



January 20, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**CANADIAN PACIFIC RAILWAY OVERHEAD STRUCTURE NBL,
SITE NO.44-460/1
HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529
NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5404-05-00; WP 5143-08-01**

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GEOCRES NO.: 41H-138

Report Number: 09-1111-6014-3524

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REPORT





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PART A

FOUNDATION INVESTIGATION REPORT

CANADIAN PACIFIC RAILWAY OVERHEAD STRUCTURE - NBL,
SITE NO. 44-460/1

HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529
NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5404-05-00; WP 5143-08-01



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 provide detail foundation engineering services for the proposed Highway 69 Northbound Lane (NBL) structure over the Canadian Pacific Railway (CPR), which is within the Contract 3 limits of the new Highway 69 alignment. The proposed work in Contract 3 is part of the four-laning of Highway 69 from 1.7 km north of Highway 529 northerly to 3.9 km north of Highway 522, for a total distance of 19.7 km, which includes: high fill embankments and embankments over swamps; the Canadian National Railway (CNR) re-alignment; the Bekanon Road and Highway 522 interchanges and structures; the Still River, Straight Lake and Key River structures; the Canadian Pacific Railway and Canadian National Railway (CNR) Overhead structures; as well as culvert crossings. The CPR Overhead NBL structure is to be located approximately 1.5 km east of the existing Highway 69 and about 200 m south of Straight Lake. The general location of this proposed bridge along the new Highway 69 four-laning alignment is shown on the Site Location Plan on Drawing 1.

The Terms of Reference (TOR) for the foundation investigation are outlined in MTO's Request for Proposal, dated December 2008. Golder's proposal (Scope of Work) for foundation engineering services associated with the Contract 3 CPR Overhead NBL structure is contained in Section 6.8 of URS's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated April 19, 2010.

This report addresses the investigation carried out for the CPR Overhead NBL structure and the associated approach embankments only (Site No. 44-460/1). A separate report addresses the foundation investigation for the CPR Overhead southbound lane structure.

The purpose of this investigation is to establish the subsurface conditions at the proposed bridge structure location, including the associated approach embankments, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The foundation units/limits for this investigation were located in the field by Callon Dietz Inc. (Callon Dietz), a professional surveying company retained by URS. The investigation area is shown in plan on Drawing 2.

2.0 SITE DESCRIPTION

The proposed Highway 69 alignment is oriented generally in a south-north direction spanning the Township of Wallbridge to the south, the Township of Henvey, and the Township of Mowat to the north. The Contract 3 section of the new four-lane Highway 69 alignment is also oriented generally in a south-north direction within the overall project limits, for a total distance of 5.5 km in the Township of Henvey. The proposed CPR Overhead NBL structure is located within the Contract 3 highway alignment and is located approximately 100 m south of the northern limit of Contract 3, corresponding to approximately 1.5 km east of the existing Highway 69 alignment and about 8.5 km northeast of the junction between existing Highway 69 and Highway 526.

In general, the topography of this section of the overall project limits consists of rolling terrain, including sparsely to densely populated tree covered areas and numerous bedrock outcrops separated by valleys and swamps containing areas of standing water and various types of vegetation and organic soils. The proposed overhead structure and associated approach embankments are to be situated relatively close to an existing rock cut and/or rock outcrop in a sparsely to densely treed area adjacent to the existing CPR track alignment. The existing



ground surface within the limits of the proposed structure and approach embankments varies between about Elevations 200 m and 191 m, referenced to Geodetic datum, and is sloping downward from south to north.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed CPR Overhead NBL structure was carried out between February 26 and March 7, 2013 during which time a total of eight (8) boreholes, and four (4) hand shovel excavations were advanced at the locations of the structure foundation footprints and approach embankments. A summary of the respective boreholes and hand shovel excavations advanced at each foundation element and approach embankment is presented below.

Foundation Element/Approach Embankment	Investigation Type	
	Borehole	Hand Shovel Excavation
South Approach Embankment	--	B304-01
South Abutment	B304-03 B304-04 B304-05	B304-02 B304-06
North Abutment	B304-08 B304-09 B304-10 B304-11 B304-12	--
North Approach Embankment	--	B304-07

The Record of Borehole/Drillhole sheets and the results of the laboratory testing are presented in Appendix A and Appendix B, respectively. The locations of the boreholes and hand shovel excavations are shown in plan on Drawing 2.

The field borehole investigation was carried out using a portable drill rig supplied and operated by Landcore of Chelmsford, Ontario. Hand shovel excavation methods were used as appropriate depending on the terrain and to confirm refusal conditions at shallow borehole locations where possible. The boreholes were generally advanced through the overburden using NW casing. Where possible, soil samples were obtained at ground surface and at intervals of depth of about 0.75 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by a manual hammer on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes at the locations of the foundation elements were typically advanced to hand shovel, casing and/or sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring in selected boreholes. Refusal condition at the boreholes at the south and north approach embankments was confirmed by hand shovel excavation. The boreholes were advanced to depths of up to about 8.7 m below existing ground surface, including coring of bedrock for core lengths between about 3.1 m and 6.0 m in Boreholes B304-03 to B304-05, B304-08, B304-10 and B304-12.



The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. Within the limits of the north abutment, a piezometer was installed in Borehole B304-10 to monitor the ground water level at this location. The piezometer consists of 35 mm diameter PVC pipe, with a slotted screen sealed at a selected depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen (and sand pack) were backfilled to the surface with bentonite pellets. Piezometer installation details and a water level reading are described on the Record of Borehole sheet presented in Appendix A. All open boreholes were backfilled with bentonite upon completion and the piezometer was abandoned on March 9, 2013 in accordance with Ontario Regulation 903, Wells (as amended).

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

The perimeter limits of each foundation unit were located in the field by Callon Dietz prior to drilling. The staked borehole locations and ground/ice surface elevations were surveyed by Callon Dietz. The locations given on the Record of Borehole/Drillhole sheets and shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground/ice surface elevations are referenced to Geodetic datum. The borehole locations and ground/ice surface elevations are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground/Ice Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
B304-01	5082691.6	223179.6	198.9	0.5
B304-02	5082705.6	223166.5	197.6	0.3
B304-03	5082710.4	223164.4	197.7	6.1
B304-04	5082711.5	223171.0	197.5	3.7
B304-05	5082712.4	223177.9	200.0	5.8
B304-06	5082717.5	223174.5	196.2	0.1
B304-07	5082765.0	223148.7	194.8	0.4
B304-08	5082745.4	223149.9	191.1	8.7
B304-09	5082740.7	223151.8	191.1	2.6
B304-10	5082746.6	223156.4	191.2	7.3
B304-11	5082752.8	223161.4	191.2	4.1
B304-12	5082748.1	223163.3	191.1	7.0



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

The bedrock in the area consists typically of crystalline gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in *Geology of Ontario*, OGS Special Volume 4². Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented on the attached Record of Borehole and Drillhole sheets and on the laboratory test figures provided in Appendix A and Appendix B, respectively. The results of the in situ field tests (i.e. SPT 'N'-values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the area of the CPR Overhead NBL structure consist of a layer of organic silt or peat, underlain by thin, discontinuous non-cohesive deposits of silt to sandy silt to silt and sandy gravel, underlain by bedrock. The overburden thickness at the boreholes advanced for the proposed bridge structure ranges from less than about 0.1 m at the northeast area of the south abutment and (exposed bedrock in places) to about 3.9 m at the northeast corner of the north abutment.

A detailed description of the subsurface conditions encountered in the boreholes at the abutments and approach embankments is provided in the following sections.

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

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



4.2.1 Peat/Organic Silt

An approximately 0.1 m to 1.0 m thick layer of peat and/or organic silt up was encountered at the ground surface in Boreholes B304-01 to B304-04, B304-06 and B304-07, and immediately below ice/water in Boreholes B304-08 to B304-12.

The SPT 'N'-values measured within the peat deposit, across the interface of peat and organic silt layer and within the organic silt layer range from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration, suggesting a very soft consistency.

The natural water content measured on two (2) samples of the peat deposit is about 734 per cent and 1058 per cent and on three (3) samples of the organic silt deposit is about 72 per cent to 109 per cent.

The organic content measured on two (2) samples of the organic silt deposit is about 5 per cent and 6 per cent.

An Atterberg limits test was carried out on a sample of organic silt deposit and measured a liquid limit of about 63 per cent, a plastic limit of about 42 per cent and plasticity index of about 21 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B1 in Appendix B, and indicates that the material is classified as organic silt of high plasticity.

4.2.2 Silt to Silt and Sand to Sandy Gravel

A deposit of non-cohesive soils comprised of brown to grey silt, trace to some sand to sandy silt to silt and sand to sandy gravel was encountered below the peat/organic silt layers in Boreholes B304-01, B304-07 to B304-12. The deposit in places contains trace gravel, trace clay and trace organics. The top of this deposit was encountered between about Elevations 198.8 m and 189.8 m and the thickness of the deposit ranges between about 0.3 m and 3.3 m.

The SPT 'N'-values measured within the non-cohesive deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on ten (10) samples of the non-cohesive deposit ranges between about 1 per cent and 26 per cent.

The results of grain size distribution tests completed on seven (7) samples of the silt to sandy silt to silt and sand portion of the deposit are shown on Figure B2 in Appendix B. The result of a grain size distribution test completed on a sample of sandy gravel layer is shown on Figure B3 in Appendix B.

Atterberg limits tests carried out on two (2) samples of the silt to sandy silt portion of deposit and indicate that the fine material is non-plastic.

4.2.3 Bedrock/Refusal

Bedrock was encountered and core samples were recovered from Boreholes B304-03 to B304-05, B304-08, B304-10 and B304-12. Bedrock outcrops are present in the immediate area of the proposed structure. The bedrock surface was inferred from hand shovel excavation or split-spoon/casing refusal in Boreholes B304-01, B304-2, B304-06, B304-07 and B304-09. The depths to bedrock below ground surface and the corresponding bedrock surface elevation at the investigation locations are summarized below.



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Foundation Element / Approach Embankment	Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
South Approach Embankment	B304-01	0.5	198.4	Hand Shovel Excavation
South Abutment	B304-02	0.3	197.3	Hand Shovel Excavation
	B304-03	0.1	197.6	6.0 m of Bedrock Cored
	B304-04	0.3	197.2	3.4 m of Bedrock Cored
	B304-05	<0.1	200.0	5.8 m of Bedrock Cored
	B304-06	0.1	196.1	Hand Shovel Excavation
	B304-08	3.3 ¹	187.5	5.1 m of Bedrock Cored
North Abutment	B304-09	2.4 ¹	188.5	Split-Spoon and Casing Refusal
	B304-10	3.8 ¹	187.2	3.3 m of Bedrock Cored
	B304-11	3.9 ¹	187.1	Split-Spoon and Casing Refusal
	B304-12	3.6 ¹	187.2	3.1 m of Bedrock Cored
North Approach Embankment	B304-07	0.4	194.4	Hand Shovel Excavation

Note: 1. Depth to Bedrock measured from ground surface below ponded water.

In general, the bedrock surface in the area of the CPR Overhead NBL structure is blocky, with horizontal and vertical fragments and slopes downward from south to north, as shown on the site photographs on Figures B4 and B5. The difference in the bedrock surface elevation at the investigated locations is up to 3.9 m at the south abutment, up to 1.4 m at the north abutment and up to about 13 m between the abutments at the borehole locations.

Based on a review of the bedrock core samples, the bedrock consists of granitic gneiss. In general, the bedrock samples are described as slightly weathered to fresh, thinly laminated to thinly bedded, fine to coarse crystalline, faintly porous, medium strong to very strong, pink, light to dark grey with pink interbeds, as presented on the Record of Drillhole sheets in Appendix A, and shown on the photographs of the recovered core samples on Figures B6 to B11 in Appendix B. The degree of weathering of the bedrock samples (i.e. lightly weathered to fresh – W1 to W2), and the strength classification of the intact rock mass based on field identification (i.e. strong to very strong – R4 to R5) are described in accordance with the International Society for Rock Mechanics (ISRM³) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 60 per cent to 100 per cent, indicating a rock mass of fair to excellent quality as per Table 3.10 of CFEM (2006⁴). However, portions of core recovered from Boreholes B304-04 and B304-08 contain fractured rock with RQD values of about 36 per cent and 41 per cent, indicating a rock mass of poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples recovered are between 90 per cent and 100 per cent and between 44 per cent and 100 per cent, respectively.

³ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

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



Point load strength index tests (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification) were carried out on selected samples of the bedrock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets and are presented in Table B1 in Appendix B. The axial tests carried out on twenty-three (23) samples of the granitic gneiss bedrock core measured Is_{50} values ranging from about 3.1 MPa to 12.3 MPa, but generally greater than 6.1 MPa, and the diametral tests carried out on twenty-four (24) samples of the granitic gneiss bedrock core measured Is_{50} values ranging from about 3.2 MPa to 10.8 MPa, but generally greater than 5.0 MPa.

Two (2) Unconfined Compression (UC) tests (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) were carried out on selected core samples of the granitic gneiss bedrock obtained in Boreholes B304-03 and B304-12 and measured uniaxial compressive strengths of about 125 MPa and 106 MPa, respectively, as summarized in Table B2-1 and detailed in Tables B2-2 and B2-3 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength index based on a relationship between Is_{50} and UCS, which is given by a correlation factor (K) which varies depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 12.

Based on the laboratory UC and the axial point load index test results, and in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.4 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. The water level observed in boreholes upon completion of drilling varied between about Elevations 198.5 m and 191.0 m, measured between 0.3 m above ground surface (i.e. equivalent ponded column of water) and depths up to about 4.8 m below ground surface. Boreholes B304-06 and B304-07 were dry upon completion of hand excavation.

A standpipe piezometer was installed in Borehole B304-10 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A. The groundwater level measured in the piezometer installation is summarized below.

Foundation Element	Borehole	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Abutment	B304-10	190.9	191.1	March 9, 2013

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.



5.0 CLOSURE

Mr. Matt J. Riopelle, field technician with Golder, directed the drilling program. This report was prepared by Mr. Al Varshoi, M.E.Sc., and was reviewed by Mr. J. Paul Dittrich, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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[http://capws/sites/0911116014highway69fourlaning/contract 3/reporting/final/cpr overhead structure nbl/09-1111-6014-3524 rpt 14jan20 cpr overhead structure nbl.docx](http://capws/sites/0911116014highway69fourlaning/contract%203/reporting/final/cpr%20overhead%20structure%20nbl/09-1111-6014-3524%20rpt%2014jan20%20cpr%20overhead%20structure%20nbl.docx)



PART B

FOUNDATION DESIGN REPORT

CANADIAN PACIFIC RAILWAY OVERHEAD STRUCTURE - NBL ,

SITE NO. 44-460/1

HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529

NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5404-05-00; WP 5143-08-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed CPR Overhead NBL structure (Site No. 44-460/1). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the CPR Overhead NBL structure within Contract 3 along the proposed section of four-laning of Highway 69 in the Township of Henvey. Based on the General Agreement (GA) Drawing dated July 10, 2013 provided by URS, the CPR Overhead NBL structure will consist of a single span with a span length of 33 m and abutments located south and north of the existing tracks. The grade of the proposed Overhead NBL bridge deck will vary between about Elevations 201.5 m and 201.3 m and the proposed south and north approach embankments will be up to about 5 m and 10.5 m high, respectively, above existing ground surface.

6.2 Foundation Options

Given the proposed footing founding level at the south abutment of the CPR Overhead NBL structure at about Elevation 195.2 m, a shallow foundation comprised of a strip footing founded directly on bedrock is considered the preferred foundation alternative to support the south abutment. This option will require excavation of medium strong to very strong bedrock to depths of about 4.0 m below the existing ground surface. Considering the proposed founding level relative to the bedrock surface at the south abutment, steel H-pile or drilled steel casing foundations would not be practical due to the significant amount of excavation and trenching into the bedrock required to achieve the minimum pile lengths for a deep foundation support system.

For the north abutment, consideration could be given to a shallow foundation comprised of a strip footing founded on a compacted granular pad perched within the approach embankment. Alternatively, a deep foundation system consisting of either driven steel H-piles to bedrock, steel H-piles socketed into bedrock using “Down-the-Hole” (DTH) hammer drilling or drilled steel casings socketed into bedrock could be considered for design. Given the preference for an integral abutment design supporting the proposed overhead structure, steel H-piles driven to bedrock is considered the preferred foundation alternative at this location.

The following sections provide recommendations for shallow spread footings, steel H-pile and drilled steel casing foundations, where applicable, to support the bridge abutments.

The advantages, disadvantages, relative costs and risks/consequences for the foundation options are summarized in Tables 1 and 2 for the south and north abutments, respectively.



6.3 Spread Footings

A shallow foundation comprised of spread/strip footing founded directly on bedrock is considered feasible for the support of the south abutment. A shallow foundation comprised of a spread/strip footing on a compacted Granular 'A' pad may be considered for support of the north abutment.

6.3.1 Geotechnical Axial Resistance and Reaction

Based on the GA Drawing provided by URS, the south abutment footing is currently proposed to be founded at about Elevation 195.2 m. Given this proposed founding level, up to about 4 m of bedrock excavation will be required to construct the spread footing on properly prepared granitic gneiss bedrock.

For the north abutment, consideration could be given to a spread footing founded on a granular pad perched within the approach embankment fill. The pad may be founded at about Elevation 190 m after removing all the existing organic and/or any loosened/softened soils. The pad should extend 1 m beyond the footing footprint, should be not less than 1 m thick and constructed of OPSS.Prov 1010 (Aggregates) Granular 'A' or Granular 'B' Type II (see also Section 6.12.2).

The following factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement may be used for design of a spread/strip footing founded on properly prepared bedrock or on a compacted Granular 'A' pad.

Foundation Unit	Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
South Abutment	Spread/Strip footing on properly prepared granitic gneiss bedrock	10,000	N/A ¹
North Abutment	Spread/Strip footing ² on a minimum 1 m thick Granular 'A' pad	900	350

- Notes:
1. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS for spread footings on bedrock and as a result the SLS condition does not apply.
 2. The geotechnical reaction at SLS is estimated for a 3 m wide spread/strip footing.

The geotechnical resistance/reaction provided above is given for loads applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC, 2006) and its Commentary.

Following excavation of the overburden and removal of the bedrock and prior to placing any concrete at the south abutment, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the footprint of the footing to ensure a proper bond of the concrete to the bedrock. Recommendations for such excavation and blasting are provided in Section 6.13.



6.3.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footing and the bedrock or concrete footing and the granular pad, should be calculated in accordance with Section 6.7.5 of CHBDC (2006). The following presents the coefficient of friction, $\tan \delta$, for the interface between the concrete footing and bedrock and $\tan \phi$ for the concrete zone between the concrete footing and the granular pad.

Interface Material(s)	Coefficient of Friction ($\tan \delta$ or $\tan \phi$)
Concrete Footing on Bedrock	0.70
Concrete Footing on Compacted Granular 'A' Pad	0.58

The values presented above represent unfactored values.

If necessary, the sliding resistance between the mass concrete/concrete footings and the bedrock can be supplemented by dowelling into the bedrock at the south abutment. 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 or stronger than concrete, the design of the dowels into the rock may be handled in the same way as the dowels 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 dowels and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (an example NSSP is included in Appendix C).

6.3.3 Frost Protection

The following should be noted for the design of spread footings founded on a Granular 'A' pad:

- The required thickness of conventional soil cover for frost protection of the footing is 1.8 m, as per OPSD 3090.010 (Frost Penetration Depths for Southern Ontario) as measured perpendicular from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope).
- If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration, however, the edge of the footing must be set-back at least 1 m (measured perpendicular) from the crest of the adjacent slope.

For the spread footing founded directly on the properly prepared granitic gneiss bedrock at this site, a minimum soil cover for frost protection is not required.

6.3.4 Footing Set-Back from Rock Face

The spread footing at the south abutment will be situated on the bedrock outcrop above the adjacent CPR tracks. The footing must be maintained an adequate distance away from the edge of the new rock cut face, and the rock face must be adequately cleaned and/or protected such that the integrity of the rock face and founding



rock surface is maintained. In this regard, the abutment footing should be located away from the new rock face a distance as defined by a line projected upwards at 0.5 horizontal to 1 vertical (0.5H:1V) from the toe of the rock cut face and not closer than 2 m from the crest (edge) of the nearest cut face, whichever is greater. If the bridge layout does not allow for this minimum setback zone, special measures (such as the installation of vertical rock dowels behind the crest (edge) of the rock face prior to rock excavation) will be required to control and pre-support the rock face during blasting and following construction.

6.4 Driven Steel H-Pile Foundations

Given that the bedrock surface is higher at the south abutment than the proposed foundation founding level, driven steel H-pile foundations are not suitable.

Taking into consideration the elevation of the underside of the proposed pile cap at the north abutment (i.e. at about Elevation 195.2 m) and the depth to the bedrock surface (i.e. between Elevations 187.1 m to 188.5 m at the investigated locations), a deep foundation system consisting of steel HP 310x110 piles driven to bedrock is considered appropriate for support of the north abutment.

Based on the GA Drawing provided by URS, Corrugated Steel Pipes (CSPs) will be installed around the upper portion of the piles as part of the integral abutment design. The CSPs should be backfilled with loose/uniform sand meeting the gradation specified in the NSSP presented in Appendix C, which should be included in Contract Documents. It is also noted that a compacted granular pad is proposed between the ground surface and the underside of the CSPs through which the piles will be driven. Recommendations for construction of the granular pad are given in Section 6.12.2.

For design, the pile tip elevations presented below may be assumed for piles terminating on the bedrock surface, based on the depth to bedrock as encountered in the boreholes advanced at the north abutment. There should be a provision in the Contract Documents for dealing with varying pile lengths considering the variability in the elevation of the bedrock surface in this area.

Foundation Unit	Proposed Underside of the Pile Cap Elevation (m)	Approximate Pile Tip Elevation (m)	Approximate Pile Length (m)
North Abutment	195.2	188.5 to 187.1	6.7 to 8.1

6.4.1 Geotechnical Axial Resistance and Reaction

For steel HP 310x110 piles driven to the bedrock at the north abutment, a factored geotechnical axial resistance at ULS of 2,000 kN per pile may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical reaction at SLS for 25 mm of settlement (for the length of piles required at this location) will be greater than the factored geotechnical axial resistance at ULS, since the granitic gneiss bedrock is considered to be unyielding; ULS conditions will govern for this foundation type.



6.4.2 Downdrag Load (Negative Skin Friction)

Considering the relatively thin and granular nature of the overburden, the resulting downdrag loads from the north approach embankment fill are expected to be negligible and will not cause any appreciable settlement.

6.4.3 Set Criteria

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known.

The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be established to also avoid overdriving and possibly damaging the piles.

Based on pile installation projects in the Northern Ontario, consideration should be given to the following preliminary set criteria and procedures:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs), using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced by about 75 per cent and the pile should then be re-driven by increasing the hammer energy in 25 per cent stages up to the maximum rated energy of about 50 kilojoules.
- A final set of no less than 10 blows per 12 mm of penetration should then be obtained at the maximum hammer energy. Provisions should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations).

6.4.4 Pile Driving Note

The pile driving note to be added to the drawings for this project is Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008), as follows:

- “Piles to be driven to bedrock”.

6.4.5 Frost Protection

The pile cap at the north abutment should be provided with a minimum of 1.8 m of conventional soil cover for frost protection, as per OPSD 3090.101 (Foundation, Frost Penetration Depths for Southern Ontario).



6.5 Socketed Steel H-Pile Foundations

Given that the bedrock surface at the south abutment is higher than the proposed foundation founding level, a significant amount of excavation and trenching/socketing into the bedrock would be required to achieve the minimum lengths for a deep foundation support system at this location. As such, steel H-pile foundations socketed into bedrock at the south abutment are not practical.

If necessary to increase pile lengths or improve lateral fixity at the base of the piles at the north abutment, consideration could be given to socketing the HP 310x110 piles into bedrock. In order to socket the steel H-piles into the bedrock, it will be necessary to core approximately 600 mm diameter holes into the medium strong to very strong granitic gneiss bedrock using DTH hammer drilling. The socket should extend at least 1 m below top of the bedrock. The portion of the pile embedded in the bedrock will need to be backfilled with concrete using tremie methods to achieve fixity at the base of H-piles.

In addition, the CSPs surrounding the upper portion of the piles immediately below the pile cap, which are installed as part of the integral abutment design, should be backfilled with loose sand meeting the gradation specified in the NSSP presented in Appendix C, which should be included in the Contract Documents.

For design of a socketed pile foundation at the north abutment, the pile tip elevations presented below may be assumed. There should be a provision in the Contract for dealing with varying pile lengths considering the variability in the elevation of the bedrock surface in this area.

Foundation Unit	Proposed Underside of the Pile Cap Elevation (m)	Approximate Pile Tip Elevation (m)	Approximate Pile Length (m)
North Abutment	195.2	187.5 to 186.1	7.7 to 9.1

6.5.1 Geotechnical Axial Resistance/Reactions

For steel HP 310x110 piles socketed 1 m into bedrock at the north abutment, a factored geotechnical axial resistance at ULS of 2,000 kN may be assumed for design. Since the granitic gneiss bedrock is considered to be unyielding, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS and therefore, ULS conditions will govern for this foundation type.

6.6 Drilled Steel Casing Foundations

Given that the bedrock surface is higher than the proposed founding level at the south abutment, drilled steel casing foundations at the south abutment are not suitable.

For the north abutment, consideration could be given to the use of drilled steel casings socketed into bedrock. Drilled steel casings up to 600 mm diameter advanced with DTH hammer drilling techniques should be readily able to advance sockets into the medium strong to very strong bedrock.



6.6.1 Geotechnical Axial Resistance/Reactions

For 600 mm diameter drilled steel casings socketed 1.2 m into bedrock at the north abutment, a factored geotechnical axial resistance at ULS of 2,750 kN may be assumed for design. Since the granitic gneiss bedrock is considered to be unyielding, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS and therefore, ULS conditions will govern for this foundation type.

6.7 Resistance to Lateral Loads

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

Lateral loading could be resisted fully or partially by the use of battered piles.

The resistance to lateral loading in front of a single pile may be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (MPa/m). However, the response of a pile/drilled steel casing to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile/drilled steel casing deflections are less than 1 per cent of the pile diameter, where the loading is static (not cyclic) and where the pile material is linear (CFEM, 2006). If one or more of these conditions are not satisfied, then it is recommended that the lateral pile analysis be carried out using p-y curves.

The following equations (CFEM, 1992 as referenced in the *CHBDC Commentary*, 2006) may be used to calculate values of k_h .

for non-cohesive soils:

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

where:

n_h	=	constant of subgrade reaction (MPa/m)
z	=	depth (m)
B	=	pile diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native subsoils to be utilized for the structural analysis of the piles at this site are presented below.

Foundation Unit	Soil Unit	Elevation (m)	n_h (MPa/m)
North Abutment	Compacted Granular 'B' Pad (Fill)	192.2 to 190.5	12
	Very Loose Silt to Silt and Sand	190.5 to 187.1	1.3
	Bedrock*	Below 187.1	11,500

* Note: This n_h value and associated soil unit is not applicable to the design of the preferred foundation alternative.



Where the integral design includes the installation of CSP liners (with the annular space between the pile and liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles installed inside the CSP will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.

For piles socketed into bedrock, the lateral resistance of the piles will be developed primarily from the fixity (in concrete) at the base of drilled sockets. In this case, the structural resistance of the pile or concrete will govern the lateral resistance.

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

Pile Spacing in Direction of Loading (d = pile diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

In order to estimate the lateral capacity of a single pile at ULS and SLS, a detailed analysis of pile response to lateral loads should be carried out when the detailed pile design (such as loading, pile head fixity, cross-section properties) becomes available.

6.8 Seismic Site Consideration

6.8.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC (2006) may be taken as 1.0, consistent with Soil Profile Type I.

6.8.2 Seismic Analysis Coefficient

According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the CHBDC and its Commentary), the site specific peak horizontal ground acceleration for Sudbury and Parry Sound area is 0.051 (for a probability of exceedance of 10 per cent in 50 years). For the thicknesses and type of overburden soils at the site, an amplification factor of 1.0 of the ground motion is recommended for design. As such, the ground surface acceleration would be about 0.05.

Given that the proposed CPR Overhead NBL structure is a single-span bridge, and in accordance with Sections 4.4.5.2 of the CHBDC, seismic analysis is not required for this structure.



6.9 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stem wall and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.8.2 a seismic (earthquake) analysis is not required for this structure.

The following recommendations are made concerning the design of abutment walls. It should be noted that these design recommendations and parameters are applicable to 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 OPSS.Prov1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Compaction of the backfill (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill) and OPSD 3121.150 (Walls, Retaining, Backfill).
- 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 specification as outlined in the Northeastern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.8 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the commentary to the CHBDC).

The lateral earth pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27
Rock Fill	19 kN/m ³	0.35	0.22

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 wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active



pressure to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC (2006).

6.10 Approach Embankment Design

Based on the GA Drawing provided by URS, the proposed road grade for the new Highway 69 NBL approaches to the CPR Overhead structure will be at about Elevation 201.5 m, requiring placement of up to about 5.0 m and 11.5 m of fill (including the sub-excavation and replacement of organic materials) within the limits of the south approach embankment and north approach embankment, respectively.

It is understood that rock fill is the preferred embankment fill material for this project. In this regard, the stability and settlement analyses have been carried out on the basis that the highway embankment will be constructed of rock fill. In accordance with MTO Northern Region Pavement Practices and Guidelines (1997) as amended by MTO Memorandum “Use of Mid-Slope Berms for Rockfill Embankments” (2005), 2 m wide berms should be incorporated into the rock fill embankment side slope profile for uninterrupted slopes greater than 10 m high, and as sections of the north approach embankment are expected to be in excess of 10 m high, 2 m wide mid-slope berms will be required along the higher sections of the embankment.

The following sections address stability and settlement analysis for the new approach embankments.

6.10.1 Stability

Given that the south approach embankment will be founded directly on bedrock and that the north approach embankment will be founded on a deposit of silt to silt and sand (up to about 3.3 m thick) underlain by granitic gneiss bedrock, global instability of the embankments is not expected, so long as embankment side slopes are constructed no steeper than 2H:1V for granular fill and no steeper than 1.25H:1V for rockfill, and a 2 m wide mid-height berm is incorporated into the north approach embankment.

6.10.2 Settlement

6.10.2.1 Methodology

The proposed south approach embankment will be founded directly on bedrock and therefore, the source of settlement is considered to be self-weight compression of the embankment fill material only. The proposed north approach embankment will be founded on a deposit of silt to silt and sand (up to about 3.3 m thick) underlain by bedrock and therefore, the sources of settlement are considered to include immediate settlement of the native granular soils and self-weight compression of the embankment fill material. The thickness of the compressible soils varies along the north approach embankment, and as such, the immediate settlement along the length of the approach embankment will vary. Given that the analysis is carried out at the critical section with the thickest overburden, the estimated settlements generally represent the maximum value along the north approach embankment.



6.10.2.2 Parameter Selection

Given that the south approach embankment will be founded directly on bedrock there will be no settlement of the foundation strata at this location. The simplified stratigraphy and the associated strength and unit weight employed for the estimation of settlement of the foundation soils at the north approach embankment are provided below. The immediate compression of the overburden soils are modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

Approach Embankment	Soil Type	Thickness (m)	Unit Weight (kN/m ³)	Elastic Modulus, E (MPa)
North Approach Embankment	Silt to Silt and Sand	3.3	19	5

For the purpose of the settlement analysis, the groundwater level is assumed to be located at the ground surface.

6.10.2.3 Settlement of Foundation Soils

The result of the analysis of the estimated settlement of the foundation soils at the north approach embankment under the loading imposed by the up to about 11.5 m high rock fill mass is presented below.

Approach Embankment	Soil Type	Estimated Settlement of Foundation Soils (mm)
North Approach Embankment	Silt to Silt and Sand	145

This settlement is expected to occur rapidly (i.e. during construction) in response to filling based on the non-cohesive nature of the foundation soils.

6.10.2.4 Settlement of Rock Fill Embankment

It is understood that rock fill is to be used for the construction of the approach embankments and as such, there will be settlement due to compression of the rock fill itself under self-weight. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Rock fill should be placed, in a controlled manner (i.e. not end dumped) in accordance with SP 206S03 (Rock Excavation, Grading). According to "MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates" (2010) the settlement of rock fill placed in this manner is expected to be nominal and the magnitude is estimated to be up to about 0.75 per cent (south approach) and 1.0 per cent (north approach) of the effective height of the rock fill embankment. As such, the estimated settlement of rock fill for the approach embankments is as following:



Embankment	Maximum New Embankment Height ¹ (m)	Estimated Settlement of Rock Fill (mm)	
		Short Term	Long Term
South Approach Embankment	5.0	40	5
North Approach Embankment	11.5	115	15

Note: 1. After removal of organics.

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six (6) months and will be mostly completed within one (1) year following the completion of the embankment to full height. Therefore, the north approach embankment should be constructed sufficiently in advance of other construction operations to allow for a recommended minimum four (4) month preload period. Following a four month preload period, the long-term post-construction settlement of the rockfill embankment meets the settlement performance criterion of 25 mm or less of settlement over a 20-year period following the completion of construction in accordance with MTO's "Embankment Settlement Criteria for Design" (2010).

6.10.3 Embankment Platform Widening

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, "Northern Region Embankment Design Guidelines" (1998), the minimum required embankment widening at this site to account for the estimated post-construction settlement and for future pavement overlays is 2 m per embankment side.

6.11 Subgrade Preparation and Embankment Construction

The following sections provide details on the recommendations for subgrade preparation and embankment construction.

6.11.1 Removal of Organic Materials

Prior to the placement of any fill, all organic materials (including peat and organic silts) as encountered at the site should be stripped from the plan limits of the proposed works.

6.11.2 Embankment Fill Placement

Placement of rock fill material for approach embankment construction should be carried out in accordance with the requirements as outlined in the Special Provision 206S03 Grading. The rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Blading, dozing and 'chinking' the rock fill 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 1.25 horizontal to 1 vertical (1.25H:1V).



6.12 Design and Construction Considerations

6.12.1 Overburden Excavation

In order to construct the bridge foundation on the bedrock at the proposed elevation for the south abutment, an excavation extending through up to about 0.5 m of peat and silty sand and into the bedrock will be required. The excavation will extend below the local perched groundwater level. The overburden soils are considered Type 4 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). Excavation through these overburden soils should be carried out with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). Where the excavation extends into the bedrock to accommodate the shallow foundations, it may be made with vertical or near vertical cut slopes as discussed in Section 6.12.4.

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended).

6.12.2 Granular Pad

At the north abutment, an excavation up to about 1.5 m deep will be required for the removal of organic materials followed by construction of a compacted granular pad to the underside of the CSP liners which are required as part of the integral abutment design. The granular pad should be constructed of OPSS.Prov 1010 Granular 'B' Type II and in accordance with SP206S03 (Earth Excavation, Grading). The top of the granular pad should extend at least 1 m beyond the plan limits of the pile cap in all directions and be sloped no steeper than one horizontal to one vertical (1H:1V). Given the requirement for sub-excavation of about 1.5 m of organic soils, proposed underside of pile cap at Elevation 195.2 m and 3 m long steel H-piles inside the CSP liners, the granular pad is expected to be about 2.5 m thick.

6.12.3 Control of Groundwater and Surface Water

The groundwater level measured at this site is generally between about 0.3 m above ground surface (i.e. equivalent ponded column of water) and 4.8 m below existing ground surface as noted in the boreholes/drillholes during the subsurface investigation. Hand shovel excavations B304-06 and B304-07 were dry upon completion. Foundation construction for the south abutment will require an excavation up to about 0.5 m of overburden to reach the bedrock and hence groundwater inflow into the excavation is not expected to be an issue.

At the north abutment, the groundwater level was measured at the ground surface or at about the proposed footing elevation, therefore the removal of the 1.5 m thick deposit of organic materials will be carried out below the groundwater level. However, it is expected that pumping from properly filtered sumps from within the excavation will be sufficient to control the groundwater inflow within the relatively shallow depths of excavation required. All surface water should be directed away from the excavations.

6.12.4 Temporary Protection System

Where there are space restrictions/constraints for an open excavation at the north abutment due to the proximity of the CP railway tracks, temporary protection system should be designed and constructed in accordance with AREMA requirements.



6.13 Recommendations for Rock Excavation and Blasting

6.13.1 Rock Excavation

It should be noted that the bedrock at the site is generally classified as strong (R4) to very strong (R5) (with measured unconfined compressive strengths of about 106 MPa and 125 MPa). As such, bedrock excavation in the vicinity of the proposed structure foundations for the south abutment should be carried out using line drilling and pre-shearing techniques to minimize blast damage to the rock (i.e. shattering and over-break) and provide better control over the configuration of the founding surface. The overall slope of the rock face may be formed vertically, or near vertically at the south abutment. In addition, following excavation, it will be necessary to remove all loose, shattered and/or fractured rock within the footprint of the foundations and to ensure that the founding rock is cleaned and protected such that the integrity of the rock is maintained.

At the north abutment, if foundation piles need to extend into bedrock to achieve a minimum pile length for integral abutment design or to improve lateral fixity at the base of the pile, socketing into the bedrock could be carried out by DTH hammer drilling.

6.13.2 Blasting

The use of explosives should follow the specifications outlined in OPSS 120 (Use of Explosives). It is recommended that control of all blasting operations be carried out in accordance with SP 299F06 (Rock Excavation (Controlled Blasting)).

It is recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a Quality Verification Engineer (QVE) soon after blasting to assess if the blasting operations have affected the integrity of the rock mass that will ultimately be supporting the new abutment footings. All loose, unstable rock should be removed from the cut faces in accordance with SP 299F03 (Rock Excavation (Machine Scaling)).

6.13.3 Rock Bolting

At the south abutment, if potentially unstable rock blocks or wedges cannot be removed safely or where the removal of such blocks/wedges could undermine the rock mass above, rock bolting may be required. As such, if rock bolting is necessary it should be carried out in accordance with SP 229S07 (Rock Bolting). The extent of any rock bolting, if required, will only be known following an inspection of the final rock faces.

7.0 CLOSURE

This report was prepared by Messrs. Al Varshoi, M.E.Sc., and Christopher Ng, P.Eng. The technical aspects of the report were reviewed by Mr. J. Paul Dittrich, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P. Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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[http://capws/sites/0911116014highway69fourlaning/contract 3/reporting/final/cpr overhead structure nbl/09-1111-6014-3524 rpt 14jan20 cpr overhead structure nbl.docx](http://capws/sites/0911116014highway69fourlaning/contract%203/reporting/final/cpr%20overhead%20structure%20nbl/09-1111-6014-3524%20rpt%2014jan20%20cpr%20overhead%20structure%20nbl.docx)



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Ontario Geological Society. 1991. Geology of Ontario, Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Reese, L.C., 1975. Laterally Loaded Piles. GESA Report D-75-14, UCCC Report 75-14, Geotechnical Engineering Software Activity, University of Colorado Computing Centre, Boulder.

Terzaghi, K. 1955. Evaluation of Coefficients of Subgrade Modulus. Geotechnique, V. 297-326.

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D5731	Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications
ASTM D7012	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Contract Design Estimating and Documentation (CDED):

Special Provision 206S03	Amendment to OPSS 206 – Rock Excavation, Grading
Special Provision 299F03	Rock Excavation (Machine Scaling)
Special Provision 299F06	Rock Excavation (Controlled Blasting)
Special Provision 229S07	Rock Bolting



Ministry of Transportation Ontario:

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MTO Foundations. Embankment Settlement Criteria for Design. March 2010.

MTO Foundations. MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

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Northeastern Region Engineering Directive. Backfill to Structures Adjacent to Rock Embankment Approaches. November 2002.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provisional Standard Drawing:

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirements
OPSD 3101.200	Walls – Abutment, Backfill – Rock
OPSD 3121.150	Walls – Retaining, Backfill – Minimum Granular Requirement

Ontario Provincial Standard Specification:

OPSS 120	General Specification for Use of Explosives
OPSS 501	Construction Specification for Compacting
OPSS 903	Construction Specification for Deep Foundations
OPSS. Prov 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



TABLES



Table 1: Evaluation of Foundation Alternatives – CPR Overhead Structure NBL– South Abutment

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread/Strip Footings on Bedrock	1	<ul style="list-style-type: none"> Relatively straightforward construction. 	<ul style="list-style-type: none"> Requires removal of up to about 5.0 m of soil and bedrock to reach the proposed founding level. Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break. Precludes integral abutment design unless the north abutment is founded on Steel H-piles. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation options. Additional costs for vertical dowels in bedrock, if required to improve lateral resistance. 	<ul style="list-style-type: none"> Footing must be set-back from face of rock cut at least a distance defined by a line projected at 0.5H:1V upwards from toe of rock cut and not less than 2 m. Risk of rock shattering and over-break requiring repair with mass concrete before footing construction unless controlled blasting techniques used during rock excavation.
Steel H-piles Driven to Bedrock	NA ¹	<ul style="list-style-type: none"> The bedrock surface is higher than the proposed founding level at the south abutment and therefore, steel H-pile foundations driven to bedrock are not suitable. 			
Steel H-piles Socketed into Bedrock	NP ²	<ul style="list-style-type: none"> Allows for integral abutment design. 	<ul style="list-style-type: none"> Involves significant amount of excavation into medium strong to very strong bedrock to form trenches or rock socket drilling required to achieve minimum pile lengths. 	<ul style="list-style-type: none"> Higher relative cost than spread footing option due to additional costs for excavating trenches or drilling sockets into bedrock. 	<ul style="list-style-type: none"> Potential difficulty with trench excavation or installing sockets into the medium strong to very strong bedrock.
Drilled steel casings Socketed into Bedrock	NA ¹	<ul style="list-style-type: none"> The bedrock surface is higher than the proposed founding level at the south abutment and therefore, drilled steel casing foundations are not applicable. 			

Notes: 1. NA – Not Applicable
2. NP – Not Practical

Prepared by: AV/CN

Reviewed by: JPD/JMAC



Table 2: Evaluation of Foundation Alternatives – CPR Overhead Structure NBL– North Abutment

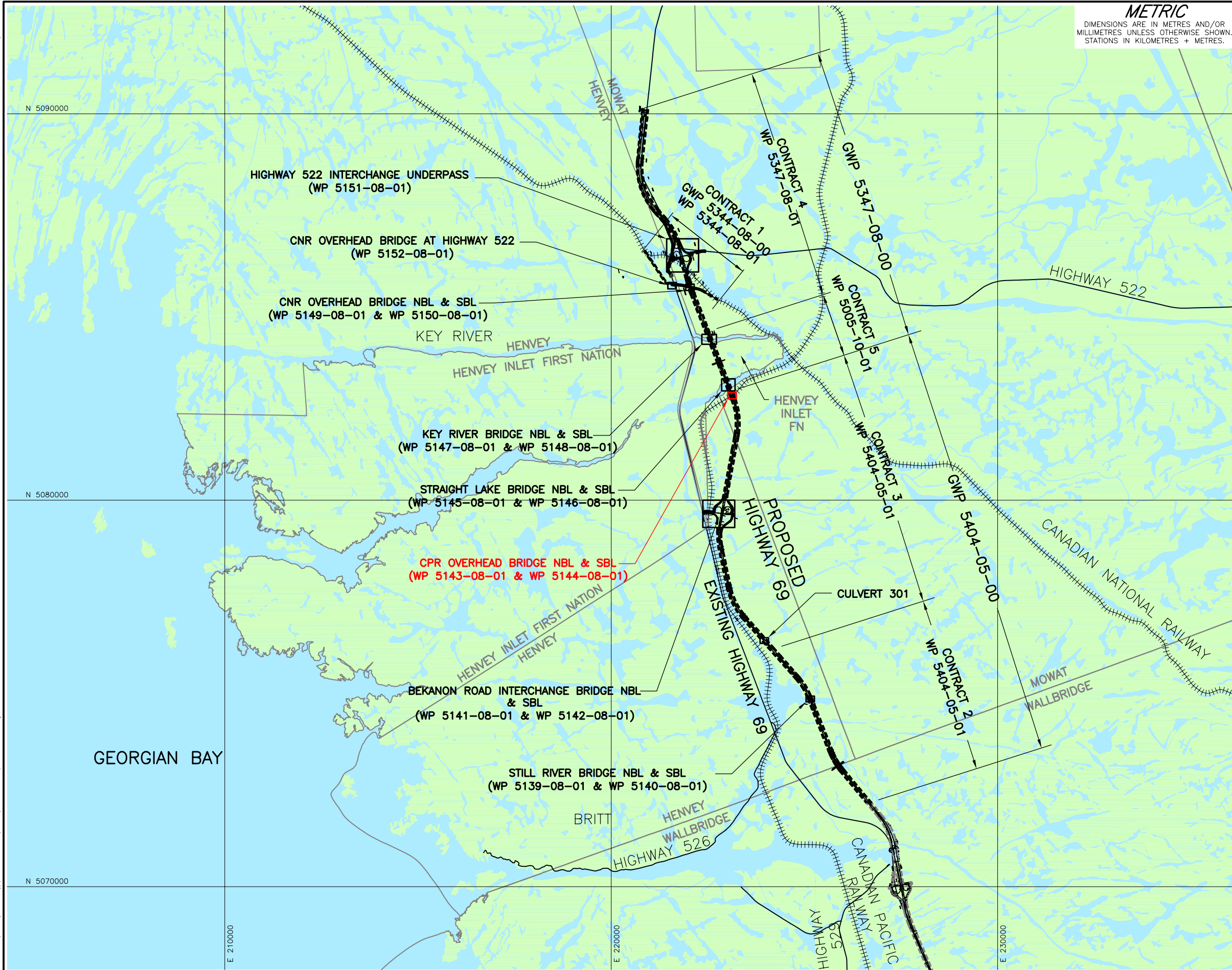
Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Steel H-piles Driven to Bedrock	1	<ul style="list-style-type: none"> Allows for integral abutment design. Negligible post-construction settlement. 	<ul style="list-style-type: none"> Sufficient lateral fixity may not be achieved within the overburden surrounding lower portion of piles below CSP. Socketing into bedrock may be required. 	<ul style="list-style-type: none"> Higher relative cost than spread footing option. Lower relative cost than steel H-piles and drilled steel casings socketed into bedrock. 	<ul style="list-style-type: none"> Risk that minimum length of pile required for integral abutment design may not be achieved.
Steel H-piles Socketed into Bedrock (using DTH Hammer drilling)	2	<ul style="list-style-type: none"> Allows for integral abutment design. Negligible post-construction settlement. 	<ul style="list-style-type: none"> Drilling with DTH hammer through into the medium strong to very strong bedrock required. Tremie concreting at the bottom of the rock sockets required to achieve pile fixity. 	<ul style="list-style-type: none"> Higher relative cost than spread footing and driven steel H-pile options. Lower relative cost than drilled steel casings socketed into bedrock option. Additional cost for specialized equipment for drilling holes/sockets into medium strong to very strong bedrock with DTH hammer. 	<ul style="list-style-type: none"> Potential for difficulty advancing temporary steel liner during DTH drilling into bedrock for sockets.
Spread Footings on Compacted Granular 'A' Pad	3	<ul style="list-style-type: none"> Relatively straightforward construction. 	<ul style="list-style-type: none"> Some post-construction settlement possible, but estimated to be less than 25 mm. Precludes integral abutment design if both the north and south abutments are on spread footings. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation options. Additional costs for construction of Granular 'A' pad. 	<ul style="list-style-type: none"> None.
Drilled Steel Casings socketed into Bedrock	4	<ul style="list-style-type: none"> Negligible post-construction settlement. 	<ul style="list-style-type: none"> Precludes integral abutment design. Drilling with DTH hammer into the medium strong to very strong bedrock required. 	<ul style="list-style-type: none"> Higher relative cost than spread footing and steel H-pile options. Additional cost for specialized equipment for drilling holes/sockets into medium strong to very strong bedrock with DTH hammer. 	<ul style="list-style-type: none"> Potential for difficulty advancing temporary steel liner during DTH drilling into bedrock for sockets.

Prepared by: AV/CN

Reviewed by: JPD/JMAC



DRAWINGS



PLAN



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

CONT No.
GWP No. 5404-05-00

HIGHWAY 69
SITE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN
NOT TO SCALE

SHEET

REFERENCE				
Base Data - MNR NRVIS, obtained 2004, CANMAP v2008 Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2008 Datum : NAD 83 Projection : MTM Zone 10				
Geocres No. 41H-138				
NO.	DATE	BY	REVISION	
HWY. 69	PROJECT NO. 09-1111-6014		DIST.	
SUBM'D. TVA	CHKD. TVA	DATE: Oct. 2013		SITE:
DRAWN: JFC	CHKD. CN	APPD. JPD/JMAC		DWG. 1

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

CONT No.
WP No. 5143-08-01



HIGHWAY 69

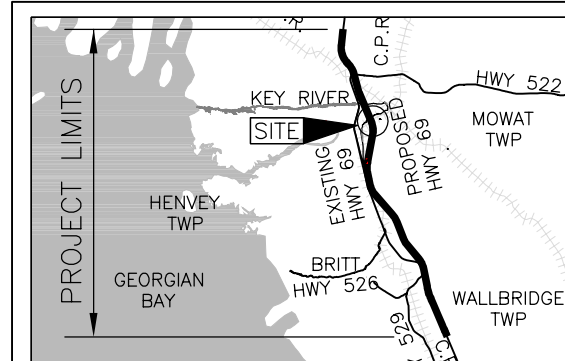
CPR OVERHEAD BRIDGE NBL

SHEET

BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
5 0 5 10 km

LEGEND

- Borehole - Current Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on March 09, 2013
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B304-01	198.9	5082691.6	223179.6
B304-02	197.6	5082705.6	223166.5
B304-03	197.7	5082710.4	223164.4
B304-04	197.5	5082711.5	223171.0
B304-05	200.0	5082712.4	223177.9
B304-06	196.2	5082717.5	223174.5
B304-07	194.8	5082765.0	223148.7
B304-08	191.1	5082745.4	223149.9
B304-09	191.1	5082740.7	223151.8
B304-10	191.2	5082746.6	223156.4
B304-11	191.2	5082752.8	223161.4
B304-12	191.1	5082748.1	223163.3

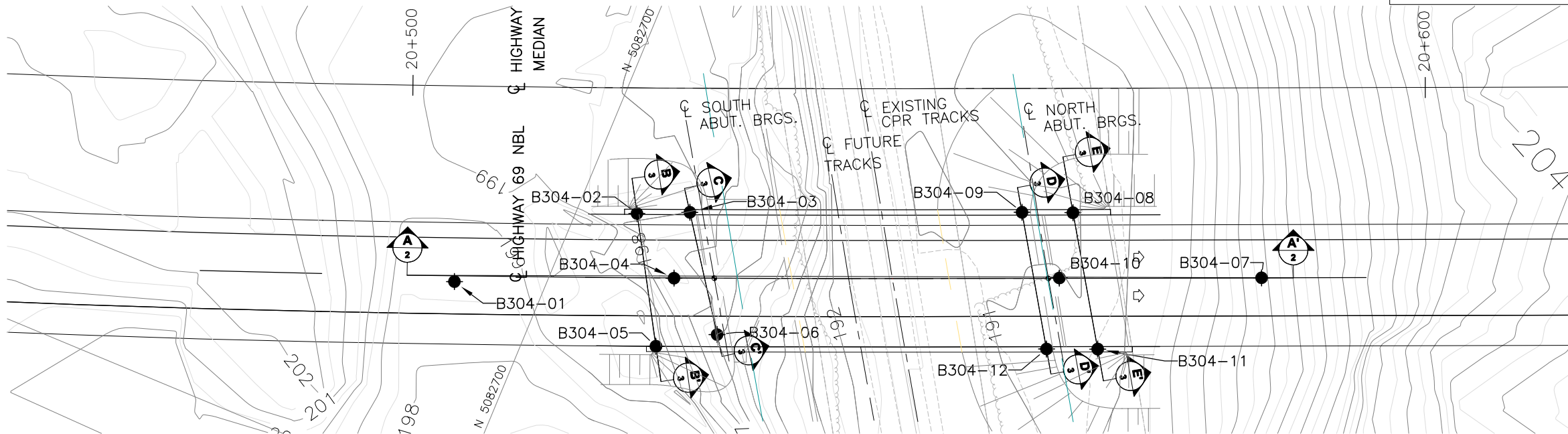
NOTES

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

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

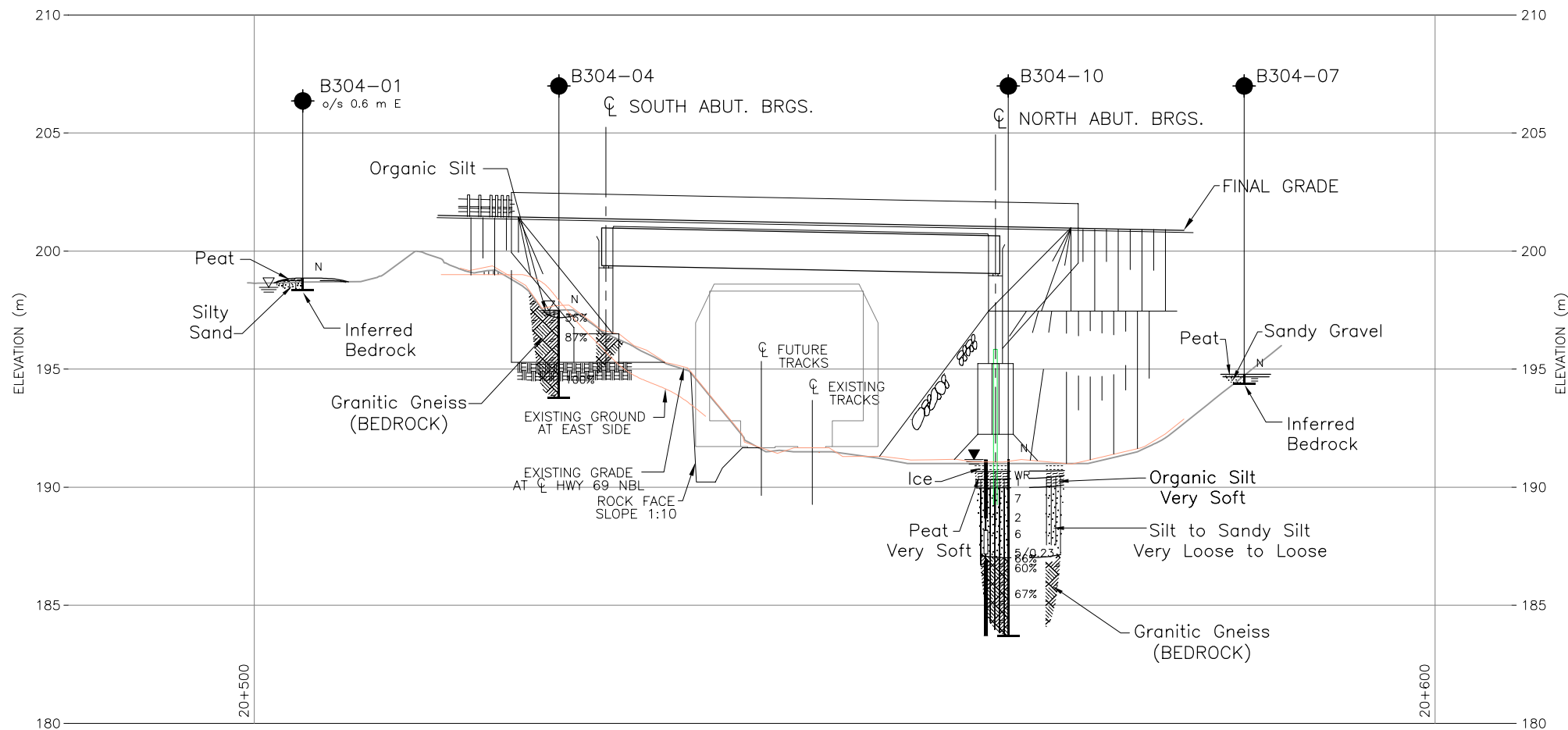
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

NO.	DATE	BY	REVISION
Geocres No. 41H-138			
HWY. 69	PROJECT NO. 09-1111-6014		DIST.
SUBM'D. AV	CHKD. CN	DATE: Oct. 2013	SITE: 44-460/1
DRAWN: MR	CHKD. AV	APPD. JPD/JMAC	DWG. 2



PLAN

SCALE
5 0 5 10 m



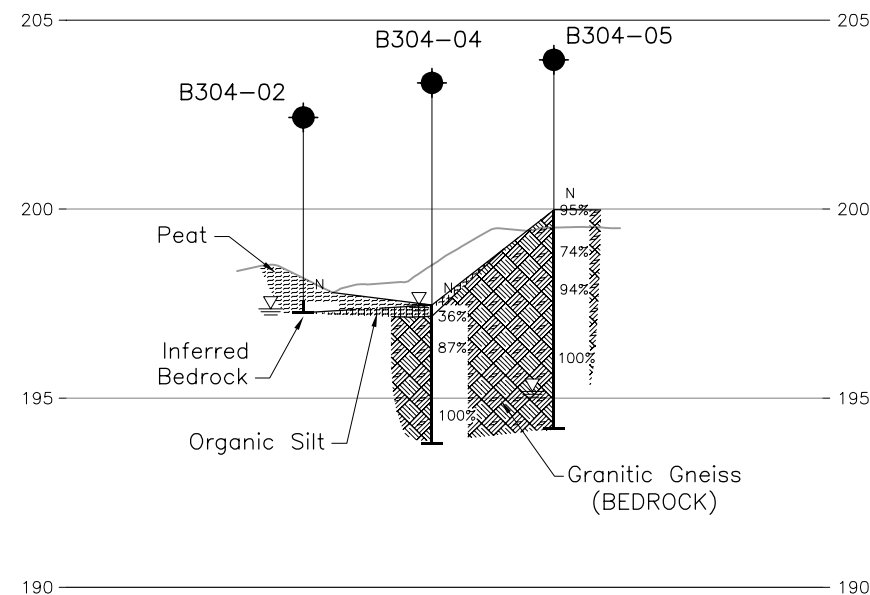
**CENTRELINE PROFILE
HIGHWAY 69 (NBL)**

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

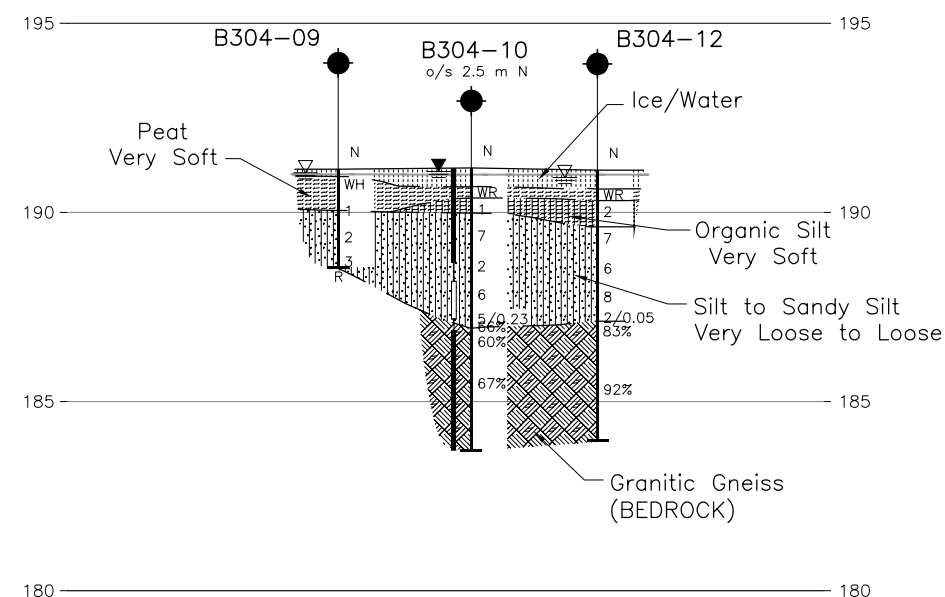
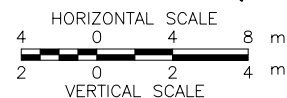
REFERENCE

Base plans provided in digital format by URS, drawing file nos.
Hwy69_Contour-Plan_C3.dwg, Hwy69_Contour-Plan_C5.dwg, received April
23, 2013 and August 31, 2012. CPR NBL Skew_GA.dwg, received July 11,
2013.

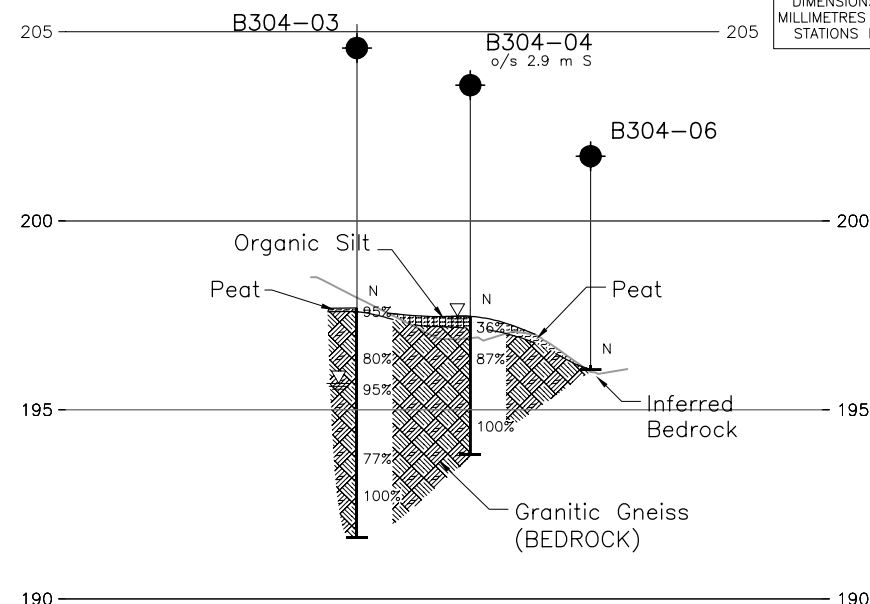
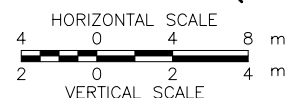




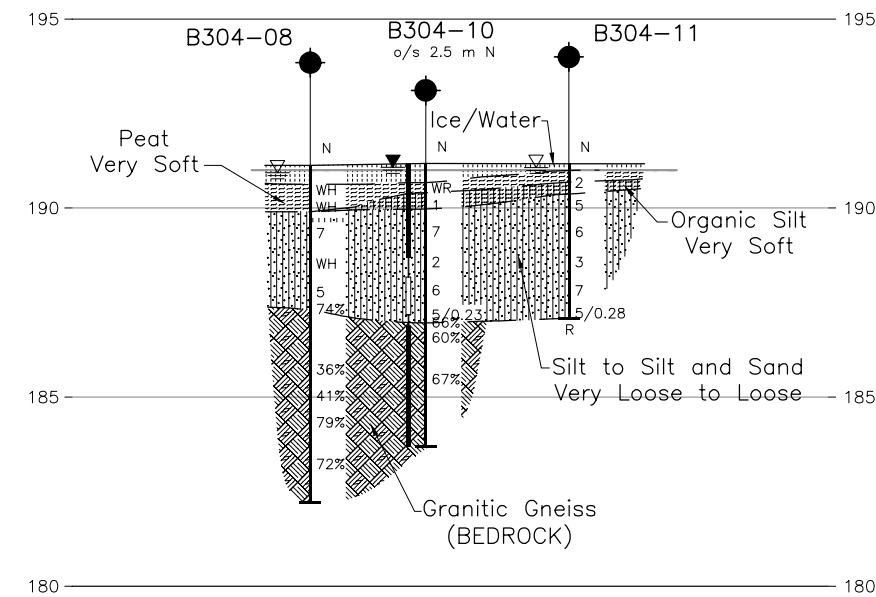
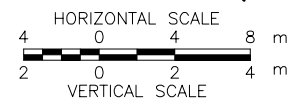
SOUTH ABUTMENT
HIGHWAY 69 (NBL)



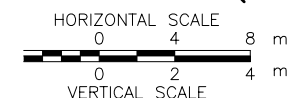
(D-D')
2



SOUTH ABUTMENT
HIGHWAY 69 (NBL)



NORTH ABUTMENT
HIGHWAY 69 (NBL)



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

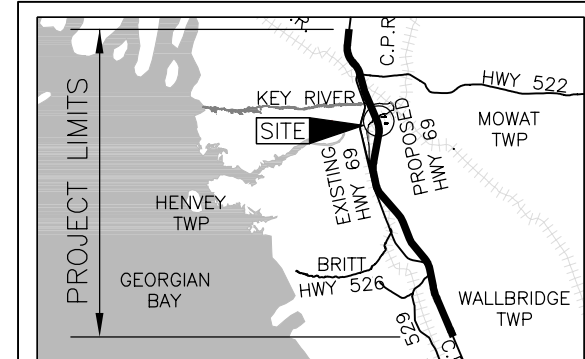
CONT No.
WP No. 5143-08-01

HIGHWAY 69
CPR OVERHEAD BRIDGE NBL
SOIL STRATA

SHEET








Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
5 0 5 10 km

LEGEND

- | | |
|---|--|
|  | Borehole — Current Investigation |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
|  | WL in piezometer, measured on March 9, 2013 |
|  | WL upon completion of drilling |

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NO.	DATE	BY	REVISION		
Geocres No. 41H-138					
HWY. 69			PROJECT NO. 09-1111-6014		DIST.
SUBM'D. AV		CHKD. CN	DATE: Oct. 2013		SITE: 44-460/1
DRAWN: MR		CHKD. AV	APPD. JPD/JMAC		DWG. 3





APPENDIX A

Record of Borehole and Drillhole Sheets



LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	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_α	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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.

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS 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 and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied 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 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 abbreviation 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

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B304-01		SHEET 1 OF 1		METRIC										
W.P. 5143-08-01		LOCATION N 5082691.6 ; E 223179.6		ORIGINATED BY MJR												
DIST _____ HWY 69		BOREHOLE TYPE Hand Shovel Excavation		COMPILED BY AV												
DATUM Geodetic		DATE March 1, 2013		CHECKED BY JPD												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
198.9	GROUND SURFACE															
0.0	PEAT (Fibrous) Dark Brown Wet					▽										
198.4	Silty SAND, some gravel, trace organics Brown Wet															
0.5	END OF EXCAVATION INFERRED BEDROCK															
NOTE: 1. Water level in excavation at a depth of 0.4 m below ground surface (Elev. 198.5 m) upon completion.																

PROJECT		RECORD OF BOREHOLE		No B304-02		SHEET 1 OF 1		METRIC								
W.P.		LOCATION		ORIGINATED BY												
DIST		BOREHOLE TYPE		COMPILED BY												
DATUM		DATE		CHECKED BY												
09-1111-6014		N 5082705.6 ; E 223166.5		MJR												
5143-08-01		Hand Shovel Excavation		AV												
Geodetic		March 1, 2013		JPD												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
197.6	GROUND SURFACE															
0.0	PEAT (Fibrous)															
197.3	Dark brown															
0.3	Wet															
	Cobbles															
	END OF EXCAVATION															
	INFERRED BEDROCK															
	NOTE:															
	1. Water level in excavation at a															
	depth of 0.2 m below ground															
	surface (Elev. 197.4 m) upon															
	completion.															

PROJECT		RECORD OF BOREHOLE		No B304-03		SHEET 1 OF 1		METRIC									
W.P. 09-1111-6014		LOCATION		N 5082710.4 ; E 223164.4		ORIGINATED BY		MJR									
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, NQ Coring		COMPILED BY									
AV		DATE		February 27, 2013		CHECKED BY		JPD									
Geodetic																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
197.7	GROUND SURFACE																
0.0	PEAT (Fibrous) Dark brown Wet		1	CS	-												
	Granitic Gneiss (BEDROCK)		1	RC	REC 100%												RQD = 95%
	Bedrock cored from depths of 0.1 m to 6.1 m		2	RC	REC 99%												RQD = 80%
	For bedrock coring details refer to Record of Drillhole B304-03.		3	RC	REC 98%												RQD = 95%
			4	RC	REC 100%												RQD = 77%
			5	RC	REC 100%												RQD = 100%
191.6	END OF BOREHOLE																
6.1	NOTE: 1. Water level in open borehole at a depth of 2.0 m below ground surface (Elev. 195.7 m) upon completion of drilling.																

PROJECT		RECORD OF BOREHOLE		No B304-04		SHEET 1 OF 1		METRIC	
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE	
DATE		COMPILED BY		CHECKED BY		DATUM		DATE	
09-1111-6014		N 5082711.5 ; E 223171.0		MJR		HWY 69		Portable Equipment, NQ Coring	
Geodetic		February 27, 2013		JPD					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	20	40	60			
197.5	GROUND SURFACE																
0.0	ORGANIC SILT		1	CS	-												
197.2	Dark brown																
0.3	Wet																
	Granitic Gneiss (BEDROCK)		1	RC	REC 90%												RQD = 36%
	Bedrock cored from depths of 0.3 m to 3.7 m																
	For bedrock coring details refer to Record of Drillhole B304-04.		2	RC	REC 99%												RQD = 87%
			3	RC	REC 100%												RQD = 100%
193.8	END OF BOREHOLE																
3.7	NOTE: 1. Water level in open borehole at ground surface (Elev. 197.5 m) upon completion of drilling.																

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B304-04

SHEET 1 OF 1

LOCATION: N 5082711.5 ; E 223171.0

DRILLING DATE: February 27, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: LANDCORE DRILLING

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	RECOVERY			FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY K, cm/sec			Diametral Point Load Index (MPa)	RMC -Q- AVG.	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
					DEPTH (m)	TOTAL CORE %				SOLID CORE %	R.Q.D. %	TYPE AND SURFACE DESCRIPTION					Jr	Ja	Jn	10 to 15	15 to 20				20 to 25	25 to 30																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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			Continued from Record of Borehole B304-04		197.18																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
1	NRQ February 27, 2013		Slightly weathered to fresh, thinly laminated to thinly bedded, light to dark grey with pink interbeds, fine to coarse grained, faintly porous, strong to very strong GRANITIC GNEISS		0.30																				7.9 MPa (Axial)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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DEPTH SCALE

1 : 50



LOGGED: MJR/CS

CHECKED: JPD

GTA-RCK 018 09-1111-6014.GPJ GAL-MISS.GDT 01/16/14

PROJECT		RECORD OF BOREHOLE		No B304-05		SHEET 1 OF 1		METRIC									
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE		COMPILED BY		DATUM		DATE		CHECKED BY	
09-1111-6014		N 5082712.4 ; E 223177.9		MJR		HWY 69		Portable Equipment, NQ Coring		AV		Geodetic		February 26, 2013		JPD	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
200.0	GROUND SURFACE							20	40	60	80	100	W _p	W	W _L		
0.0	Granitic Gneiss (BEDROCK)		1	RC	REC 100%												RQD = 95%
	Bedrock cored from ground surface to a depth of 5.8 m		2	RC	REC 98%												RQD = 74%
	For bedrock coring details refer to Record of Drillhole B304-05.		3	RC	REC 100%												RQD = 94%
			4	RC	REC 100%												RQD = 100%
194.2	END OF BOREHOLE																
5.8	NOTE: 1. Water level in open borehole at a depth of 4.8 m below ground surface (Elev. 195.2 m) upon completion of drilling.																

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B304-05

SHEET 1 OF 1

LOCATION: N 5082712.4 ; E 223177.9

DRILLING DATE: February 26, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: LANDCORE DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SH - 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 VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
				DEPTH								
				(m)								
0	NORQ February 26, 2013	Continued from Record of Borehole B304-05		199.98								
0		Slightly weathered to fresh, thinly laminated to thinly bedded, light to dark grey with pink interbeds, fine to coarse grained, faintly porous, medium strong to very strong GRANITIC GNEISS		0.00								
1												
2												
3												
4												
5												
6		END OF DRILLHOLE		194.19								
7												
8												
9												
10												

DEPTH SCALE

1 : 50



LOGGED: MJR/CS

CHECKED: JPD

GTA-RCK 018 09-1111-6014.GPJ GAL-MISS.GDT 01/16/14

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B304-06		SHEET 1 OF 1		METRIC										
W.P. 5143-08-01		LOCATION N 5082717.5 ; E 223174.5		ORIGINATED BY MJR												
DIST _____ HWY 69		BOREHOLE TYPE Hand Shovel Excavation		COMPILED BY AV												
DATUM Geodetic		DATE March 1, 2013		CHECKED BY JPD												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
196.2	GROUND SURFACE															
0.0	PEAT (Fibrous) Dark brown Wet															
0.1	END OF BOREHOLE INFERRED BEDROCK															
NOTE: 1. Excavation dry upon completion.																

GTA-MTO 001 09-1111-6014.GPJ GAL-GTA.GDT 01/16/14

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B304-07		SHEET 1 OF 1		METRIC										
W.P. 5143-08-01		LOCATION N 5082765.0 ; E 223148.7		ORIGINATED BY MJR												
DIST _____ HWY 69		BOREHOLE TYPE Hand Shovel Excavation		COMPILED BY AV												
DATUM Geodetic		DATE March 6, 2013		CHECKED BY JPD												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
194.8	GROUND SURFACE															
0.0	PEAT (Fibrous)		1	CS	-											
194.4	Dark brown		2	CS	-											
0.4	Wet															
	Sandy GRAVEL, trace to some silt, trace clay															
	Brown															
	Wet															
	END OF EXCAVATION INFERRED BEDROCK															
	NOTE: 1. Excavation dry upon completion.															

PROJECT		RECORD OF BOREHOLE		No B304-08		SHEET 1 OF 1		METRIC					
W.P.		LOCATION		ORIGINATED BY		COMPILED BY		CHECKED BY					
DIST		BOREHOLE TYPE		DATE		DATE		DATE					
Geodetic		March 1, 2013		March 1, 2013		March 1, 2013		March 1, 2013					
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL		
191.1	ICE SURFACE												
0.0	ICE												
0.3	PEAT (Fibrous) Very soft Dark brown Wet		1	SS	WH								
			2	SS	WH								
189.9	SILT and SAND, trace clay Very loose to loose Grey Wet		3	SS	7								
1.2			4	SS	WH								
			5	SS	5								
187.5	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 3.6 m to 8.7 m For bedrock coring details refer to Record of Drillhole B304-08.		1	RC	REC 99%								
3.6			2	RC	REC 100%								
			3	RC	REC 97%								
			4	RC	REC 100%								
			5	RC	REC 99%								
182.4	END OF BOREHOLE NOTE: 1. Water level in open borehole at ice surface (Elev. 191.1 m) upon completion of drilling.												
8.7													

SHEET 1 OF 1

DATUM: Geodetic

DRILL RIG: Portable Equipment

CHECKED: JPD

GTA-RCK 018 09-1111-6014.GPJ GAL-MISS.GDT 01/16/14

PROJECT		RECORD OF BOREHOLE		No B304-09		SHEET 1 OF 1		METRIC							
W.P.		LOCATION		ORIGINATED BY		COMPILED BY		CHECKED BY							
DIST		BOREHOLE TYPE		DATE		DATE		DATE							
Geodetic		March 7, 2013		March 7, 2013		March 7, 2013		March 7, 2013							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	W _p W W _L	W _p W W _L	γ	GR SA SI CL	
191.1	ICE SURFACE														
0.0	ICE														
0.2	WATER														
	PEAT (Amorphous)		1	SS	WH		191								
	Very soft														
	Dark brown		2A	SS	1		190								
	Wet		2B	SS	1										
190.0	SILT, some sand, trace clay, trace gravel														
1.1	Very loose														
	Grey		3	SS	2										
	Wet														
188.5	END OF BOREHOLE SPOON AND CASING REFUSAL		4	SS	3		189								
2.6	NOTE: 1. Water level in open borehole at a depth of 0.1 m below ice surface (Elev. 191.0 m) upon completion of drilling.														

PROJECT		RECORD OF BOREHOLE		No B304-10		SHEET 1 OF 1		METRIC						
W.P. 09-1111-6014		LOCATION		N 5082746.6 ; E 223156.4		ORIGINATED BY		MJR						
DIST _____ HWY 69		BOREHOLE TYPE		Portable Equipment, NW Casing, Wash Boring		COMPILED BY		AV						
DATUM Geodetic		DATE		March 5, 2013		CHECKED BY		JPD						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
191.2	ICE SURFACE							20	40	60	80	100		
190.9	ICE													
190.6	PEAT (Fibrous) Very soft Dark brown Wet		1	SS	WR									
190.2	ORGANIC SILT Very soft Dark brown Wet		2A	SS	1									
1.0	SANDY SILT, trace clay Very loose to loose Grey Wet		2B											
			3	SS	7									
			4	SS	2									
188.4	SILT, trace to some sand, trace clay Loose Grey Wet		5	SS	6									0 21 77 2
2.8			6	SS	5/0.23									Non-Plastic
187.2	Granitic Gneiss (BEDROCK)		1	RC	REC 95%									0 11 86 3
4.0	Bedrock cored from depths of 4.0 m to 7.3 m For bedrock coring details refer to Record of Drillhole B304-10.		2	RC	REC 100%									RQD = 66%
			3	RC	REC 96%									RQD = 60%
184.0	END OF BOREHOLE													RQD = 67%
7.3	NOTES: 1. Water level in piezometer at a depth of 0.1 m below ice surface (Elev. 191.1 m) on March 9, 2013. 2. Piezometer decommissioned on March 9, 2013.													

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: LANDCORE DRILLING

[illegible]

CHECKED: JPD

GTA-RCK 018 09-1111-6014.GPJ GAL-MISS.GDT 01/16/14

PROJECT		RECORD OF BOREHOLE		No B304-11		SHEET 1 OF 1		METRIC								
W.P. 09-1111-6014		LOCATION		N 5082752.8 ; E 223161.4		ORIGINATED BY		MJR								
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, NW Casing, Wash Boring		COMPILED BY								
CC/AV		DATE		March 7, 2013		CHECKED BY		JPD								
DATUM		Geodetic														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
191.2	ICE SURFACE															
0.0	ICE															
190.7	WATER															
190.4	PEAT(Fibrous)		1A	SS	2											
0.8	Very soft Dark brown Wet		1B	SS	5											
	ORGANIC SILT, trace sand		2	SS	5											
	Very soft															
	Grey		3	SS	6											
	Wet															
	Sandy SILT, trace organics															
	Very loose to loose		4	SS	3											
	Grey															
	Wet		5	SS	7											
187.1	END OF BOREHOLE SPOON AND CASING REFUSAL		6	SS	5/0.28											
4.1	NOTE: 1. Water level in open borehole at a depth of 0.1 m below ice surface (Elev. 191.1 m) upon completion of drilling.															

PROJECT		RECORD OF BOREHOLE		No B304-12		SHEET 1 OF 1		METRIC														
W.P.		LOCATION		ORIGINATED BY																		
DIST		BOREHOLE TYPE		COMPILED BY																		
DATUM		DATE		CHECKED BY																		
09-1111-6014		N 5082748.1 ; E 223163.3		MJR																		
5143-08-01		Portable Equipment, NW Casing, Wash Boring		CC/AV																		
Geodetic		March 6, 2013		JPD																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
191.1	0.0	ICE SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 20 40 60			81.5 OC = 4.5%					
		ICE						191														
		WATER																				
190.5	0.6	PEAT(Fibrous) Very soft Dark brown Wet		1	SS	WR																
				2	SS	2																
189.8	1.3	ORGANIC SILT Very soft Dark brown Wet		3	SS	7																
		SILT, trace to some sand, trace clay Loose Grey Wet		4	SS	6																
188.3	2.8	Sandy SILT Loose Grey Wet		5	SS	8																
				6	SS	2/0.05																
187.2	3.9	Granitic Gneiss (BEDROCK)																				
		Bedrock cored from depths of 3.9 m to 7.0 m For bedrock coring details refer to Record of Drillhole B304-12.		1	RC	REC 99%																
				2	RC	REC 98%																
184.1	7.0	END OF BOREHOLE																				
		NOTE: 1. Water level in open borehole at ice surface (Elev. 191.1 m) upon completion of drilling.																				

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B304-12

SHEET 1 OF 1

LOCATION: N 5082748.1 ;E 223163.3

DRILLING DATE: March 7, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: LANDCORE DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SH - 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 VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
				DEPTH									
				(m)									
4	NW Casing	Continued from Record of Borehole B304-12		187.23									
5	NQRQ March 7, 2013	Slightly weathered to fresh, thinly laminated to thinly bedded, light to dark grey with pink interbeds, fine to coarse grained, faintly porous, strong to very strong GRANITIC GNEISS		3.89	1								
6					2								
7		END OF DRILLHOLE		184.11									
8				7.01									
9													
10													
11													
12													
13													

DEPTH SCALE

1 : 50



LOGGED: MJR/CS

CHECKED: JPD

GTA-RCK 018 09-1111-6014.GPJ GAL-MISS.GDT 01/16/14



APPENDIX B

Laboratory Test Results and Photographs

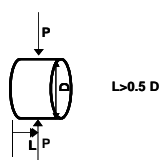
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm)	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
B304-03	1	0.50	197.2	Granite Gneiss	Axial	38.30	41.40	9.45	113
B304-03	1	1.30	196.4	Granite Gneiss	Axial	31.50	41.50	11.507	138
B304-03	2	2.10	195.6	Granite Gneiss	Axial	30.30	41.90	9.240	111
B304-03	3	3.80	193.9	Granite Gneiss	Axial	31.80	41.60	12.205	146
B304-03	4	4.50	193.2	Granite Gneiss	Axial	28.00	41.60	12.277	147
B304-03	5	5.80	191.9	Granite Gneiss	Axial	31.00	41.60	11.913	143
B304-03	1	0.50	197.2	Granite Gneiss	Diametral	41.40	79.00	3.485	42
B304-03	1	1.30	196.4	Granite Gneiss	Diametral	41.50	87.00	4.829	58
B304-03	2	2.10	195.6	Granite Gneiss	Diametral	41.90	71.00	6.742	81
B304-03	3	3.80	193.9	Granite Gneiss	Diametral	41.60	106.00	6.838	82
B304-03	4	4.50	193.2	Granite Gneiss	Diametral	41.60	78.00	8.744	105
B304-03	5	5.80	191.9	Granite Gneiss	Diametral	41.60	67.00	7.837	94
B304-04	1	0.30	197.2	Granite Gneiss	Axial	51.1	113.00	7.941	95
B304-04	2	1.80	195.7	Granite Gneiss	Axial	51.00	81.00	8.598	103
B304-04	3	3.20	194.3	Granite Gneiss	Axial	51.00	92.00	10.878	131
B304-04	1	0.30	197.2	Granite Gneiss	Diametral	41.0	77.00	6.096	73
B304-04	2	1.80	195.7	Granite Gneiss	Diametral	41.50	101.00	7.431	89
B304-04	3	3.20	194.3	Granite Gneiss	Diametral	41.60	94.00	7.786	93
B304-05	1	0.80	199.2	Granite Gneiss	Axial	34.1	41.40	8.315	100
B304-05	2	1.90	198.1	Granite Gneiss	Axial	28.20	41.60	3.131	38
B304-05	3	3.30	196.7	Granite Gneiss	Axial	28.7	41.60	6.122	73
B304-05	4	4.60	195.4	Granite Gneiss	Axial	33.50	41.50	10.168	122
B304-05	4	5.70	194.3	Granite Gneiss	Axial	33.80	41.60	9.166	110
B304-05	1	0.80	199.2	Granite Gneiss	Diametral	41.40	79.00	6.778	81
B304-05	2	1.90	198.1	Granite Gneiss	Diametral	41.60	63.00	5.023	60
B304-05	3	3.30	196.7	Granite Gneiss	Diametral	41.60	73.00	7.776	93
B304-05	4	4.60	195.4	Granite Gneiss	Diametral	41.50	95.00	9.111	109
B304-05	4	5.70	194.3	Granite Gneiss	Diametral	41.60	96.00	8.058	97
B304-08	1	4.50	186.6	Granite Gneiss	Axial	41.40	79.00	3.485	42
B304-08	3	6.20	184.9	Granite Gneiss	Axial	31.00	41.60	6.219	75
B304-08	4	7.30	183.8	Granite Gneiss	Axial	22.60	41.60	11.018	132
B304-08	5	8.00	183.1	Granite Gneiss	Axial	40.00	41.60	7.518	90
B304-08	1	4.50	186.6	Granite Gneiss	Diametral	41.70	89.00	5.346	64
B304-08	3	6.20	184.9	Granite Gneiss	Diametral	41.60	73.00	3.197	38
B304-08	4	7.30	183.8	Granite Gneiss	Diametral	41.60	104.00	7.010	84
B304-08	5	8.00	183.1	Granite Gneiss	Diametral	41.60	84.00	10.822	130

⁽¹⁾ $I_{s50} \times K$, from ASTM Designation: D 5731 "Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications". A value of $K = 12$ has been used based on 4 UCS tests done for SBL and NBL samples at the bridge location

DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

note: Diametral tests are perpendicular to core axis
(planes of weakness)



AXIAL SPECIMEN SHAPE REQUIREMENTS

note: Axial tests are parallel to core axis
(planes of weakness)

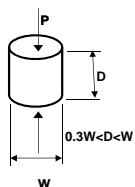


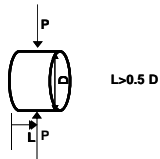
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm)	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
B304-10	2	5.00	186.2	Granite Gneiss	Axial	21.50	41.30	8.852	106
B304-10	3	6.10	185.1	Granite Gneiss	Axial	30.50	41.40	11.586	139
B304-10	3	7.00	184.2	Granite Gneiss	Axial	31.20	41.50	10.137	122
B304-10	2	5.00	186.2	Granite Gneiss	Diametral	41.30	112.00	6.783	81
B304-10	3	6.10	185.1	Granite Gneiss	Diametral	41.40	89.00	8.078	97
B304-10	3	7.10	184.1	Granite Gneiss	Diametral	41.50	84.00	7.815	94
B304-12	1	4.90	186.2	Granite Gneiss	Axial	26.90	41.60	11.128	134
B304-12	2	6.00	185.1	Granite Gneiss	Axial	30.70	41.50	10.201	122
B304-12	1	4.90	186.2	Granite Gneiss	Diametral	41.60	101.00	6.314	76
B304-12	2	6.00	185.1	Granite Gneiss	Diametral	41.50	80.00	7.036	84
B304-12	2	6.90	184.2	Granite Gneiss	Diametral	41.50	87.00	6.935	83

⁽¹⁾ $I_{S50} \times K$, from ASTM Designation: D 5731 "Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications". A value of $K = 12$ has been used based on 4 UCS tests done for SBL and NBL samples at the bridge location

DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

note: Diametral tests are perpendicular to core axis (planes of weakness)



AXIAL SPECIMEN SHAPE REQUIREMENTS

note: Axial tests are parallel to core axis (planes of weakness)

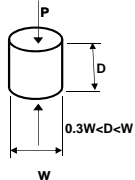


TABLE B2-1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
CPR OVERHEAD STRUCTURE NBL
HIGHWAY 69 GWP 5404-05-00; WP 5143-08-01

Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B304-03 (1)	1.1	196.6	Granite Gneiss	41.5	125.1
B304-12 (1)	5.1	186.0	Granite Gneiss	41.4	105.7

Compiled By: AVChecked By: CNReviewed By: JPD/JMAC

Table B2-2
UNCONFINED COMPRESSION TEST (UC)
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1111-6014	RUN NUMBER	1
BOREHOLE NUMBER	B304-03	SAMPLE DEPTH, m	1.00-1.10

TEST CONDITIONS

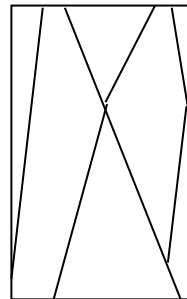
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.20

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.14	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	4.15	UNIT WEIGHT, kN/m ³	26.61
SAMPLE AREA, cm ²	13.51	DRY UNIT WT., kN/m ³	26.58
SAMPLE VOLUME, cm ³	123.49	SPECIFIC GRAVITY	-
WET WEIGHT, g	335.20	VOID RATIO	-
DRY WEIGHT, g	334.80		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	125.1
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REMARKS:

DATE:

4/8/2013

TABLE B2-3
UNCONFINED COMPRESSION TEST (UC)
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1111-6014	RUN NUMBER	1
BOREHOLE NUMBER	B304-12	SAMPLE DEPTH, m	5.00-5.10

TEST CONDITIONS

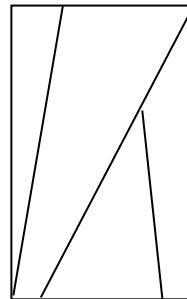
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.28	WATER CONTENT, (specimen) %	0.23
SAMPLE DIAMETER, cm	4.14	UNIT WEIGHT, kN/m ³	26.34
SAMPLE AREA, cm ²	13.46	DRY UNIT WT., kN/m ³	26.28
SAMPLE VOLUME, cm ³	124.90	SPECIFIC GRAVITY	-
WET WEIGHT, g	335.59	VOID RATIO	-
DRY WEIGHT, g	334.82		

VISUAL INSPECTION

FAILURE SKETCH



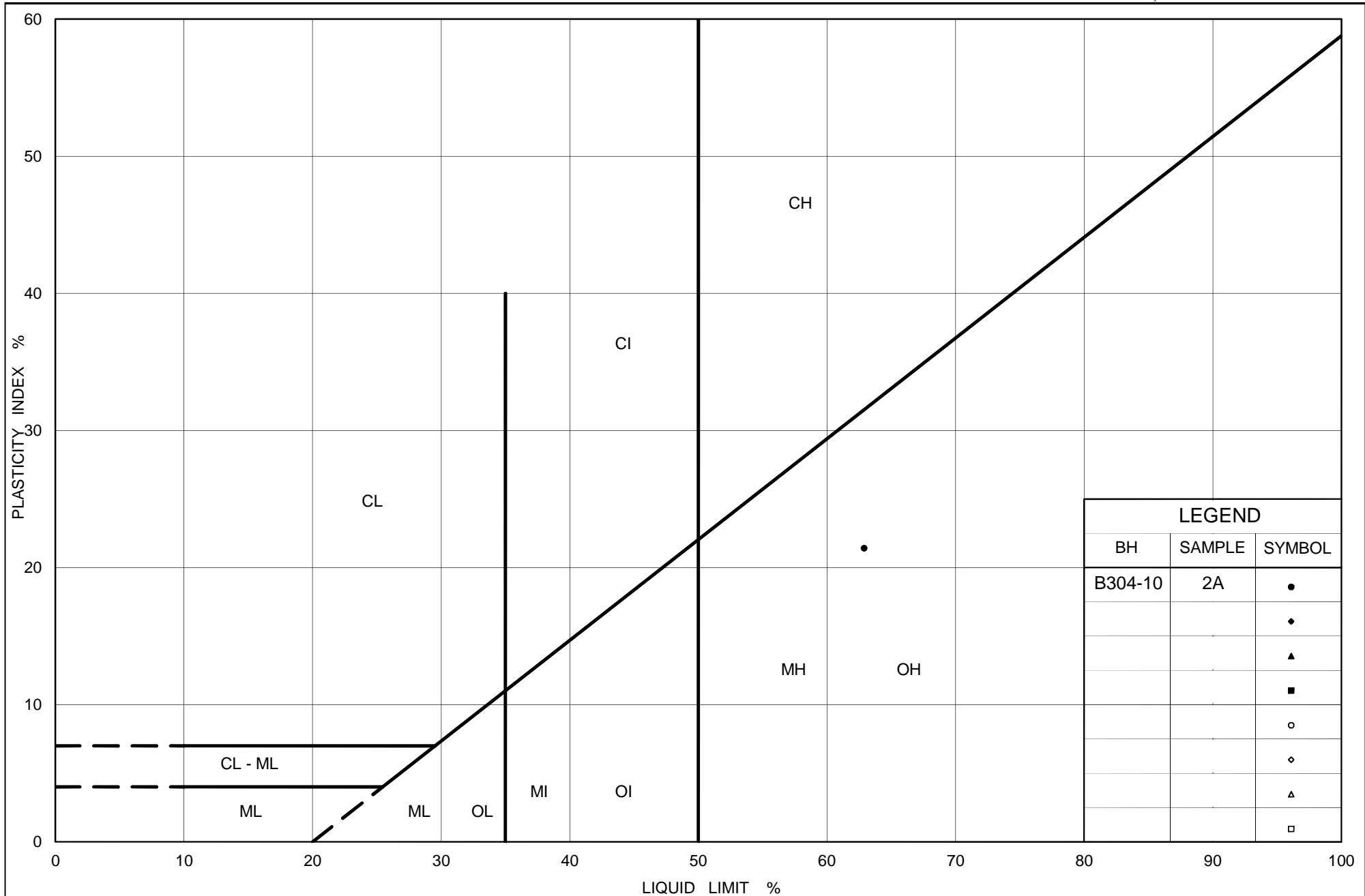
TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	105.7
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REMARKS:

DATE:

4/8/2013



Ministry of Transportation

Ontario

PLASTICITY CHART
Organic Silt
CPR Overhead Structure NBL

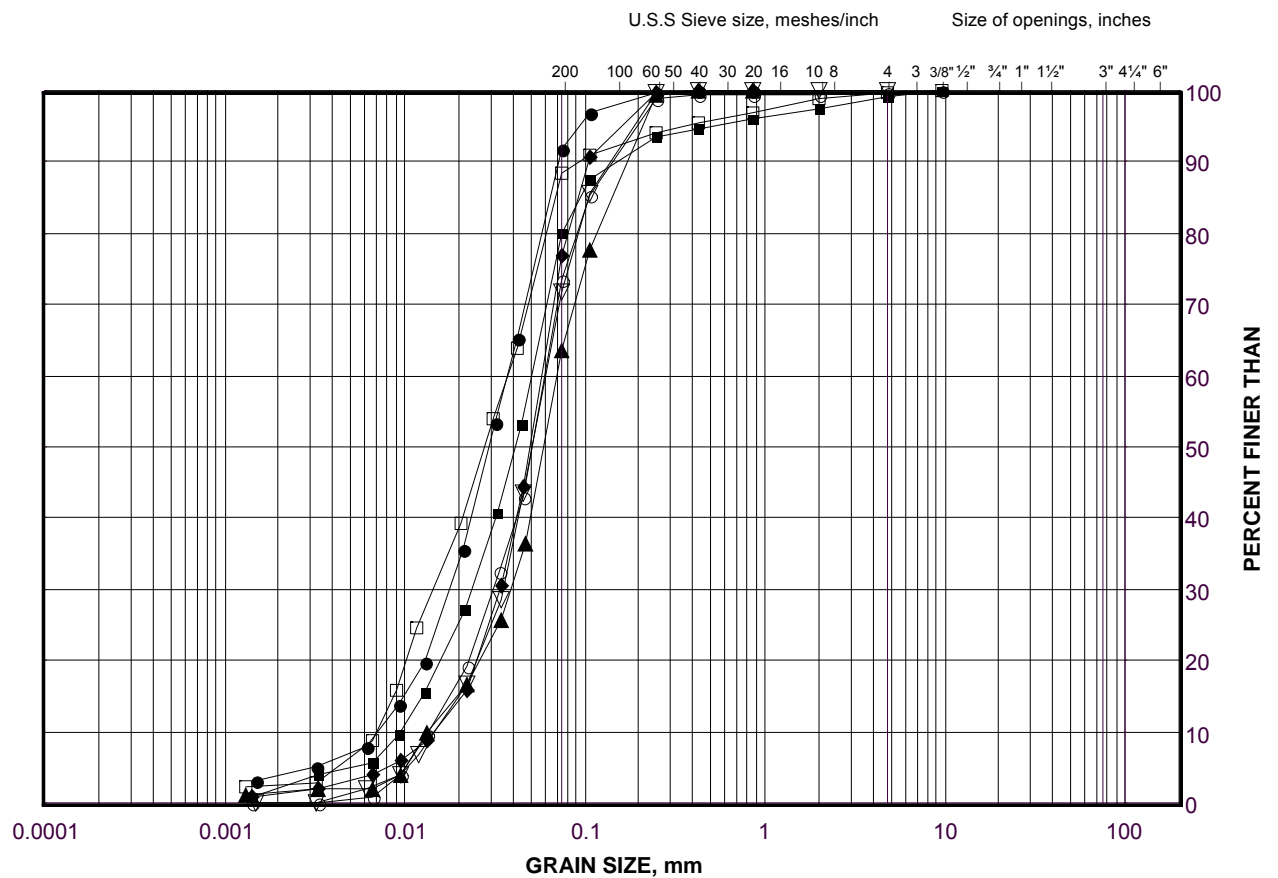
Figure No. B1

Project No. 09-1111-6014

Checked By: GRL

Silt to Silt and Sand
CPR Overhead Structure NBL

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B304-12	3	189.5
■	B304-09	3	189.3
◆	B304-10	4	188.6
▲	B304-08	4	188.7
▽	B304-11	4	188.6
○	B304-12	5	187.9
□	B304-10	6	187.3

Project Number: 09-1111-6014

Checked By: GRL

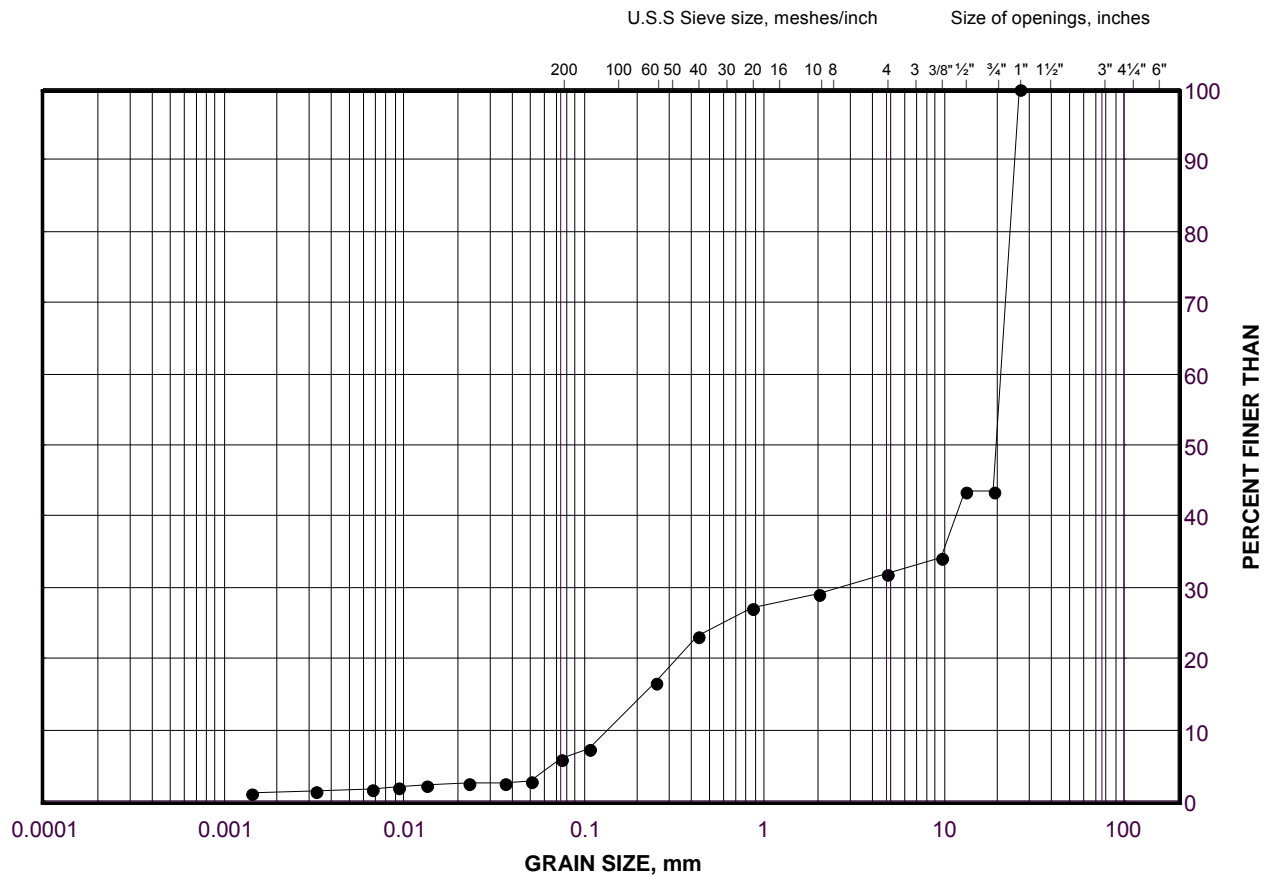
Golder Associates

Date: 19-Jun-13


GRAIN SIZE DISTRIBUTION

Sandy Gravel
CPR Overhead Structure NBL


FIGURE B3





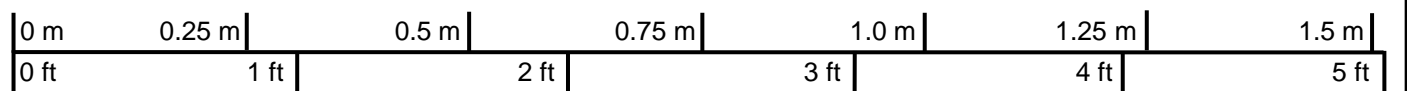
PROJECT	CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01				
	TITLE Rock Surface Photograph Highway 69 (NBL)				
	PROJECT No.	09-1111-6014	FILE No.	----	
	DESIGN	AV	AUG 13	SCALE	NTS
	CADD	--	---		REV
	CHECK	CN	AUG 13	FIGURE B4	
	REVIEW	JPD/JMAC	AUG 13		




PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01			
TITLE		Rock Surface Photograph Highway 69 (NBL)			
	PROJECT No.	09-1111-6014		FILE No.	----
	DESIGN	AV	AUG 13	SCALE	NTS
	CADD	--	---	FIGURE B5	
	CHECK	CN	AUG 13		
	REVIEW	JPD/JMAC	AUG 13		

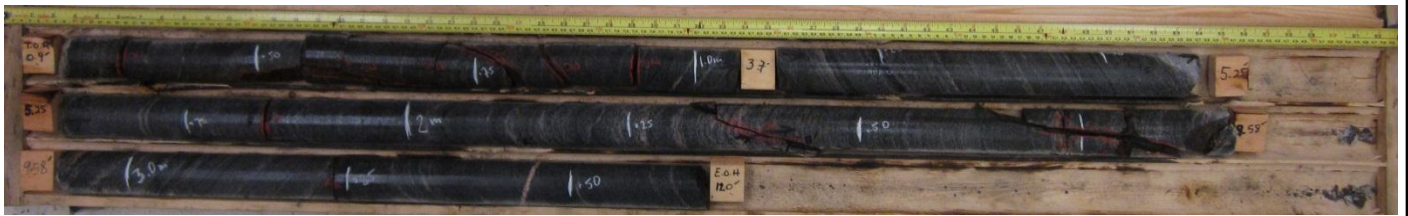
Three long, dark, cylindrical objects, possibly ancient scrolls or artifacts, are laid out horizontally on a wooden surface. They are marked with handwritten numbers and labels. The top object is labeled "For 025" and "44". The middle object is labeled "44" and "89". The bottom object is labeled "89" and "1308". A yellow measuring tape is visible along the top edge.

Two long, dark, cylindrical objects, possibly astronomical instruments, lying horizontally on a wooden surface. A yellow measuring tape is visible at the top. The objects have various markings, including numbers and letters. The top object is labeled '130' and '140'. The bottom object is labeled '163' and '150'. A small white label on the right of the bottom object reads '6000' and '19.9'.

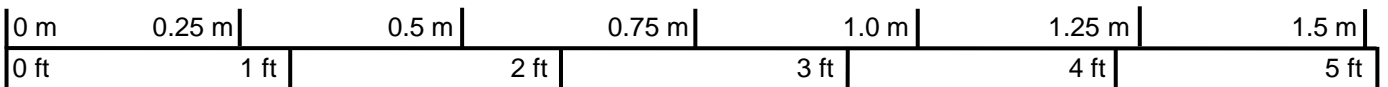


PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01					
TITLE Bedrock Core Photograph – B304–03 Highway 69 (NBL)							
		PROJECT No. 09-1111-6014		FILE No. ----			
		DESIGN	AV	JUN 13	SCALE	NTS	REV.
		CADD	-- --				
		CHECK	CN	JUN 13	FIGURE B6		
		REVIEW	JPD/JMAC	AUG 13			

Borehole B304-04



Box 1: 0.30 m – 3.66 m




Scale

PROJECT		CPR OVERHEAD NBL STRUCTURE Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01			
TITLE		Bedrock Core Photograph – B304–04 Highway 69 (NBL)			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	AV	JUN 13	SCALE NTS
		CADD	--	--	REV.
		CHECK	CN	JUN 13	FIGURE B7
		REVIEW	JPD/JMAC	AUG 13	

Two long, dark, cylindrical objects, possibly measuring tapes or rods, are laid out horizontally on a wooden surface. The top object has markings and numbers: 12.5, 1.0, 1.25, 1.5, 1.75, 2.0. The bottom object has markings and numbers: 1.50, 1.75, 2.0. A small label is attached to the bottom object with the text "E.O.H. 190".

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01					
TITLE Bedrock Core Photograph – B304–05 Highway 69 (NBL)							
		PROJECT No. 09-1111-6014		FILE No. ----			
		DESIGN	AV	JUN 13	SCALE	NTS	REV.
		CADD	--				
		CHECK	CN	JUN 13	FIGURE B8		
		REVIEW	JPD/JMAC	JUN 13			

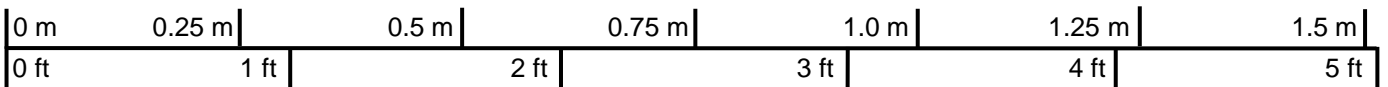
Borehole B304-08




Box 1: 3.80 m – 7.89 m



Box 2: 7.89 m – 8.93 m



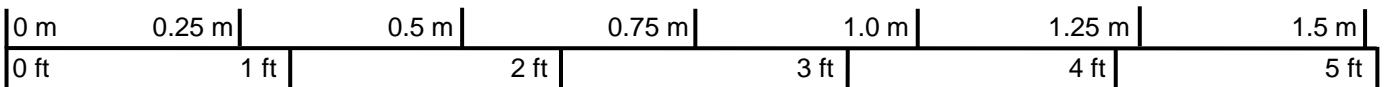
Scale

PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01			
TITLE		Bedrock Core Photograph – B304–08 Highway 69 (NBL)			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	AV	JUN 13	SCALE NTS
		CADD	--	--	REV.
		CHECK	CN	JUN 13	FIGURE B9
		REVIEW	JPD/JMAC	JUN 13	


Borehole B304-10



Box 1: 4.20 m – 7.47 m




Scale

PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01			
TITLE		Bedrock Core Photograph – B304–10 Highway 69 (NBL)			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	AV	JUN 13	SCALE NTS
		CADD	--	--	REV.
		CHECK	CN	JUN 13	FIGURE B10
		REVIEW	JPD/JMAC	JUN 13	

Three sections of a dark, cylindrical core sample, likely a geological drill core, laid out horizontally on a wooden surface. A yellow measuring tape is positioned above the sections. The sections are marked with handwritten labels and numbers. The top section is labeled "70.9" and "13.25". The middle section is labeled "19.16". The bottom section is labeled "50.4" and "25.5". The sections show varying degrees of weathering and discoloration.



PROJECT		CPR OVERHEAD STRUCTURE NBL Highway 69 Four-Laning GWP 5404-05-00; WP 5143-08-01					
TITLE Bedrock Core Photograph – B304–12 Highway 69 (NBL)							
		PROJECT No. 09-1111-6014		FILE No. ----			
		DESIGN	AV	JUN 13	SCALE	NTS	REV.
		CADD	---				
		CHECK	CN	JUN 13	FIGURE B11		
		REVIEW	JPD/JMAC	AUG 13			



APPENDIX C

Non-Standard Special Provisions

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

SCOPE

This special provision covers the requirements for the placement and field testing of dowels into rock.

CONSTRUCTION

Dowels into rock shall be constructed in accordance with OPSS 904 Concrete Structures. All reinforcing steel supplied shall be in accordance with OPSS 1440 Steel Reinforcement for Concrete (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole 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 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 D3689, ASTM D1143/D1143M and ASTM D4435. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

PERFORMANCE TESTS

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

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 69 / CPR Overhead Structure (NBL)	North and South 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:

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 point.

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, three (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 per cent of the theoretical elastic elongation of the free stressing length and is less than the theoretical elongation of the free stressing length plus 50 per cent of the bond length.

BASIS OF PAYMENT

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

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract

drawings, and shall be galvanized in accordance with CSA G164-M. CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to bedrock.
4. Place loose sand into 600 mm diameter CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

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