may require bedrock excavation with mass concrete placement to achieve level footing. If necessary, bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break. Cohesionless soils overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose bedrock. Groundwater control and/or temporary shoring system may have to be incorporated. Rail protection may be required.	Additional costs may be required for installation of temporary shoring and dewatering. Higher relative costs for rail protection (deeper excavation required).	If bedrock is higher than anticipated, bedrock removal is required. Variability in bedrock surface will impact mass concrete quantities and excavation depths. Dewatering system may be required if seepage volumes are greater than those anticipated. Rail protection may be required.
Spread Footings on native cohesionless subsoils		Can eliminate bedrock removal and/or mass concrete placement.	Cohesionless soils overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose founding level. Groundwater control and/or temporary shoring system may have to be incorporated. Rail protection may be required. Inadequate soil cover (i.e. less than 1.8 m) for frost protection due to proposed drainage ditch adjacent to wall (50 mm to 75 mm rigid insulation may be required).	Lower relative costs than footings on mass concrete / bedrock since less excavation required. However; possible higher relative costs then spread footing on bedrock since lower allowable bearing capacity may require larger footing size. Additional costs may be required for installation of temporary shoring and dewatering. Additional costs for rail protection. Additional costs for rigid insulation.	Dewatering system may be required if seepage volumes are greater than those anticipated. Rail protection may be required. Rigid insulation may be required to provide frost protection.

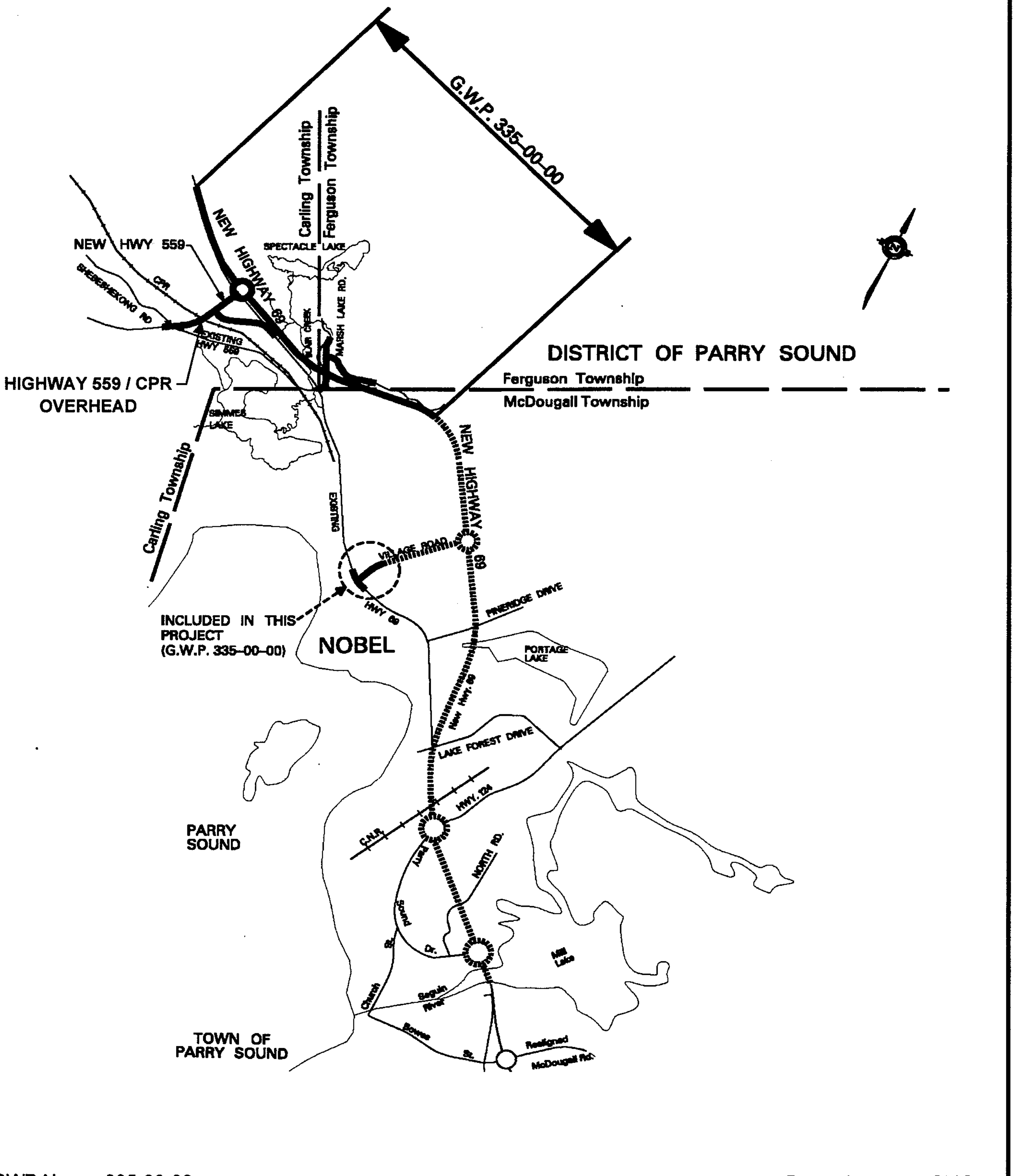
n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\draft\5 - highway 559-cp bridge\tables\table3_retaining wall foundation alternatives.doc

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

FIGURES

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

FIGURE 1



GWP No. 335-00-00
Date: December 2005
Project: 03-1111-028-5

Golder Associates

Drawn by: CMG
Checked by: JPD

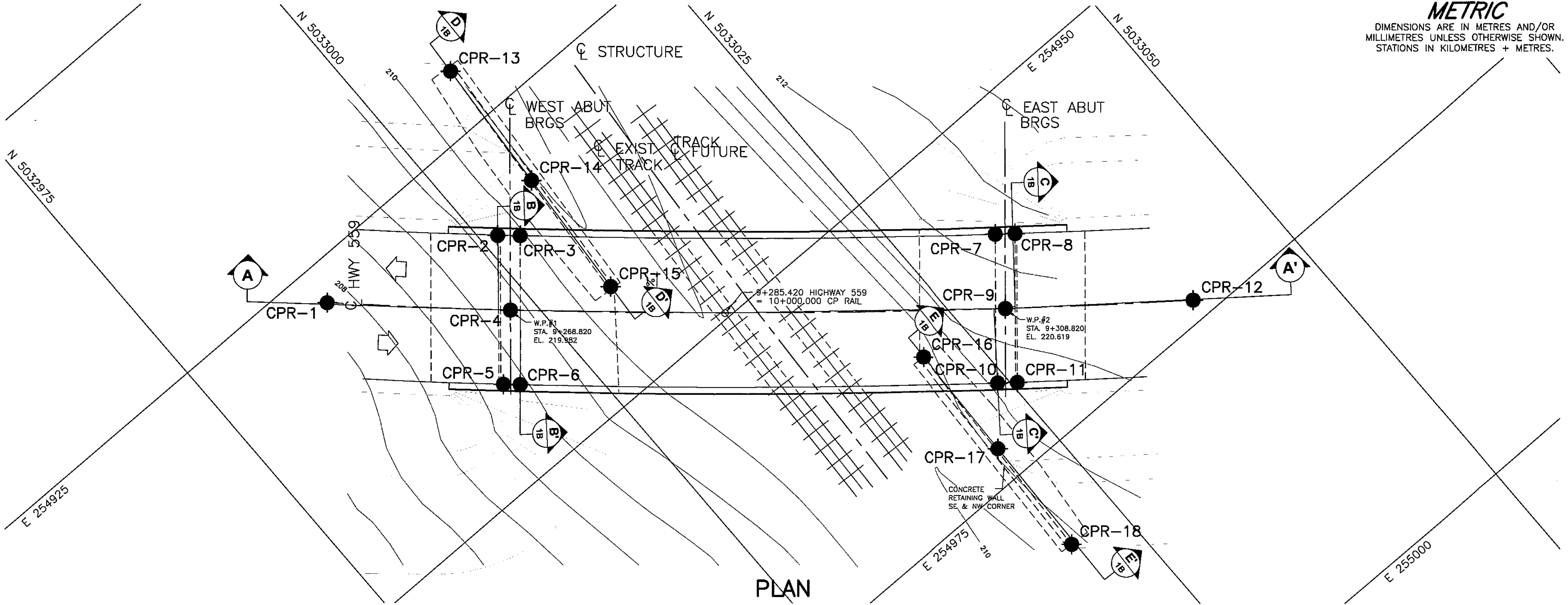
Provided in digital format by URS on January 7, 2005

N:\Active\2003\1111\03-1111-028 URS Hwy 69 Parry Sound\Reporting\Final\5 - Highway 559-CP Bridge\Figures\Figure 1.xls

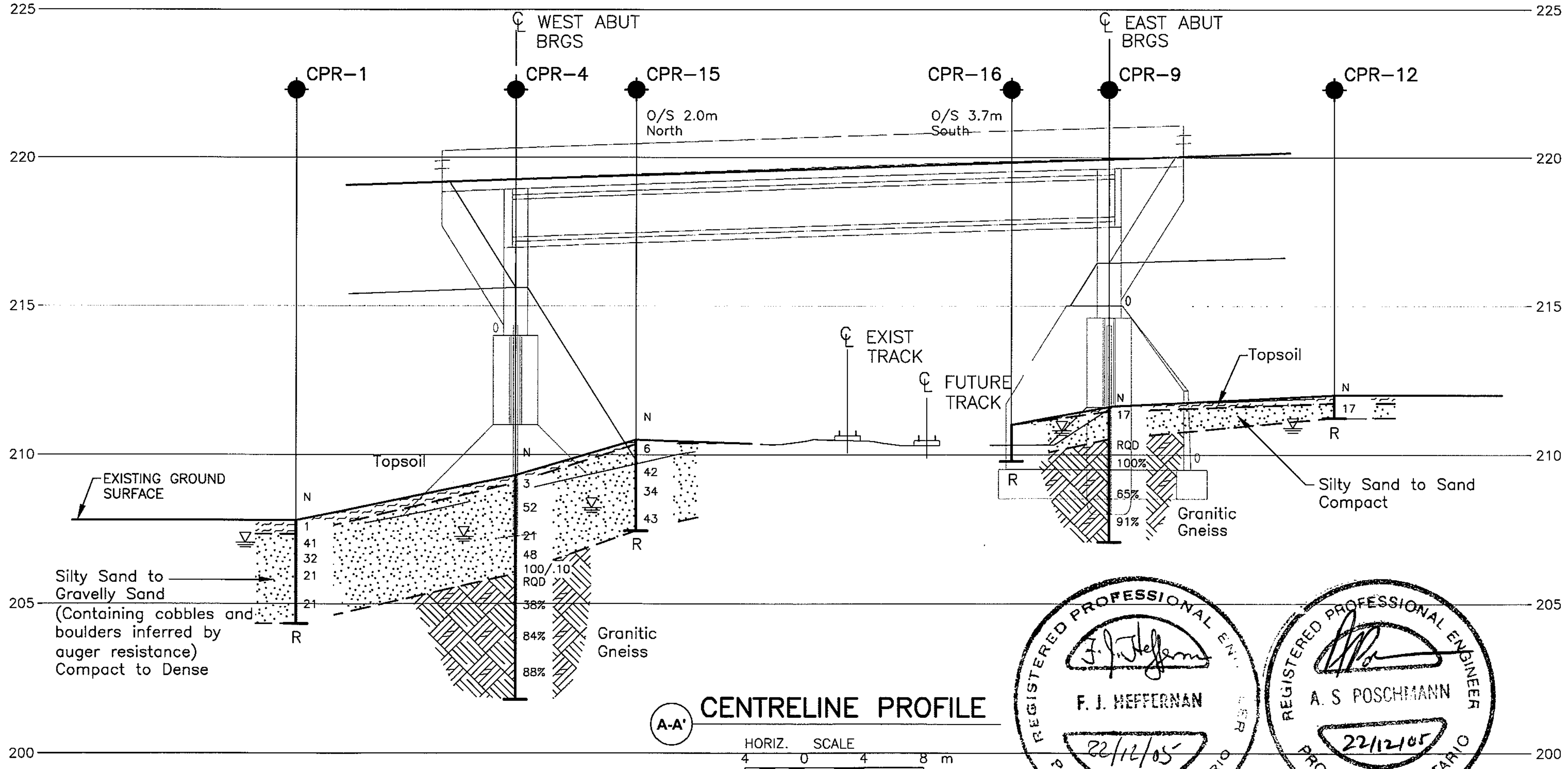
DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

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



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

F. J. HEFFERNAN
22/12/05

A. S. POSCHMANN
22/12/05

CONT No.
GWP No. 335-00-00

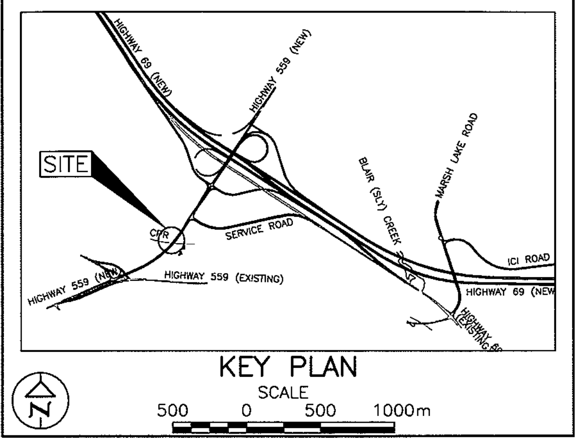
HIGHWAY 559
PROPOSED CPR OVERHEAD

BORHOLE LOCATIONS & SOIL STRATA

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
500 0 500 1000m

LEGEND

●	Borehole - Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Dec 21, 2004
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
CPR-1	207.8	5032987.3	254928.0
CPR-2	209.8	5033001.3	254932.9
CPR-3	210.0	5033002.6	254934.1
CPR-4	209.3	5032998.1	254938.1
CPR-5	208.8	5032993.8	254942.3
CPR-6	208.9	5032994.8	254943.2
CPR-7	211.9	5033031.9	254959.0
CPR-8	211.9	5033033.1	254960.0
CPR-9	211.6	5033028.6	254964.0
CPR-10	211.1	5033024.2	254968.2
CPR-11	211.3	5033025.4	254969.2
CPR-12	212.0	5033040.6	254973.4
CPR-13	210.4	5033006.9	254920.4
CPR-14	210.4	5033006.2	254931.3
CPR-15	210.5	5033005.5	254941.9
CPR-16	211.0	5033021.0	254962.7
CPR-17	210.5	5033020.8	254972.3
CPR-18	209.9	5033020.3	254982.0

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 559CPR int_ga.dwg, dated Nov. 2004, received Nov. 05, 2004.

NO.	DATE	BY	REVISION

Geocres No. 41H - 52

HWY. 559		PROJECT NO. 03-1111-028	DIST. 52
SUBM'D. CMG	CHKD. ASP	DATE: DEC., 2005	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1A

PLOT DATE: December 13, 2005
FILENAME: T:\Projects\2003\03-1111-028 (URSCI, Parry Sound)\~IB~ (CPR-FINAL)\0311110281B001A.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

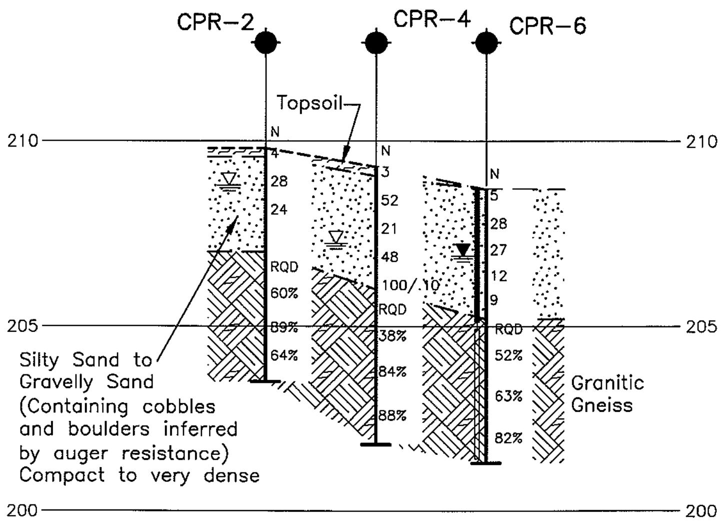
CONT No.
GWP No. 335-00-00

HIGHWAY 559
PROPOSED CPR OVERHEAD

SOIL STRATA

SHEET

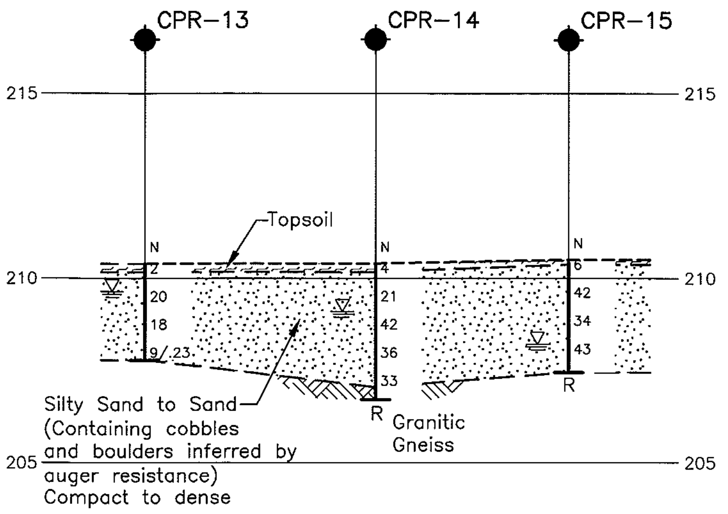
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



B-B' 1A WEST ABUTMENT

HORIZ. SCALE 4 0 4 8 m

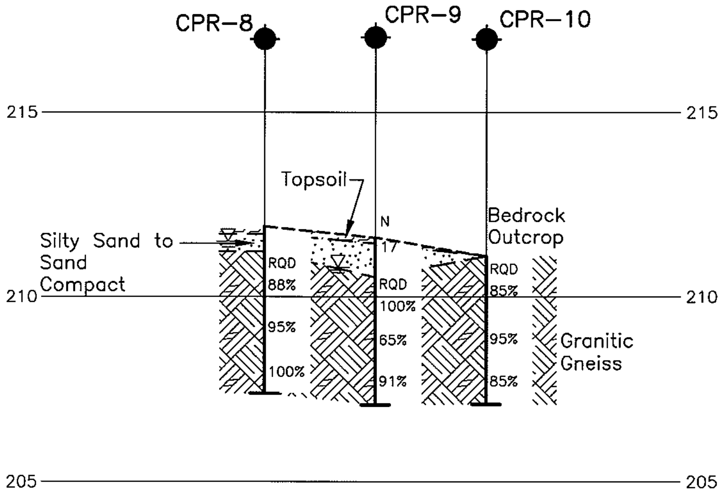
VERT. SCALE 2 0 2 4 m



D-D' 1A NW RETAINING WALL

HORIZ. SCALE 4 0 4 8 m

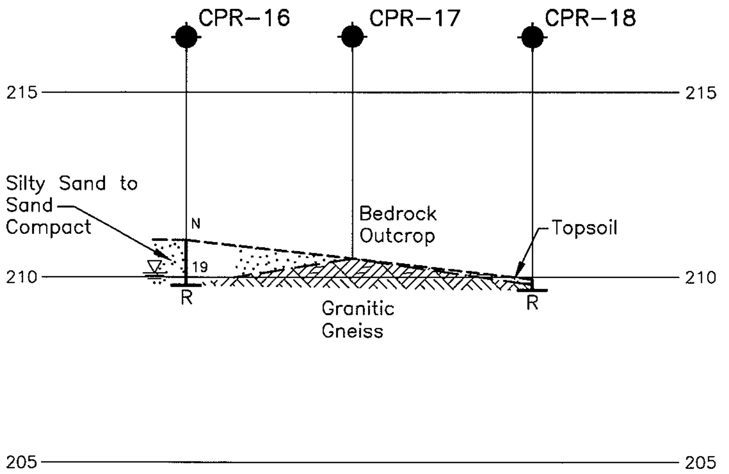
VERT. SCALE 2 0 2 4 m



C-C' 1A EAST ABUTMENT

HORIZ. SCALE 4 0 4 8 m

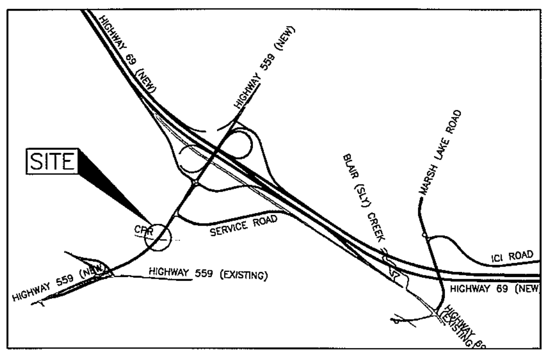
VERT. SCALE 2 0 2 4 m



E-E' 1A SE RETAINING WALL

HORIZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m



KEY PLAN

SCALE 500 0 500 1000m

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Dec 21, 2004
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
CPR-1	207.8	5032987.3	254928.0
CPR-2	209.8	5033001.3	254932.9
CPR-3	210.0	5033002.9	254934.3
CPR-4	209.3	5032998.1	254938.1
CPR-5	208.8	5032993.8	254942.3
CPR-6	208.9	5032995.3	254943.6
CPR-7	211.9	5033031.9	254959.0
CPR-8	211.9	5033033.1	254960.0
CPR-9	211.6	5033028.6	254964.0
CPR-10	211.1	5033024.2	254968.2
CPR-11	211.3	5033025.4	254969.2
CPR-12	212.0	5033040.6	254973.4
CPR-13	210.4	5033006.9	254920.4
CPR-14	210.4	5033006.2	254931.3
CPR-15	210.5	5033005.5	254941.9
CPR-16	211.0	5033021.0	254962.7
CPR-17	210.5	5033020.8	254972.3
CPR-18	209.9	5033020.3	254982.0

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For subsurface information only.

REFERENCE

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REGISTERED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO
F. J. HEFFERNAN
22/12/05

A S POSCHMANN
22/12/05

NO.	DATE	BY	REVISION

Geocres No. 41H – 52

HWY. 559		PROJECT NO. 03-1111-028	DIST. 52
SUBM'D. CMG	CHKD. ASP	DATE: DEC., 2005	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1B

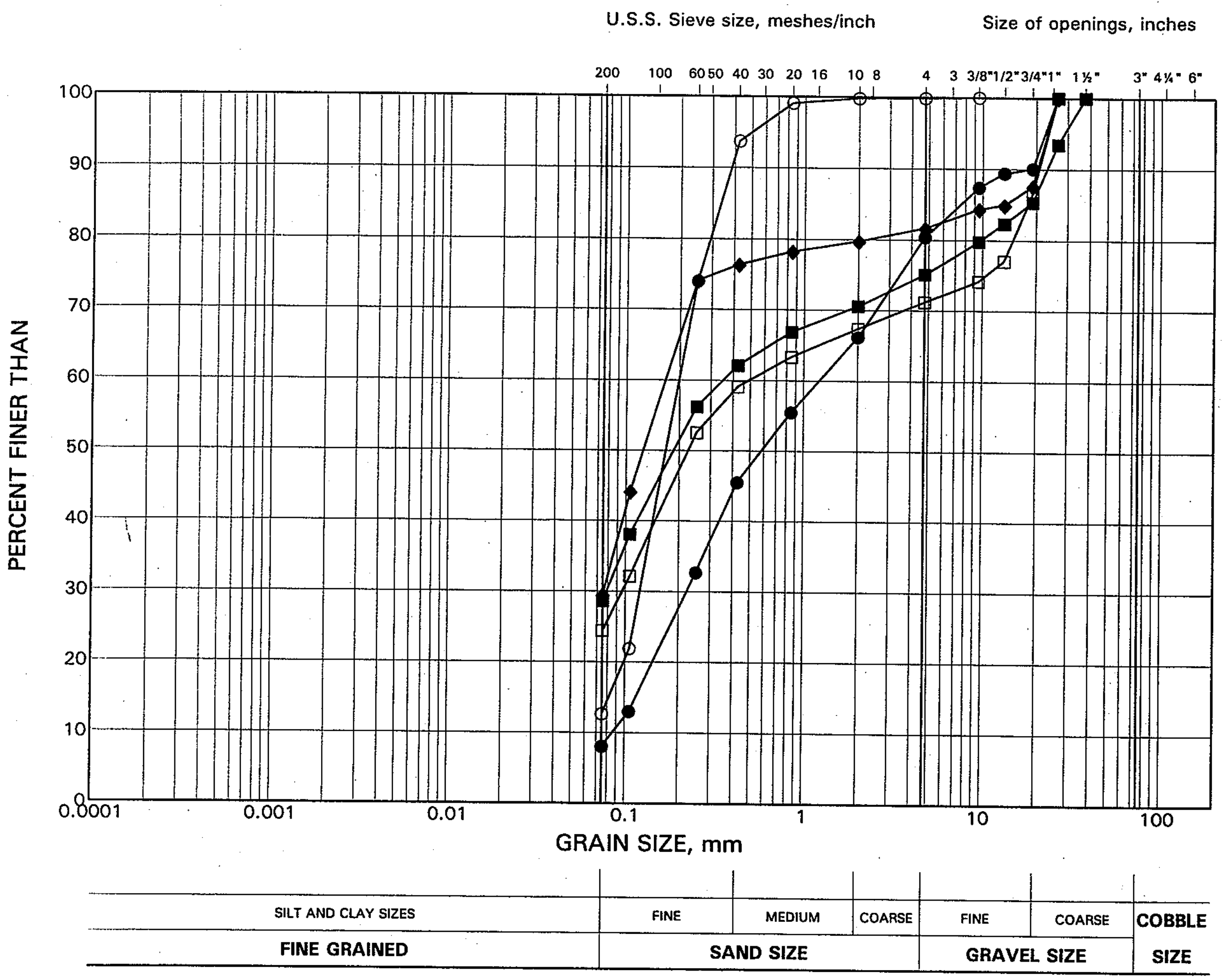
PLOT DATE: December 13, 2005
FILENAME: T:\Projects\2003\03-1111-028 (URSCI, Parry Sound)\-IB- (CPR-FINAL)\031111028IB001B.dwg

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Silty Sand to Gravelly Sand

FIGURE A-1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	CPR-1	3	206.0
■	CPR-4	4	206.7
◆	CPR-6	2	207.8
○	CPR-13	2	209.5
□	CPR-14	3	208.7

Date September 2005

Project 03-1111-028

Golder Associates

Prepared by LG

Checked by JPD

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/or retaining wall footings.

Construction

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

Basis of Payment

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

DOWELS Into Rock – Item No.

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 559/CPR Overhead	West Abutment	2
Highway 559/CPR Overhead	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.

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

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:

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.