



December 15, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**KEY RIVER NBL BRIDGE, SITE NO.44-462/1
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 5005-10-00; WP 5147-08-01**

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REPORT





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PART A

FOUNDATION INVESTIGATION REPORT

KEY RIVER NBL BRIDGE, SITE NO. 44-462/1

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 5005-10-00; WP 5147-08-01



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 foundation investigation services for the proposed Highway 69 northbound lane (NBL) structure over Key River (Site No. 44-462/1), which is within the Contract 5 limits of the new Highway 69 alignment. The proposed work in Contract 5 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 (CPR) and Canadian National Railway (CNR) overpass structures; as well as culvert crossings. The Key River NBL structure is to be located approximately 550 m east of the existing Highway 69. The general location of this proposed bridge along the new Highway 69 four-laning alignment is shown on the Index Plan on Drawing 1.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated December, 2008. Golder's proposal for foundation engineering services associated with the Contract 5 Key River 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 Quality Control Plan for foundation engineering services for this project, dated April 19, 2010.

This report addresses the investigation carried out for the Key River NBL structure only. Separate reports address the foundation investigations for the related swamp crossings and high fill areas, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed bridge location, 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. The general arrangement of the proposed structure presented on Drawing 2 was provided to us by URS on November 4, 2013.

Preliminary subsurface information for this project is available and was supplied by the MTO, specifically:

- Preliminary Foundation Investigation and Design Report for Structural Areas (Foundation Investigation 2), Highway 69 Four Laning, From 3.5 km North of Highway 559 to 3.8 km North of Highway 522, GWP 5377-02-00, GEOCRE No. 41H-57, dated July 2006, by Amec Earth and Environmental.

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 Henvey Inlet First Nation Reserve No. 2 and the Township of Mowat to the north. The Contract 5 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 1.6 km in Henvey Inlet First Nation Reserve No. 2. The proposed Key River NBL structure is located approximately 0.5 km east of the existing Highway 69 alignment within the Contract 5 highway alignment and is located approximately 0.2 km from the northern limit of Contract 5, corresponding to approximately 10.3 km north 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 or 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. In the immediate area of the NBL bridge site, the topography consists of rolling terrain with densely treed areas and high bedrock outcrops, covered in places with low scrub-brush adjacent to the river. The bedrock outcrops generally slope upward steeply from the river to the north and south shores near the proposed north and south abutments. At the south abutment and along the approach embankment, the bedrock outcrops rise from the river surface (at about Elevation 176 m) and extend as high as about Elevation 198 m, resulting in outcrop heights up to about 22 m above the river level. At the north abutment and approach embankment, the bedrock outcrops rise from the river surface to greater than Elevation 204 m, resulting in outcrop heights greater than about 28 m above the river level.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

Golder's fieldwork for the proposed Highway 69 NBL structure over Key River was carried out between November 7 and 16, 2012 as well as between July 16 and August 7, 2014, during which time a total of eighteen (18) boreholes were advanced at or adjacent to the locations of the footprint of each proposed foundation element footprints and the approaches. These boreholes were supplemented with two (2) boreholes advanced along the centreline between the NBL and SBL structures. A summary of the respective boreholes and their respective locations relative to each foundation element and approach area is presented below.

Foundation Element/ Approach Area	Borehole No.
South Approach Embankment	B504-03
South Abutment	B504-04
	B504-05
	B504-06
	B504-07
	B504-08
	B504-01
South Pier (Pier 1)	B504-09
	B504-10*
	B504-11
	B504-02
North Pier (Pier 2)	B504-12
	B504-13
	B504-14*
	B504-15
North Abutment	B504-16
	B504-17
	B504-18
	B504-19
	B504-20
North Approach Embankment	B504-20

Note: *Boreholes advanced near centreline between NBL and SBL structures.



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 are shown in plan on Drawing 2.

The boreholes at the approaches/abutments on the bedrock outcrops were advanced using portable drilling equipment supplied and operated by Ohlmann Geotechnical Services (OGS) Inc. of Almonte, Ontario. The bedrock surface was exposed and confirmed at these boreholes under a thin layer of overburden encountered at several locations. Boreholes B504-06 and B504-17 were advanced at the centre of the proposed south and north abutments by coring to depths of 8.1 m and 10.0 m, respectively, below ground surface. Photographs of the bedrock outcrops on the south and north shores of the river in the immediate vicinity of the proposed abutments are presented on Figures 1 and 2 respectively.

The boreholes in Key River were advanced from a barge using a D-55 or D-120 drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes in the river were advanced to depths of up to about 34.2 m below the water surface, to between about 14.9 m and 28.7 m below river bottom, through a water column between about 2.4 m and 7.4 m deep.

Photographs of the set-up of the drilling operations on the bedrock outcrops at the abutments and on the barge in the river are shown on Figures C1 and C2 in Appendix C. In addition, it is noted that an underwater hydro cable (owned by Hydro One Inc.) exists within Key River in the vicinity of the proposed south pier footprints of the SBL and NBL bridge structures. This cable had to be located prior to start of the in-water field investigations using an underwater diving supplied by ASI Group. Photographs showing the set-up of the dive crew are shown on Figure C3 in Appendix C. The approximate location of the underwater cable in the vicinity of the south piers is shown on Drawing 2.

The boreholes were advanced through the overburden using HW casing with wash boring techniques. In general, soil samples were obtained at intervals of depth between about 0.75 m and 3.0 m, using a 50 mm outer diameter split-spoon sampler operated by automatic hammers on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Bedrock coring was carried out using an 'HQ' and/or 'NQ' core barrel. Photographs of the recovered rock core samples are provided in Appendix B. It is noted that no split-spoon sampling was carried out in boreholes B504-10, B504-11, B504-13 and B504-14, however bedrock was confirmed by coring in each of these boreholes. The overburden in these boreholes was inferred from observations during the drilling and from information in the adjacent boreholes.

The groundwater conditions were observed during the drilling operations and all boreholes were backfilled upon completion 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, consisting of 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.

At the abutments, approaches and piers, the boreholes were located in the field and the ground/water surface elevations were surveyed by Callon Dietz prior to drilling. The locations given on the Record of



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Borehole/Drillhole sheets and shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Water/Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
B504-01	5084111.4	222585.9	175.5*	17.3
B504-02	5084194.9	222551.6	175.5*	29.5
B504-03	5084050.1	222611.8	195.6	0.6 (Bedrock Outcrop)
B504-04	5084066.7	222597.1	191.1	Bedrock Outcrop
B504-05	5084072.5	222594.7	189.7	Bedrock Outcrop
B504-06	5084069.8	222603.7	190.8	8.1 (Bedrock Outcrop)
B504-07	5084067.3	222613.1	192.4	Bedrock Outcrop
B504-08	5084073.2	222610.7	190.1	Bedrock Outcrop
B504-09	5084121.2	222583.1	176.3*	29.3
B504-10	5084119.3	222562.6	176.3*	24.8
B504-11	5084118.3	222594.6	176.3*	29.3
B504-12	5084188.2	222554.7	176.3*	34.2
B504-13	5084192.0	222564.3	176.3*	32.4
B504-14	5084190.2	222532.4	176.3*	29.1
B504-15	5084236.5	222526.5	196.6	Bedrock Outcrop
B504-16	5084242.4	222524.0	199.6	Bedrock Outcrop
B504-17	5084239.6	222532.9	197.1	10.0 (Bedrock Outcrop)
B504-18	5084237.1	222542.4	193.6	Bedrock Outcrop
B504-19	5084243.1	222540.0	195.4	Bedrock Outcrop
B504-20	5084259.3	222524.8	204.4	Bedrock Outcrop

*Water surface; Borehole Depth includes water column.

3.2 Optical Borehole Logging

Geophysical borehole surveys (optical borehole logging) were carried out by Golder personnel on August 7 and 8, 2014. The surveys were conducted in the boreholes located at the mid-point at each abutment location (Boreholes B504-06 and B504-17) to collect detailed, oriented optical images of the borehole walls, and the images were interpreted for the type and orientation of the discontinuities intersected by the boreholes. The survey depths are summarized below.

Borehole No.	Borehole Location	Optical Televiwer Depth Range (m)	Caliper Depth Range (m)
B504-06	South Abutment	1.6 to 7.95	1.35 – 7.7
B504-17	North Abutment	1.6 to 8.8	1.35 – 9.65

The optical televiwer and caliper surveys were carried out using an ALT Optical Borehole Imager (ALT-OB140) and a Caliper Probe (2PCA-1000), respectively. The optical televiwer generates a high resolution digital image



of the borehole wall and is capable of resolving fractures as narrow as 0.1 mm at a radial resolution of 1 degree. The data is recorded together with data from an internal magnetometer and a tiltmeter allowing the determination of the orientation (dip and dip direction) of the structural features recorded. The caliper probe measures the borehole diameter with three linked arms operating a single resistive sensor in the probe. The data is used to determine the average borehole diameter and indicate borehole anomalies such as rough borehole walls or washouts.

The survey data was processed using WellCAD software (Advanced Logic Technology Ltd.) and oriented to magnetic north prior to image interpretation. The downhole logs from the optical borehole survey are shown on the Geophysical Record of Borehole sheets presented in Appendix D.

The data were oriented to geographic (true) north prior to interpretation using a magnetic declination of 10.318 degrees.

3.3 Evaluation of Photographic Records and Bedrock Mapping

The bedrock conditions in the area of the abutment locations were assessed using the data from the optical borehole logging as well as photographic records of the rock faces (see Figures D1 to D8 in Appendix D) and the results were used to identify potential failure modes which might require pre-support, stabilization or remedial measures during or following excavation.

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.

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

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



4.2 Subsurface Conditions

The detailed subsurface soil, bedrock 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 Record of Borehole and Drillhole sheets and on the laboratory test figures provided in Appendix A and Appendix B, respectively. The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-sections are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The bedrock surface has been inferred from observations made during drilling and coring and generally represents a transition from overburden to the bedrock surface and should not be inferred to represent the exact surface elevation of the bedrock. Furthermore, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawings 2 to 4 is a simplification of the subsurface conditions.

The subsurface conditions at the site of the NBL structure are characterized essentially by: granitic gneiss bedrock outcrops, in places covered by thin layers of topsoil and granular soils, at the south and north abutments/approaches; and by a sequence of organic silt, clayey silt and gravelly sandy silt to sand to gravelly sand deposits, underlain by granitic gneiss bedrock at the south and north pier below the river water level.

The results of the strength tests on the rock core samples are presented in Tables B1 and B2 and the results of the laboratory testing on the soil samples are presented on Figures B1 to B3, in Appendix B. Photographs of the bedrock core samples are presented on Figures B4 to B10, inclusive, in Appendix B.

A detailed description of the subsurface conditions encountered in the boreholes at the approaches/abutments and at the piers is provided in the following sections.

Because the boreholes were advanced on bedrock outcrops or in the water, and water was introduced into the boreholes during the drilling process, the water level noted in the boreholes is not considered representative of groundwater conditions. Further, the groundwater and river water levels are subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.3 South Abutment/Approach

A total of six boreholes (B504-03 to B504-8) were advanced in the vicinity of the proposed south abutment/approach. Bedrock coring was carried out in Borehole B504-06. The interpreted stratigraphy at the south abutment is shown in profile on Drawing 2 and in cross-section on Drawing 3.

4.3.1 Topsoil

A 0.1 m thick deposit of topsoil was encountered at ground surface in Borehole B504-03 at Elevation 195.6 m.

4.3.2 Silty Sand

A 0.5 m thick deposit of brown silty sand with traces of rootlets was encountered below the topsoil layer in Borehole B504-03.

The natural water content measured on the sample of the silty sand is about 26 per cent.



4.3.3 Bedrock

Exposed bedrock outcrops were observed at ground surface at each of the borehole locations and bedrock core samples were recovered from Borehole B504-06. The depth to bedrock below ground surface in the south abutment/approach area and the corresponding bedrock surface elevation are summarized below.

Foundation Element	Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
South Abutment/Approach	B504-03	0.6	195.0	Refusal/Inferred Bedrock
	B504-04	0.0	191.1	Bedrock Exposed
	B504-05	0.0	189.7	Bedrock Exposed
	B504-06	0.0	190.8	Bedrock Exposed-Cored
	B504-07	0.0	192.4	Bedrock Exposed
	B504-08	0.0	190.1	Bedrock Exposed

In general, the bedrock surface along the south approach and in the area of the proposed south abutment of the NBL structure slopes downward from south to north and east to west, with the bedrock surface elevation changing by as much as about 2.7 m at the abutment borehole locations and up to 5.3 m relative to the approach borehole about 21 m south of the abutment.

Discontinuities in the rock mass noted in the walls of Borehole B504-06 recorded by the optical televiewer were predominantly minor open joints (opening width less than 10 mm) or healed joints. In general, no major anomalies were identified along the borehole walls.

Based on a review of the bedrock core samples recovered from Borehole B504-06, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh to slightly weathered, foliated, coarse grained, slightly porous, medium strong to strong, grey and pink, as presented in the Record of Drillhole sheet in Appendix A, and shown on the photograph of the recovered core samples on Figure B5 in Appendix B. The degree of weathering of the bedrock samples (e.g. slightly weathered –W2) is based on field identification, and the strength classification of the intact rock mass is based on laboratory identification (i.e., medium strong to strong – R3 to R4) is described in accordance with the International Society for Rock Mechanics (ISRM)³ standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 50 per cent to 98 per cent, indicating a rock mass of fair to excellent quality as per Table 3.10 of CFEM (2006)⁴. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 91 per cent and 100 per cent and between 14 per cent and 97 per cent, respectively.

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

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. The Canadian Geotechnical Society c/o BiTech Publisher Ltd., British Columbia.



sheets and are presented in Table B1 in Appendix B. The axial tests carried out on two samples of the granitic gneiss bedrock core measured Is_{50} values of about 6.2 MPa and 8.3 MPa and the diametral tests carried out on two samples of the granitic gneiss bedrock core measured Is_{50} values of about 7.4 MPa and 7.7 MPa.

One Unconfined Compression (UC) test (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) carried out on a selected sample of the granitic gneiss bedrock obtained in Borehole B504-06 measured a compressive strength of about 97 MPa as summarized in Table B2-1 and detailed in Table B2-3 in Appendix B.

Table B1 also presents estimated Uniaxial Compressive Strengths (UCS) correlated to the Point Load Test (PLT) strengths based on the relationship between Is_{50} and UCS. The relationship between Is_{50} and UCS values, given by correlation factor (K), varies depending on the size of the core sample and the strength of the rock. For the NBL (as well as for SBL) bridges using the consolidated rock strength data from both sites, an average correlation factor (K) was calculated by matching UCS test values and PLT values at similar depths from the same boreholes. The average correlation factor (K) of 14 was estimated.

Based on the UCS and PLT test results at the south abutment, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa).

4.3.4 Groundwater Conditions

The water level in Borehole B504-06 was not recorded upon completion of coring operations.

4.4 South Pier (Pier 1)

A total of four boreholes (B504-01 and B504-09 to B504-11) were advanced in the vicinity of the proposed south pier: soil sampling and bedrock coring was carried out in Boreholes B504-01 and B504-09; while Boreholes B504-10 and B504-11 were advanced to the bedrock surface without soil sampling but were cored to obtain additional bedrock information. The soil strata shown on the Record of Boreholes B504-10 and B504-11 are inferred based on the soil information from the adjacent Boreholes B504-01 and B504-09; the inferred soil strata from Boreholes B504-10 and B504-11 are not included in Sections 4.4.2 and 4.4.3. The interpreted stratigraphy at the south pier is shown in profile on Drawing 2 and in cross-section on Drawing 3.

4.4.1 Water

The water surface measured in Key River at the time of drilling Boreholes B504-01 and B504-09 to B504-11 in November 2012 and August 2014 was at Elevations 175.5 m and 176.3 m, respectively, and the depth of water at the boreholes was between 2.4 m and 3.8 m.

4.4.2 Organic Silt and Clayey Silt

A 5.8 m and 6.5 m thick deposit of brown to grey organic silt was encountered from the riverbed in Boreholes B504-01 and B504-09 at Elevations 173.1 m and 172.5 m, respectively, underlain by a 5 m and 3.4 m thick deposit of grey clayey silt and encountered at Elevations 167.3 m and 166.0 m, respectively. In Borehole



B504-01, sandy silt to silty sand seams were encountered within the clayey silt deposit, which extended to the top of bedrock.

The SPT 'N'-values measured within the organic silt deposit are 0 blows (i.e., weight of hammer), whereas the SPT 'N'-values in the clayey silt deposit range between 0 blows (i.e., weight of hammer) and 4 blows per 0.3 m of penetration. In situ field vane tests carried out within these deposits measured undrained shear strengths ranging between 9 kPa and 25 kPa in the organic silt deposit and between 19 and 46 kPa in the clayey silt deposit. The sensitivity is calculated at 2 to 3 in the organic silt deposit and range between 2 and 5 in the clayey silt. The field vane test results indicate that the organic silt and clayey silt deposits have a very soft to soft and soft to firm consistency, respectively.

The natural water content measured on four samples of the organic silt range from 53 per cent and 108 per cent and on three samples of the clayey silt deposit and from about 29 per cent to 43 per cent. The organic content measured on two samples of the organic silt deposit is 4.6 per cent and 11.4 per cent.

Atterberg limits tests were carried out on six samples of the deposits and measured liquid limits ranging from about 25 per cent to 54 per cent, plastic limits ranging from about 13 per cent to 33 per cent and plasticity indices ranging from about 12 per cent to 21 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figures B1A and B1B in Appendix B and, together with the organic contents, indicate that the materials are classified as an organic silt of intermediate to high plasticity and clayey silt of low plasticity, respectively.

The results of the grain size distribution tests completed on two samples of the organic silt and one sample of the clayey silt deposit are shown on Figure B2A and B2B in Appendix B, respectively.

4.4.3 Sand to Gravelly Sand

In Borehole B504-09, a 9.6 m thick deposit of grey sand to gravelly sand was encountered below the clayey silt deposit at Elevation 162.6 m. Cobbles were encountered below a depth of 22.9 m (Elevation 153.4 m) within the deposit immediately overlying the bedrock surface.

The natural water content measured on samples of this deposit range from 13 per cent and 25 per cent

The results of the grain size distribution tests completed on three samples of this deposit are shown on Figure B3A in Appendix B.

4.4.4 Boulders

In Borehole B504-10 a 0.6 m thick layer of boulders was inferred from the resistance to casing advancement at Elevation 155.6 m overlying the bedrock.

4.4.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes B504-01 and B504-09 to B504-11. The depths to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below.



Foundation Element	Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
Pier 1 (South Pier)	B504-01	13.2	162.3	Bedrock Cored
	B504-09	23.3	153.0	Bedrock Cored
	B504-10	21.3	155.0	Bedrock Cored
	B504-11	24.1	152.2	Bedrock Cored

In general, the bedrock surface in the area of the proposed Pier 1 (South) NBL structure slopes downward from south to north and west to east, with the bedrock surface elevation changing by as much as about 10.1 m at the borehole locations.

Based on a review of the bedrock core samples recovered from the boreholes, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh to slightly weathered, foliated or massive, fine to coarse grained, slightly porous, medium strong to very strong, grey and black to dark grey, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered core samples on Figures B4, B6 and B7 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 57 per cent to 100 per cent, indicating a rock mass of fair to excellent quality as per Table 3.10 of CFEM (2006). However, the upper 0.3 m of core recovered from Borehole B504-09 is highly fractured with an RQD value of 0 per cent, indicating that the upper portion of the rock mass is of very poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 90 per cent and 100 per cent and between 0 per cent and 97 per cent, respectively.

Point load strength index tests (ASTM D5731) 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 two samples of the granitic gneiss bedrock core measured Is_{50} values of about 7.1 MPa and 14.4 MPa and the diametral tests carried out on five samples of the granitic gneiss bedrock core measured Is_{50} values ranging from about 7.1 MPa to 23.9 MPa.

Two Unconfined Compression (UC) tests (ASTM D7012) carried out on selected samples of the granitic gneiss bedrock obtained in Borehole B504-09 measured compressive strengths of about 109 MPa and 178 MPa as summarized in Table B2-1 and detailed in Tables B2-4 and B2-5 in Appendix B.

Table B1 also presents estimated Uniaxial Compressive Strengths (UCS) correlated to the PLT strengths based on the relationship between Is_{50} and UCS and applying an average correlation factor (K) of 14 as discussed in Section 4.3.3.

Based on the UCS and PLT test results at the south abutment, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa).

4.5 North Pier (Pier 2)

A total of four boreholes (B504-2 and B504-12 to B504-14) were advanced in the vicinity of the proposed north pier: soil sampling and bedrock coring was carried out in Boreholes B504-2 and B504-12; while



Boreholes 504-13 and B504-14 were advanced to the bedrock surface without soil sampling but cored to obtain additional bedrock information. The soil strata shown on the Record of Boreholes B504-13 and B504-14 are inferred based on the soil information from the adjacent Boreholes B504-02 and B504-12; the inferred soil strata from Boreholes B504-13 and B504-14 are not included in Sections 4.5.2 and 4.5.3. The interpreted stratigraphy at the north pier is shown in profile on Drawing 2 and in cross-section on Drawing 4.

4.5.1 Water

The water surface measured in Key River at the time of drilling Boreholes B504-02 and B504-12 to B504-14 in November 2012 and August 2014 was at Elevations 175.5 m and 176.3 m, respectively, and the depth of water in the boreholes was between 4.1 m and 7.4 m.

4.5.2 Organic Silt and Clayey Silt

A 6.6 m and 7.3 m thick deposit of brown to grey organic silt was encountered from the riverbed in Boreholes B504-02 and B504-12 at Elevations 171.1 m and 170.8 m, respectively. In Boreholes B504-02 and B504-12, the organic deposit is underlain by a 2 m and 1.5 m thick stratum of clayey silt based on examination of the soil samples and the higher undrained shear strengths recorded by the field vanes.

The SPT 'N'-values measured within the organic silt deposit as well as in the clayey silt stratum are 0 blows (i.e., weight of hammer) per 0.3 m of penetration. In situ field vane tests carried out within the organic silt deposit measured undrained shear strengths ranging between 12 kPa and 25 kPa, while those measured on the clayey silt stratum range between about 30 kPa and 34 kPa, and the sensitivity is calculated to range between 1 and 6 and is 3 for the respective strata. The field vane test results indicate that the organic silt deposit has a very soft to soft consistency while that of the clayey silt stratum is a firm consistency.

The natural water content measured on five samples of the organic silt deposit ranges from about 62 per cent to 166 per cent and that of the clayey silt stratum is about 90 per cent on one sample tested. The organic content measured on one sample of the organic silt deposit is 12.2 per cent.

Atterberg limits tests were carried out on four samples of the organic deposit and measured liquid limits ranging from about 41 per cent to 90 per cent, plastic limits ranging from about 28 per cent to 71 per cent and plasticity indices ranging from about 13 per cent to 29 per cent. The results of the Atterberg limits tests are shown on the plasticity chart Figure B1A in Appendix B and together with the organic content indicate the material is classified as an organic silt of intermediate to high plasticity.

The result of a grain size distribution test completed on one sample of the organic silt deposit is shown on Figure B2A in Appendix B.

4.5.3 Gravelly Sandy Silt to Sand

An 11.5 m and 14.3 m deposit of brown to grey gravelly sandy silt to silt and sand to sand was encountered below the clayey silt stratum in Boreholes B504-02 and B504-12 at Elevations 162.5 m and 162.0 m, respectively. In Borehole B504-12, silt pockets were encountered at Elevation 154.7 m, clay lenses were encountered at Elevation 151.6 m and an approximately 1 m thick boulder was encountered at Elevation 150.2 m.



The SPT 'N'-values measured within this deposit range from 4 blows to 59 blows per 0.3 m of penetration indicating a loose to very dense relative density.

The natural water content measured on seven samples of this deposit range from 17 per cent to 24 per cent.

The result of the grain size distribution tests completed on six samples of this deposit are shown on Figure B3A and B3B.

4.5.4 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes B504-02 and B504-12 to B504-14. The depths to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below.

Foundation Element	Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
Pier 2 (North Pier)	B504-02	24.5	151.0	Bedrock Cored
	B504-12	28.6	147.7	Bedrock Cored
	B504-13	29.2	147.1	Bedrock Cored
	B504-14	25.5	150.8	Bedrock Cored

In general, the bedrock surface in the area of the proposed Pier 2 (North) NBL structure slopes downward from north to south and from west to east, with the bedrock surface elevation changing by as much as about 3.9 m at the borehole locations.

Based on a review of the bedrock core samples recovered from the boreholes, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh to slightly weathered, foliated, medium to coarse grained, slightly porous, medium strong to strong, pink, grey and black, as presented in the Record of Drillhole sheets in Appendix B, and shown on the photograph of the recovered core samples on Figures B4, B8 and B9 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 28 per cent to 100 per cent, indicating a rock mass of poor to excellent quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 96 per cent and 100 per cent and between 28 per cent and 100 per cent, respectively.

Point load strength index tests (ASTM D5731) 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 six samples of the granitic gneiss bedrock core measured Is_{50} values ranging from about 7.1 MPa to 11.8 MPa and the diametral tests carried out on six samples of the granitic gneiss bedrock core measured Is_{50} values ranging from about 5.5 MPa to 8.7 MPa.

One Unconfined Compression (UC) tests (ASTM D7012) carried out on a selected sample of the granitic gneiss bedrock obtained in Borehole B504-02 measured a compressive strength of about 94 MPa as summarized in Table B2-1 and detailed in Table B2-2 in Appendix B.



Table B1 also presents estimated Uniaxial Compressive Strengths (UCS) correlated to the PLT strengths based on the relationship between Is_{50} and UCS applying an average correlation factor (K) of 14 as discussed in Section 4.3.3.

Based on the UCS and PLT results at the north pier, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa).

4.6 North Abutment/Approach

A total of six boreholes (B504-15 to B504-20) were advanced in the vicinity of the proposed north abutment/approach. Bedrock coring was carried out in Borehole B504-17. The interpreted stratigraphy at the south pier is shown in profile on Drawing 2 and in cross-section on Drawing 4.

4.6.1 Peat

A 0.1 m thick layer of peat was encountered at ground surface in Borehole B504-17 at Elevation 197.1 m.

4.6.2 Cobble/Sand and Gravel

Below the peat in Borehole B504-17, a 0.1 m thick cobble and 0.1 m thick layer of brown sand and gravel was encountered. The sand and gravel contained silt, trace rootlets and organics.

The natural water content measured on the sample of the sand and gravel is 8 per cent.

4.6.3 Bedrock

The north abutment and approach are located on a bedrock outcrop as observed at the borehole locations and core samples were recovered from Borehole B504-17. The depths to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below.

Foundation Element	Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
North Abutment/Approach	B504-15	0	196.6	Bedrock Exposed
	B504-16	0	199.6	Bedrock Exposed
	B504-17	0.3	196.8	Bedrock Exposed-Cored
	B504-18	0	193.6	Bedrock Exposed
	B504-19	0	195.4	Bedrock Exposed
	B504-20	0	204.4	Bedrock Exposed

In general, the bedrock surface in the area of the proposed north abutment and along the north approach of the NBL structure slopes downward from north to south and west to east, with the bedrock surface elevation varying



by as much as about 6.0 m at the abutment borehole locations and up to 10.8 m relative to the approach borehole about 22 m north of the abutment.

Discontinuities in the rock mass noted in Borehole B504-17 walls recorded by the optical televiewer were predominantly minor open joints (opening width less than 10 mm) or healed joints. In general, no major anomalies were identified along the borehole walls.

Based on a review of the bedrock core samples recovered from Borehole B504-17, the bedrock consists of granitic gneiss. In general the bedrock samples are described as slightly weathered, foliated, coarse grained, slightly porous, medium strong, light grey to grey and pink, as presented in the Record of Drillhole sheet in Appendix B, and shown on the photograph of the recovered core samples on Figure B10 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 61 per cent to 100 per cent, indicating a rock mass of fair to excellent quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are 100 per cent and between 35 per cent and 100 per cent, respectively.

Point load strength index tests (ASTM D5731) 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 two samples of the granitic gneiss bedrock core measured Is_{50} values of about 9.0 MPa and 9.2 MPa and the diametral tests carried out on two samples of the granitic gneiss bedrock core measured Is_{50} values of about 2.5 MPa and 6.3 MPa.

Two Unconfined Compression (UC) tests (ASTM D7012) carried out on selected samples of the granitic gneiss bedrock obtained in Borehole B504-17 measured compressive strengths of about 88 MPa and 102 MPa as summarized in Table B2-1 and detailed in Tables B2-6 and B2-7 in Appendix B.

Table B1 also presents estimated Uniaxial Compressive Strengths (UCS) correlated to the PLT Strengths based on the relationship between Is_{50} and UCS applying an average correlation factor (K) of 14 as discussed in Section 4.3.3.

Based on the UCS and PLT test results at the south abutment, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as medium strong to very strong (R3 to R5, 25 MPa < UCS < 250 MPa).

4.6.4 Groundwater Conditions

The water level in open Borehole B504-17 measured the morning after the day of coring was at a depth of 7.3 m below ground surface, Elevation 189.8, likely representing residual drill water introduced during the coring operation.

5.0 CLOSURE

The drilling program was directed by Lubomir Kosci and Trevor Moxam. This report was prepared by Mr. Matt Thibeault, EIT., and reviewed by Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate 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.

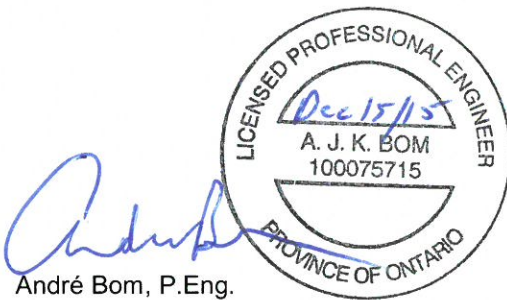


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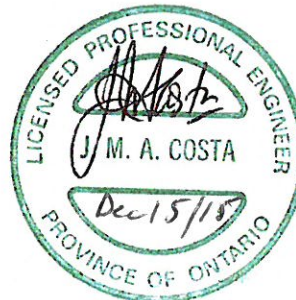
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PART B

FOUNDATION DESIGN REPORT

KEY RIVER NBL BRIDGE, SITE NO. 44-462/1

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 5005-10-00; WP 5147-08-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Key River NBL structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations and approaches. 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 on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the Key River NBL structure within Contract 5 along the proposed section of four-laning of Highway 69.

It is understood that the NBL bridge will be a three-span, variable depth steel girder structure consisting of two end-spans 45 m long and a centre span 75 m long, with abutments located south and north of the Key River and the south and north piers located in the river. The alignment for the proposed Key River Bridge is approximately 550 m east of the existing Highway 69.

Based on the General Arrangement (GA) Drawing provided by URS on April 9, 2015 (dated September 2013), the grade of the proposed bridge deck varies between about Elevation 194.4 m (south abutment) and about Elevation 196.6 m (north abutment). The proposed abutments are to be founded at about Elevation 188 m (south abutment) and about Elevation 190 m (north abutment).

At the south approach, the existing ground surface at the investigated location is at Elevation 195.6 m and at the south abutment area the ground surface varies from about Elevations 192.4 m to 189.7 m. Therefore overburden excavation of up to about 0.6 m below existing ground surface, rock cuts of up to about 0.5 m deep and fills up to about 3.5 m thick will be required in the south approach area. At the south abutment, up to about 4.5 m of bedrock excavation/leveling will be required to construct the abutment foundation, following which up to about 6.5 m of fill placement will be required immediately behind the abutment stem wall.

At the north approach, the existing ground surface at the investigated location is Elevation 204.4 m and at the north abutment area the ground surface varies from about Elevations 199.6 m to 193.6 m. Therefore rock cuts of up to about 8 m below the existing ground surface. At the north abutment, up to about 8.6 m of bedrock excavation/leveling will be required to construct the abutment foundation, following which up to about 6.5 m of fill placement will be required immediately behind the abutment stem wall.

The water level in Key River was at Elevations 175.5 m and 176.3 in November 2012 and July and August 2014, respectively, during the foundation investigation in the river for both the proposed NBL and SBL bridges. Based on the GA drawing provided by URS, the Key River water level was measured by others in April 2008 at Elevation 175.90 m and the high water level is at Elevation 177.23 m.



6.2 Foundation Options

At the abutments, given that both the proposed north and south abutment areas are located on high bedrock outcrops, shallow foundations comprised of spread footings founded directly on the bedrock are preferred for supporting the bridge structure. Pile foundations (H-piles) could be considered as an alternative system for support of the bridge structure at the abutments, however, installation of the piles would require significant excavation/trenching into the strong to very strong bedrock to achieve the minimum required pile lengths for an integral abutment design and this option would be more expensive than the shallow foundation option.

At the piers, shallow foundations are not feasible due to the presence of the deep water and relatively thick, very soft/loose overburden deposits below the river bed that are unsuitable to support spread footings. As such, deep (pile) foundations will be required for support of the piers. A discussion on different pile alternatives for the conditions at this site is provided in Section 6.4.1, however, drilled steel casings have been selected as the preferred foundation alternative due to the steeply sloping bedrock surface and strong to very strong nature of the bedrock. Steel H-piles (driven to refusal or socketted into bedrock), driven steel pipe piles and large diameter caissons are not recommended at this site as discussed in Section 6.4.1.

The following sections provide additional details and recommendations for the design of shallow foundations (spread footings) at the abutments, and deep foundations (drilled steel casings) at the piers, to support the proposed bridge foundation elements.

6.3 Spread Footings (at Abutments)

Shallow foundations comprised of spread footings founded directly on the strong to very strong bedrock are considered the preferred alternative for support of the structure abutments. The following sections outline the recommendations for footing founding options, geotechnical resistances, resistance to lateral loads and requirements for frost protection. A comparison between foundation alternatives at the abutments, including the advantages, disadvantages, relative costs and risks/consequences for each, is presented in Table 1.

6.3.1 Founding Level Alternatives

Based on the GA drawing, the footings for the south and north abutments are proposed to be founded at about Elevations 189 m and 191 m, respectively. The details of the ground surface/bedrock surface elevation and the depth of excavation to the underside of the proposed footings as encountered at the boreholes for the south and north abutments are summarized below.

South Abutment (Underside of Footing at Elev. 189 m)

Borehole Location Within South Abutment	Borehole	Ground/Bedrock Surface Elevation (m)	Depth of Bedrock Excavation Required to Underside of Proposed Footing (m)
West Side	B504-04	191.1	2.1
	B504-05	189.7	0.7
Centre	B504-06	190.8	1.8
East Side	B504-07	192.4	3.4
	B504-08	190.1	1.1

Note: bedrock exposed at ground surface at all borehole locations at south abutment.



North Abutment (Underside of Footing at Elev. 191 m)

Borehole Location Within South Abutment	Borehole	Ground/Bedrock Surface Elevation (m)	Depth of Bedrock Excavation Required to Underside of Proposed Footing (m)
West Side	B504-15	196.6	5.6
	B504-16	199.6	8.6
Centre	B504-17	197.1/196.8	0.3 (overburden) 5.8
East Side	B504-18	193.6	2.6
	B504-19	195.4	4.4

Note: bedrock exposed at ground surface at all borehole locations at north abutment except B504-17 where 0.3 m of overburden present.

Based on the borehole results, bedrock excavation up to about 3.4 m deep will be required to reach the proposed south abutment founding level and up to about 8.6 m of bedrock excavation will be required to reach the north abutment founding level. In general, the bedrock at or immediately below the proposed founding level at the borehole locations is of fair to excellent quality with the RQD generally ranging from about 61 percent to 100 percent. However, the quality of the bedrock may be variable in places and any loose or fractured bedrock encountered at the founding level will need to be sub-excavated and removed prior to footing construction and replaced with mass concrete. Recommendations for excavation of the bedrock are provided in Section 6.10. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

6.3.2 Geotechnical Axial Resistances/Reactions

The following summarizes the factored geotechnical axial resistances at Ultimate Limits States (ULS) for spread footings placed on properly prepared granitic gneiss bedrock or mass concrete (founded on the properly prepared bedrock). For spread footings founded on the properly prepared and inspected bedrock or on mass concrete on bedrock, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS and as such, ULS conditions will govern for this foundation type.

Foundation Element	Founding Alternative for the Proposed Spread Footings	Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement
South and North Abutments	Spread Footing on Granitic Gneiss Bedrock or on Mass Concrete placed directly on Bedrock	10,000 kPa	N/A

The geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load



should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC) and its Commentary.

For footings placed on mass concrete, the factored geotechnical axial resistance at ULS is as given above for bedrock assuming that the compressive strength of the mass concrete is at least 30 MPa.

Following excavation of the thin overburden (where present) and bedrock and prior to placing any concrete, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the footprint of the footings to ensure a proper bond of the mass concrete/concrete footing to the bedrock. Field inspection should be carried out when the excavation is dry and in accordance with OPSS 902 (Excavating and Backfilling). In addition, a check on the sliding resistance between any mass concrete and the bedrock should be carried out (in accordance with the recommendations provided in Section 6.3.3).

6.3.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, for the interface between the mass concrete / concrete footing and bedrock is:

Interface Materials	Coefficient of Friction ($\tan \delta$)
Mass Concrete or Concrete Footing on Bedrock	0.70

The value presented above represents an unfactored value.

The sliding/lateral resistance between the mass concrete/concrete footing and the bedrock may be supplemented by dowelling into the bedrock, if necessary. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. A value of 750 kPa (factored) may be assumed for the grout-to-rock unit bond stress assuming minimum 30 MPa grout strength. This value is based on a factor of 0.4 for static analysis in tension (CHBDC, 2006).

For this site, where the rock mass is essentially as strong or stronger than concrete, the design of the dowels into the bedrock may be considered in the same way as dowels embedded into the concrete. This assumes that the Uniaxial Compressive Strength (UCS) of the grout will be similar to that of the concrete.

The dowels should have a minimum embedded length of 1 m within the fair quality or better bedrock (i.e., rock mass with RQD greater than 50 per cent as per Table 3.10 of CFEM, 2006), and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

If dowelling into bedrock is adopted for resistance to sliding at this site, an NSSP should be included in the Contract Documents to specify the installation, material and testing of the dowels; an example NSSP is included in Appendix E.

6.3.4 Frost Protection

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



6.3.5 Footing Set-Back from Rock Faces

All footings must be maintained an adequate distance away from the edge of the rock slope (i.e., existing bedrock surface) and all rock faces should be adequately cleaned and protected such that the integrity of the rock face/founding rock is maintained. In this regard, the abutment footings should be located a horizontal distance of not less than 2 m from the nearest rock slope surface or crest of new rock cut. If the layout does not allow for this footing set-back, a NSSP should be included in the Contract Documents for vertical rock dowels to be installed between the front of the footing and the crest of the rock face (prior to any new rock excavation, where applicable) in order to provide additional support to the rock face during blasting and following construction; an example is included in Appendix E as referenced in Section 6.3.3.

6.4 Pile foundations (at Piers)

The construction of deep foundations for the in-water piers will be challenging at this site. Some of the challenges associated with foundation design and construction for the piers which affect the selection of the preferred pile alternative include:

- Depth of river water ranging from about 2.5 m to 7.4 m; variable overburden thickness ranging from about 10 m to 25 m; and the presence of weak soils in the upper portion of the subsurface deposits which offer little lateral resistance in the critical zone of influence for lateral pile design (i.e., at least 6 to 8 pile diameters below river bed). More rigorous non-linear, soil-structure interaction modelling to predict foundation response under lateral loads and/or moments is required and piles socketted into bedrock are preferred;
- Highly variable depths to bedrock (i.e., ranging from about 13 m to 29 m below river level; about 11 m to 26 m below the underside of pile caps at Elevation 173.6 m) will result in widely varying pile lengths, especially where battered.
- Sloping bedrock surface across the footprint of the piers/projected footprint of the piles at bedrock surface. Interpolation between the bedrock surface elevations as encountered in the boreholes suggests the following range of approximate bedrock surface slopes:
 - At Pier 1 (south pier) – dipping to the north at up to about 50°; and
 - At Pier 2 (north pier) – dipping to the south at up to about 45°.

The bedrock surface and associated slope may vary between and beyond the borehole locations and may be steeper or flatter in localized areas.

- Proper seating of driven piles (even if fitted with rock points) as well as liners for large diameter conventional caissons would be challenging considering the steeply sloping, very strong bedrock surface and given that layers of cobbles and boulders (up to as much as about 3 m thick) were encountered overlying bedrock at some locations.
- Drilled steel casings installed with ring bits, using rotary duplex and Down-the-Hole (DTH) hammer drilling methods offer the best chance of penetrating the cobbles and boulders layers and achieving proper sealing of the casing and creating bedrock sockets in the steeply sloping, very strong bedrock, provided that careful and controlled drilling practices are followed.



- In general, the smaller the diameter of the drill casing, the easier the constructability on over-water construction and ability to achieve a proper seal in the bedrock. However, the diameter of the pile elements must also be large enough to satisfy the structural loading requirements (in particular lateral loading) on the piers.

Based on the subsurface soil and bedrock conditions at the proposed piers, and considering the above noted challenges, a discussion on the different pile foundation options for this site is provided in the following section.

6.4.1 Pile Options

A number of different types of piles have been considered for support of the pier foundations, including:

- drilled steel casings socketed into bedrock and filled with concrete;
- micropiles socketed into the bedrock;
- steel H-piles or pipe piles driven to refusal on bedrock;
- steel H-piles fixed into bedrock sockets backfilled with concrete;
- caissons (drilled shafts) socketed into bedrock; and
- composite foundation elements comprised of a large diameter drilled shaft enclosing several micropiles socketed into the bedrock.

A brief discussion on the installation details for each of the above, including the applicability to the conditions at this site, is provided in the following sections. A comparison of the pile foundation alternatives noted above, including the advantages, disadvantages, relative costs and risks/consequences for each alternative, is presented in Table 2.

At this site, pile foundations comprised of drilled steel casings socketed into the bedrock are considered the preferred alternative for support of the south and north piers. Further, given the constraints and challenges described previously, in particular the steeply sloping bedrock surface and strong to very strong nature of the bedrock that affects the potential for proper construction of pile foundations, the 0.609 m diameter drilled steel casings alternative is considered the preferred pile type for supporting the pier foundations.

Steel H-piles (driven to refusal or socketed into bedrock), driven steel pipe piles and large diameter caissons are not recommended at this site.

6.4.1.1 Drilled Steel Casings (0.609 m to 0.760 m diameter)

To be installed by rotary duplex drilling using a sacrificial ring bit on the bottom of the permanent steel casing and a DTH hammer to clean out the centre of the pile and also to create a socket within the bedrock below the bottom of the casing.

Information from product suppliers indicates that this type of drilling system allows accurate and straight penetration in steeply sloping bedrock surfaces. In addition, based on discussions with local piling contractors, this type of system has been successfully used to drill rock sockets in very strong and very steeply sloping (60° to 70° and in some extreme cases, up to 80°), granitic bedrock in northern Ontario.



In order to develop sufficient capacity in compression and tension, an uncased rock socket with a Length/Diameter (L/D) ratio of at least 3 is recommended. To achieve the axial capacities provided in Section 6.4.2, the uncased socket should have a minimum length of 2 m into the bedrock. The permanent steel casing must be embedded at least 1 m below the lowest point of contact with the bedrock surface and a minimum of 1 m into fair quality bedrock, but additional casing embedment length may be required to satisfy the lateral loads on the piers, and also to achieve a proper seal in the bedrock prior to socket construction, if the upper bedrock at the pile location is of poor quality. Because the pile will develop its axial capacity based on the shear resistance at the rock socket wall (i.e., between the concrete and bedrock interface) and not rely on end-bearing at the base of the socket, the requirement to properly clean and inspect the base of the socket would be lessened, although a thorough and proper flushing of the side wall of the rock sockets will still be required. A reinforcing bar cage would have to be lowered through the casing and into the rock socket prior to placement of concrete by tremie methods.

6.4.1.2 *Small Diameter Drilled Steel Casings (0.406 m) or Micropiles (0.273 m diameter)*

These piles would be installed in a similar manner to that described above for the larger diameter drilled steel casings using rotary duplex drilling, a DTH hammer and a sacrificial ring bit on the bottom of a permanent steel casing. An advantage of using the smaller diameter drilled steel casings (or micropiles) is that, in general, the smaller the diameter the pile element, the easier to drill, seal and socket into steeply sloping bedrock.

The requirements for minimum casing embedment into bedrock, minimum length of rock socket and cleaning of rock socket are as described in Section 6.4.2.1. The differences are mainly in the type of equipment used for the installation in that relatively smaller equipment (perhaps more amenable to construction on floating platforms over water) could be employed given the smaller pile sizes. In addition, it is anticipated that seating the drill casings into the very strong and sloping bedrock would be easier with the smaller diameter pile elements. The smaller diameter pile section would require the use of a neat cement grout rather than concrete for backfilling and the central reinforcement would be in the form of a bundle of two or more Dywidag bars (instead of a re-bar cage) installed through the casing and into the rock socket prior to grouting by tremie methods.

A disadvantage of using the smaller diameter pile elements is that, individually, they provide relatively less lateral resistance. Given the high lateral design loads on the piers, a large number of small pile elements would likely be required. Alternatively, consideration would have to be given to installing the small rock socketed pile elements in groups of two or three within a larger diameter permanent outer steel casing that would be initially installed through the upper very soft to soft organic silt/silty clay and founded into the compact silt and sand stratum at depth. The outer casing, backfilled with concrete and the embedded small pile elements, could offer a much larger lateral resistance in the upper portions of the weak overburden.

6.4.1.3 *HP 310x110 Steel H-Piles Driven to Refusal on Bedrock*

The alternative of supporting the pier foundations on steel H-piles driven to refusal on bedrock is not recommended at this site. Although information from product suppliers suggests that special rock points (such as injector-type or Oslo-type) can be used where the bedrock surface is dipping up to 50°, based on discussions with local piling contractors, it is our understanding that proper seating of driven steel piles onto very strong granitic bedrock sloping at about 45° (or greater) can be problematic, especially where the overburden soils are weak and where battered piles are employed. The presence of cobbles and boulders over the bedrock can further complicate the installation process. In these types of conditions (which exist at this site), there is a high



risk of the piles deflecting off the side(s) of the sloping bedrock, potentially resulting in improper seating, in damage to the piles and/or much longer than anticipated pile elements.

6.4.1.4 HP 310x110 Steel H-Piles Socketted into Bedrock

To avoid problems with the seating of driven steel piles onto sloping bedrock, consideration could be given to placing steel H-piles into 0.609 m diameter sockets drilled into the bedrock (a minimum of 1.5 m deep) by rotary duplex drilling methods and backfilled with concrete. The rock sockets would be constructed using a temporary casing with ring bit and a DTH hammer to clean out the centre of the pile and also to create the socket within the bedrock. Upon completion of drilling of the rock socket, the H-pile would be lowered through the temporary casing and rest on the bedrock and the socket would be backfilled with tremie concrete prior to removing the casing. A disadvantage with this method of installation is the disturbance (softening and loosening) that will be created within the overburden upon removal of the temporary 0.609 m diameter drill casing. This disturbance will reduce the already low lateral resistance available within the overburden and would make the design for the high lateral loads on the piers, difficult. Given this, the use of steel H-piles socketted into bedrock is not recommended at this site.

6.4.1.5 0.9 m Concrete Caissons (Drilled Shafts)

Caissons (drilled shafts) would be advanced into bedrock using permanent casings and conventional large caisson drilling equipment. It is anticipated that difficulties would arise when attempting to seal the large diameter casings into the very strong, sloping and fractured bedrock at some locations and, in general, the larger the caisson diameter the greater the difficulties in sealing the caisson and drilling the rock socket. Caissons with a diameter larger than 0.9 m would not be practical at this site due to the constructability issues associated with the sloping bedrock. The presence of cobbles and boulders overlying the bedrock will make advancing the large diameter casings and sealing them into bedrock more difficult. If a proper seal cannot be formed, there will be difficulties forming the rock socket below the casing. Given the risks associated with drilling the larger diameter caisson hole and rock socket, the use of caissons is not recommended at this site.

6.4.1.6 Composite Foundation Element (Drilled Shaft and Micropiles)

Composite foundation elements, comprised of a large diameter drilled shaft (or caisson) enclosing a number of smaller diameter, equally spaced micropiles, each consisting of a permanent steel casing and central reinforcing bar advanced into bedrock from within the drilled shaft, could be considered for support of the pier foundations at this site. The large diameter drilled shafts would have the advantage of providing increased lateral resistance in the upper very soft organic silt stratum, while the small diameter micropiles would have the advantage of ease of socketing into the very strong and sloping bedrock. A detailed soil-structure interaction analysis would be required to optimize the relative lengths of the upper drilled shaft and lower micropiles, however, at a minimum, the permanent casing of the upper, large diameter drilled shaft would likely be required to extend at least 5 m into the silty sand stratum overlying the bedrock. Detailed analysis would also be required to determine the minimum number of micropiles and cross-section composition (stiffness) required to transfer the high axial loads to the bedrock.

It is noted however, that the lateral loads on the pier elements at this site are significant due to such factors as the design ice loading conditions, the depth of water (free pile length) and the height of pier columns. In



addition, the very soft and organic soils that comprise the upper portion of the overburden immediately below the river bed offer very low lateral resistance for the design of pile foundations. This combination of factors is particularly critical at the north pier of the Southbound Lanes (SBL) where the water is deepest, the very soft organic layer is thick and the underlying overburden is relatively thin and loose. At this location, the lateral forces from the pile cap are expected to transfer to a great depth along the pile and would likely create high bending moments within the micropiles where they connect the bottom of the drilled shaft to the bedrock. Since micropiles are small diameter elements, designing the micropile(s) to transfer high bending moments would be challenging and could be a limitation of this type of composite foundation element.

It should also be noted that construction of large diameter drilled shafts (even as composite foundation elements) would require the use of large equipment. Since the piers for this bridge will be constructed over water, the use of large equipment would pose additional constructability challenges.

Based on the above considerations from a foundations perspective, and after discussions with the structural engineer, this type of foundation is not considered to be the preferred alternative at this site.

6.4.2 Drilled Steel Casing Piles

The thickness of the overburden at the piers ranges from about 10 m to 25 m and is generally comprised of an upper deposit of very soft to soft organic silt and/or clayey silt, underlain by predominantly loose to compact silt and sand. As such, the contribution of the overburden soils to the axial capacity of the piles will be negligible and the drilled steel casing pile foundations will have to be advanced to and socketed into the bedrock.

Based on the information at the boreholes, the details of the river bed elevation, bedrock surface elevation, thickness of overburden, depth to bedrock below the underside of the pile cap and minimum recommended casing embedment lengths and rock sockets lengths for the drilled steel casing pile foundations are summarized below. An interpretation of the approximate bedrock surface contours based on the limited borehole information available for the site and interpolation between the various boreholes in the areas of the piers/abutments and waterline at exposed bedrock outcrops has been carried out and is presented on Figure 3. This information has been used to estimate the range of battered pile lengths below the proposed underside of pile cap (at Elevation 173.6 m as provided by URS), and is included in the summary below.



FOUNDATION REPORT – KEY RIVER NBL BRIDGE – HIGHWAY 69
GWP 5005-10-00; WP 5147-08-01

South Pier 1 (Underside of Pile Cap at Elev. 173.6 m)

Borehole and Location At South Pier	River Bed Elevation (m)	Approx. Bedrock Surface Elevation (m)	Thickness of Overburden (m)	Depth to Bedrock Below Underside of Pile Cap (m)	Minimum Casing Embedment Length below Top of Bedrock ¹ (m)	Minimum Uncased Rock Socket Length ² (m)	Estimated Length of Battered Pile Below U/S Pile Cap ³ (m)
West Side (at Hwy 69 CL): B504-10	172.6	155.0	17.6	18.6	1.0	2.0	21 to 26
Centre – Towards Shore: B504-01	173.1	162.3	10.8	11.3	1.0	2.0	17 to 18
Centre – Towards Centre of River: B504-09	172.5	153.0	19.5	20.6	1.0	2.0	23 to 24
East Side: B504-11	172.8	152.2	20.6	21.4	1.0	2.0	17 to 23

Note: 1. Bedrock is sloping. Minimum casing embedment into bedrock to be determined relative to lowest elevation/point of contact of casing onto bedrock surface. Additional casing embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.5.
2. Minimum uncased socket embedment length of pile into bedrock to satisfy the axial geotechnical resistance presented in Section 6.4.2. Additional embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.5.
3. Based on the estimated bedrock surface elevations from the contours shown on Figure 3. Bedrock surface is variable and the actual pile lengths will vary and will be determined during pile installation.

North Pier 2 (Underside of Pile Cap at Elev. 173.6 m)

Borehole and Location At North Pier	River Bed Elevation (m)	Approx. Bedrock Surface Elevation (m)	Thickness of Overburden (m)	Depth to Bedrock Below Underside of Pile Cap (m)	Minimum Casing Embedment Length below Top of Bedrock ¹ (m)	Minimum Uncased Rock Socket Length ² (m)	Estimated Length of Battered Pile Below U/S Pile Cap ³ (m)
West Side (at Hwy 69 CL): B504-14	168.9	150.8	18.1	22.8	1.0	2.0	26 to 32
Centre – Towards Shore: B504-02	171.1	151.0	20.1	22.6	1.0	2.0	25 to 26
Centre – Towards Centre of River: B504-12	170.8	147.7	23.1	25.9	1.0	2.0	28 to 29
East Side: B504-13	172.2	147.1	25.1	26.5	1.0	2.0	24 to 28

Note: 1. Bedrock is sloping. Minimum casing embedment into bedrock to be determined relative to lowest elevation/point of contact of casing onto bedrock surface. Additional casing embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.5.
2. Minimum uncased socket embedment length of pile into bedrock to provide the axial geotechnical resistance presented in Section 6.4.2. Additional embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.5.
3. Based on the estimated bedrock surface elevations from the contours shown on Figure 3. Bedrock surface is variable and the actual pile lengths will vary and will be determined during pile installation.



Due to the sloping surface of and varying depths to the bedrock at this site, the actual pile lengths at the pier foundation units will vary and the estimated pile lengths indicated above should be considered approximate only. The Contract should allow for the supply/installation of varying pile lengths.

It is noted that the minimum casing embedment length below the top of bedrock has been selected considering the variation in the RQD of the bedrock at the boreholes. The RQD of the bedrock at the site is variable and deeper casing embedment into the rock may be required at some locations in order to achieve a proper seal prior to constructing the uncased rock socket. In addition, it is noted that the bedrock surface at the site is steeply sloping in some areas and the minimum casing embedment length into bedrock should be determined relative to lowest elevation/point of contact of casing onto the bedrock surface.

A preliminary assessment of the possible range in contact angles between the drilled steel casing and the bedrock surface has been carried out based on the proposed pier locations, pile layout and pile batters (as provided by URS) and the bedrock surface elevation contours (as shown on Figure 3). Based on this information, it is estimated that the piles will come in contact with the bedrock surface at angles ranging from as steep as approximately 85° (i.e., near perpendicular contact) to as shallow as approximately 40° .

6.4.3 Geotechnical Axial Resistances / Reactions

As noted in Section 6.2, several deep (pile) foundation options have been considered for support of the piers. However, after considering the constructability of the different pile types onto and into the steeply sloping and strong to very strong bedrock, the following options are considered the most suitable for the site:

- drilled steel casings (0.609 m or 0.760 m diameter) with casings embedded a minimum of 1 m into fair quality (i.e., rock mass with RQD >50 per cent as per Table 3.10 of CFEM, 2006) bedrock and a minimum 2 m uncased rock socket below the bottom of the casing, with steel reinforcement and filled with 30 MPa concrete; and
- small diameter drilled steel casings (0.406 m diameter) or micropiles (0.273 m diameter) with casings embedded a minimum of 1 m into fair quality (CFEM 2006) bedrock and a minimum 2 m uncased rock socket below the bottom of the casing, with central reinforcement and filled with 30 MPa grout; installed in groups of two or three within a larger diameter upper steel casing to provide increased lateral resistance through the zones of soft soils below the river bed.

Pile foundations for the piers should be designed based only on side wall resistance within the rock socket along the concrete (or grout) and bedrock interface. Any contribution from end-bearing resistance with the rock socket should be ignored given the difficulties that will be involved with a proper inspection of the socket base given that the piles are long (15 m to 30 m), battered and will be filled with water.

For pile foundations supported in the fresh to slightly weathered granitic gneiss bedrock, the strength of the concrete will be less than the bedrock strength and as such the concrete strength will govern. A factored side wall resistance at ULS of 1.3 MPa may be assumed for design assuming a minimum concrete strength of 30 MPa. The casing for the drilled piles should extend a minimum of 1.0 m below the lowest elevation/point of contact of casing onto the bedrock surface.

The following summarizes the factored axial geotechnical resistance and reaction for the different foundation options at the pier locations.



Pile Foundation Alternative	Factored Axial Geotechnical Resistance at ULS¹ (kN) (Compression)	Axial Geotechnical Reaction at SLS² (kN) (Compression)	Factored Axial Geotechnical Resistance at ULS (kN) (Tension)
0.609 m diameter Drilled Steel Casings	4,500	N/A	3,000
0.760 m diameter Drilled Steel Casings	6,000	N/A	4,000
0.406 m diameter Drilled Steel Casings	3,000	N/A	2,000
0.273 m diameter Micropiles	2,000	N/A	1,500

Note: ¹ Uncased rock socket length = 2 m (minimum). Structural capacity of pile must be checked.

² The SLS reaction for 25 mm of settlement is greater than the ULS resistance and therefore ULS governs.

For all options, the recommended embedment lengths into bedrock are the minimum required to satisfy the axial loads provided above. Additional embedment length into bedrock may be required to satisfy lateral loads on the pier(s) and is to be determined by the structural engineer (refer to Section 6.4.5 below).

6.4.4 Downdrag Load (Negative Skin Friction)

Soft organic silt/clayey silt strata up to about 11 m thick was encountered in the boreholes advanced at the pier foundation units. However, given that no filling is proposed to be carried out within the river in the vicinity of the piers, consolidation and settlement of the clayey silt stratum at the piers is not anticipated and as such, no downdrag loads are expected on the pile foundations.

6.4.5 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles, the relative rigidity of the pile to the surrounding soil and bedrock, the fixity condition at the head of the pile (pile cap level) as well as at the base of the pile, the structural capacity of the pile to withstand bending moments, the soil and/or bedrock 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 mobilisation of the full lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

The resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m). However, the response of a pile/caisson to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum caisson/pile deflections are less than 1 percent of the caisson diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006).

Considering the high lateral design loads on the piers and the unique design conditions at the site (i.e., high pier columns, long free-length or unsupported length of pile through deep water in the river, and very soft organic soils below the river bed to significant depth) it is recommended that a more rigorous soil-structure interaction analysis, employing P-y curves that better represent the non-linear lateral soil behaviour, be carried out.



It is our understanding that URS will carry out such an analysis to design the layout of the pile groups at the piers employing a commercially available software package, such as FB-MultiPier (by BSI). The modelling requirements have been discussed with URS, and input has been provided on the selection of the soil and rock models for the different overburden layers and bedrock conditions. A summary of the recommended models for the lateral group pile analysis along with the key soil parameters for each is provided in Tables 3 and 4.

In the boreholes advanced in the vicinity of the piers, the bedrock generally slopes downwards towards the centre of the river and as such the proposed piles that are battered towards the centre of the river will have a greater length and will provide relatively lower lateral resistance compared to the piles that are battered towards the shore. Further, we understand from URS that the piles in the pier group that are oriented perpendicular to the centreline of the pier (and battered in a north-south direction at 1H:10V) will experience relatively higher lateral loads than those piles on the outside of the pier group and battered in an east-west direction at 1H:8V.

At the piers, where drilled steel casings are the preferred foundation alternative, for a single 0.609 m diameter drilled steel casing with a 12 mm wall thickness advanced to the design depths provided in Section 6.4.2 and battered at 1H:10V towards the centre of the river (i.e., perpendicular to the pier centreline), the estimated factored lateral resistance at ULS and the lateral reaction at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 7.05), developed by Ensoft Inc.

Foundation Location	Factored Geotechnical Lateral Resistance at Ultimate Limit States (ULS) (kN)	Geotechnical Lateral Reaction at Serviceability Limit States (SLS) for 10 mm of Deflection (kN)
South Pier (Pier 1)	150	65
North Pier (Pier 2)	130	45

The lateral resistances given above are based on an assumed pile embankment length into bedrock of 3 m, an assumed fixed-head pile condition, and an unfactored axial load of 1,700 kN applied to the top of pile. No bending moment was applied to the top of the pile. The lateral resistances should be reviewed if greater vertical loads or a different loading condition is anticipated as additional embedment length into bedrock may be required to satisfy the lateral loads on the pier, which is to be determined by structural engineer.

6.4.6 Group Effects

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, or the lateral reaction defined by the P-y curve(s) (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:



Pile Spacing in Direction of Loading d = Pile Diameter	Horizontal Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction, or the lateral reaction defined by the P-y curve(s) (NAVFAC, 1982) by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading d = Pile Diameter	Horizontal Subgrade Reaction Reduction Factor, R
4 d	1.00
1 d	0.50

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

6.4.7 Frost Protection

The underside of the 2 m thick pile caps at the piers is proposed to be at Elevation 173.6 m (according to the information provided by URS), which is about 2.3 m below the water level measured by others in April 2008. The proposed elevation of the underside of the pile cap is considered sufficient, from a frost penetration perspective, provided that ice does not extend below Elevation 173.6 m. If it is possible that the river ice could extend below Elevation 173.6 m, the proposed underside of the pile cap should be lowered.

6.5 Seismic Site Coefficient

6.5.1 Site Coefficient

For seismic design purposes, given that the bedrock is exposed at the abutments, the Site Coefficient, S , may be taken as 1.0 at the abutments considering the guidelines in Section 4.4.6 of the CHBDC (2006), consistent with Soil Profile Type I. At the piers, given the thickness and consistency/relative density of the overburden soils, the Site Coefficient, S , may be taken as 1.5 consistent with Soil Profile III.

6.5.2 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 the Sudbury and Parry Sound



area is 0.051 (for a probability of exceedance of 10 per cent in 50 years). According to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A_s , is 0.05.

Given this assessment and the fact that the proposed bridge structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.1, Table 4.2 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill 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.

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 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. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill in accordance with OPSD 3102.100 (Walls, Abutment, Backfill Drain) and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain). Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- 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*). The following parameters (unfactored) may be used for rock backfill:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Rock Fill	19 kN/m ³	0.36	0.22

- 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 walls, granular fill should be placed in a zone with the 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 walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to



1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

Fill Type	Soil Unit Weight (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27
Rock Fill	19	0.36	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 abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the CHBDC.

6.7 Approach Design

Based on the GA drawing and cross-sections provided by URS, the proposed road grade at the new south and north approaches will be at about Elevations 194.4 m and 196.6 m, respectively.

At the south approach, the existing ground surface at the investigated location (about 20 m south of the abutment) is at Elevation 195.6 m and at the south abutment area the ground surface varies from about Elevations 192.4 m to 189.7 m. Based on the cross-sections provided by URS, in the southern portion of the south approach, overburden excavation of up to about 0.6 m below existing ground surface and rock cuts of up to about 0.5 m deep will be required. Closer to the abutment, fills up to about 3.5 m thick will be required. However, following excavation of the bedrock to construct the south abutment foundation, up to about 6.5 m of fill placement (to the level of the pavement surface) will be required immediately behind the south abutment stem wall, as well as within and immediately adjacent to the abutment wingwalls.

At the north approach, the existing ground surface at the investigated location (about 20 m north of the abutment) is at Elevation 204.4 m and at the north abutment area the ground surface varies from about Elevations 199.6 m to 193.6 m. Based on the cross-sections provided by URS, rock cuts of up to about 8 m deep will be required. Closer to the abutment, the depth of rock cut will be significantly less, but no fill placement or embankment construction is anticipated to be required other than that needed for construction of the pavement structure. However, following excavation of the bedrock to construct the north abutment foundation, up to about 8.6 m of fill placement (to the level of the pavement surface) will be required immediately behind the north abutment stem wall, as well as within and immediately adjacent to the abutment wingwalls.

For the amount of fill required in the approach and abutment areas, no settlement or stability issues are anticipated so long as the following conditions are satisfied:

- all surficial topsoil/organic layers are removed prior to fill placement (note that fill will be required outside the wing walls and potentially in areas where bedrock excavation is not required);



- all fill is placed on the properly prepared bedrock surface;
- only a crushed, angular granular material (Granular B Type II or 300 mm minus rock fill as per Northeastern Region Directive (2002) for backfill of structures) is utilized for fill construction;
- the fill is placed and compacted in thin lifts; and
- the fill is constructed at an inclination of 2H:1V for the side slopes (where required) and no steeper than 1.5H:1V for the front slopes immediately adjacent to the wing walls.

Additional details on the above recommendations are provided in Section 6.8.

6.8 Subgrade Preparation and Approach Construction

The following sections provide recommendations for subgrade preparation and fill placement in the approach areas behind the north and south abutments. It is noted that the majority of the approach areas will be constructed in rock cut and so the extent of fill placement will be limited.

6.8.1 Removal of Organic Materials

Bedrock is exposed at ground surface in much of the area surrounding the abutments and within the approach limits. However, thin surficial layers of topsoil and/or organics (peat) about 0.1 m thick were present at some locations, underlain by thin layers (up to about 0.5 m thick) of silty sand and gravel. Prior to the placement of any fill, all surface and near surface layers of topsoil/organics should be stripped from the plan limits of the proposed works.

6.8.2 Embankment Fill Placement

Based on the limited fill requirements anticipated in the approaches (less than about 3.5 m thick and confined to relatively small areas), it is recommended that a crushed, angular granular material (i.e., Granular B Type II or 300 mm minus rock fill as per Northeastern Region Directive (2002) for backfill of structures) be used for fill construction. The placement of the granular fill should be carried out in accordance with the requirements as outlined in the OPSS 206 (Grading). Side slopes for granular fill placement (where required) should be no steeper than 2H:1V and the front slopes immediately adjacent to the wing walls should be no steeper than 1.5H:1V, as noted in section 6.7.

6.9 Construction Considerations

6.9.1 Overburden Excavation

In order to construct the bridge abutment foundations on the bedrock at the currently proposed footing elevations, minimal excavation of the organics and thin overburden soil above the bedrock (i.e., less than about 0.3 m) will be required. The overburden soils at the site are considered Type 3 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). Excavations in the overburden soils should be carried out with side slopes no steeper than 1H:1V. The requirements and recommendations for excavation within the bedrock are discussed in Section 6.10.



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

6.9.2 Control of Groundwater and Surface Water

At the abutments, groundwater control is not anticipated to be required. Surface water should be directed away from the excavations at all times.

6.9.3 Cofferdam Construction

Construction of the pile caps for the in-water piers will require some form of cofferdam. Conventional cofferdam construction (i.e., the use of interlocking sheet piles driven through the overburden to form a water tight box structure) will be difficult at this site because of the following challenges at the pier locations:

- Depth of water (up to about 7.5 m deep) plus thickness of upper weak, very soft overburden (up to about 10 m thick) will require cofferdams greater than 15 m deep in order to achieve sufficient lateral fixity in the more competent compact sands at depth. This will result in high pressures acting on the sides of the sheet piles.
- The presence of cobbles encountered in the overburden at some locations may be sufficient obstructions to impede the installation of the sheet piles.
- All excavation and pier foundation pile installations would have to be carried out in-the-wet until a sufficiently thick/heavy tremie-plug is constructed at the base of the cofferdams otherwise there is a high risk that base heavy failure will occur during unwatering.
- The upper weak, very soft and compressible overburden (up to about 10 m thick) will likely compress and consolidate under the weight of a heavy concrete tremie plug (during curing). This could lead to complications in maintaining an adequate water-tight seal within the cofferdam. It could also result in drag loads forming on the pier piles.
- Depth of water (up to about 7.5 m deep) will also result in the requirement for thick pile caps, resulting in a high dead load to be supported temporarily by the weak overburden (during curing) and by the pier piles (after curing).

Given the above, the use of conventional cofferdams for pier construction at this site would likely carry high costs and as well as high risks to a successful completion.

As such, it is recommended that consideration be given to using prefabricated cofferdam(s), constructed with pre-drilled holes and steel tube sleeves through the bases large enough to accommodate the foundation pile elements. These types of cofferdams could be floated and then anchored into place, act as a template during pile installation, and upon completion of piling could be backfilled with concrete to form the pile cap(s).

It is noted however that the depth of the river water is relatively shallow at the south pier (about 2.4 m deep at Borehole B504-01) and the surface of the river bed is relatively high (at about Elevation 173.1 m). If the underside of the pile cap is at Elevation 173.6 m, then there will only be about 0.5 m of draft (free water) below the pile cap. This will need to be taken into consideration in the design of the prefabricated, floating cofferdams.



6.9.4 Obstructions

The presence of cobbles was inferred from drilling resistance near the clayey silt/sand to gravelly sand interface at one location (Borehole B504-11). In addition, as cobbles and boulders were encountered at depth within the sand/gravelly sand/ silt and sand strata above the bedrock surface within several boreholes advanced in the vicinity of the piers it is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions; an example NSSP is included in Appendix E.

6.10 Assessment of Rock Cut Stability

6.10.1 Structural Mapping

Structural mapping of the exposed bedrock outcrops was carried out in the areas of the proposed abutments and approaches for the Key River NBL and SBL structures. This data was combined with the discontinuity measurements obtained from the optical televiwer surveys of Boreholes B503-06 and B504-6 (south outcrop wall) and B503-16 and B504-17 (north outcrop wall) to produce a set of stereographic projections. A total of 76 discontinuities were mapped in the area of the south abutments and approaches and these data were combined with the 36 discontinuities obtained from televiwer surveys of Boreholes B503-06 and B504-06. A total of 48 discontinuities were mapped in the area of the north abutments and approaches and these data were combined with the 19 discontinuities obtained from televiwer surveys of Boreholes B503-16 and B504-17.

The stereographic projection data from the bedrock outcrops in the area of the south abutments and the north abutments is shown on Figure D9. Based on the stereoplots there are two major orthogonal joints sets with several minor sets. Both major joint sets are steeply dipping with one set striking northeast-southwest and the other set striking northwest-southeast. In addition to the major joint sets, there is a minor shallow dipping set that typically strikes east-west but is somewhat variable and some minor inclined joints with variable dip directions. The joints are generally tight to slightly open, rough and planar to wavy or curved with little to no infilling. Given that the joints are generally tight and rough a conservative friction angle of 35 degrees has been used in the kinematic stability analysis.

6.10.2 Stability Assessment and Rock Bolting

Failures in the exposed rock cuts in the approaches and at the abutment locations will be structurally controlled, kinematic type failures, rather than larger scale failures through intact rock. These types of failures occur as the result of movement along pre-existing geological discontinuities (i.e., joint or fault planes). The three basic mechanisms of structurally controlled failures in rock cuts are planar failures, wedge failures and toppling failures.

The potential for these types of failures to occur at this site has been assessed based on a kinematic structural analysis of the data collected from the structural mapping as described in Section 6.10.1 and the results of the wedge, planar and toppling failure modes are presented on Figures D10 and D11 for the east and west side of the south approaches, respectively, and on Figures D12 and D13 for the east and west side of the north approaches, respectively.



Approach Cuts

The following summarizes the kinematic stability of the rock cuts at the approaches to the structure:

- South Approach:
 - East Side: moderate chance of wedge and a slight chance of planar failures and toppling failures; and
 - West Side: relatively high chance of wedge failures, a moderate chance of planar failures and a slight chance of toppling failures.
- North Approach:
 - East Side: moderate chance of planar and wedge failures and a slight chance of toppling failures; and
 - West Side: moderate chance of wedge and a slight chance of planar failures and toppling failures.

Given the steep nature of the planar type failures it is likely that most unstable planes would be dislodged during blasting and subsequent scaling. Many of the steeper wedges will also likely be dislodged during blasting and/or scaling; however, some of the larger wedges may require spot bolting after excavation of the cuts. Stability of the ultimate rock cut faces will therefore be mostly affected by the quality of the controlled blasting.

Abutment Front Slopes

The different types of potential failures that are anticipated for the rock slopes in front of and below the abutments has also been assessed based on a kinematic analysis of the structural data collected from the structural mapping (as described in Section 6.10.1) and the results of the wedge, planar and toppling failure modes are presented on Figures D14 and D15 for the south and north abutments of the NBL bridge, respectively.

The following summarizes the kinematic stability of the rock slopes in front of and below the abutments:

- North Abutment: low chance of wedge failures and a slight chance of toppling; and
- South Abutment: slight chance of toppling.

Rock Cuts at Abutment Footings and Wildlife Crossing

The potential failure mechanisms that are anticipated for the rock cuts required for construction of the abutment footings and the adjacent wildlife crossing have also been assessed based on a kinematic analysis of the structural data (as described in Section 6.10.1) and the results of the wedge, planar and toppling failure modes are presented on Figures D16 to D17 for the south and north abutments of the NBL bridge, respectively.

The following summarizes the kinematic stability of rock cuts at the abutments:

- North Abutment: moderate to high chance of wedge failures; low chance of planar failures; slight chance of toppling failures; and
- South Abutment: slight chance of wedge failures; low chance of planar failures; slight chance of toppling failures.



All rock cuts should be inspected by the Quality Verification Engineer (QVE) upon completion of blasting and scaling to identify potential instabilities and to determine requirements for rock bolting. Rock bolting should be carried out in accordance with OPSS 203. It is recommended that a provision for rock bolts be included in the Contract documents.

6.10.3 Rock Hazards at Rock Cuts

The proposed 4 m to 5 m wide catchment areas adjacent to the up to 8 m high rock cuts are considered sufficient in that the potential for rockfalls to reach the roadway is considered low.

6.10.4 Structure Foundations

As discussed in Section 6.3.5, the footings for the abutment foundations must be located a horizontal distance not less than 2 m back from the nearest edge/crest of the new rock cut slope surface. If the layout does not allow for this footing set-back, a NSSP should be included in the Contract Documents for vertical rock dowels to be installed between the front of the footing and the crest of the rock face prior to any new rock excavation to provide additional support to the rock face during blasting and following construction; an example is included in Appendix E.

All excavations for footing construction in the abutment areas should be inspected by a geotechnical engineer prior to placing concrete to ensure that the base has been adequately cleaned and that the bedrock conditions exposed at the founding level are consistent with the design assumptions. Where possible, the excavations for the footing foundations should be made to provide a flat (horizontal) bearing surface. Rock protrusions or cavities should be avoided such that a uniform bearing pressure surface is provided across the full area of the footing. All loose, shattered or weathered rock within the footprint of the footings and at the footing level should be removed and replaced with concrete and the foundation base should be cleaned of deleterious material using high pressure air and water.

6.11 Recommendations for Rock Excavations and Blasting

6.11.1 Rock Excavation

It should be noted that the bedrock at the south and north abutments (Boreholes B504-06 and B504-17) is generally classified as strong (R4) to very strong (R5). The three (3) Unconfined Compression (UC) tests carried out on bedrock core samples recovered at these locations measured Uniaxial Compressive Strength (UCS) between 88 MPa and 102 MPa. Based on the rock quality and strength, blasting will be required for rock excavations at the north and south abutments and approaches.

For the height of rock cuts required at this site, it is recommended that the overall slope of the rock cut faces be formed vertical.

All rock excavations should be carried out in accordance with OPSS.PROV 206 (*Grading*) by wall control blasting techniques such as line drilling and pre-shearing to minimize blast damage to the rock (i.e., shattering and over-break) and provide better control over the configuration of the founding surface.



6.11.2 Blasting

The use of explosives should follow the specifications outlined in OPSS.PROV 120 (Use of Explosives). It is recommended that control of all blasting operations, including removal of all loose, unstable rock from the cut faces, be carried out in accordance with OPSS.PROV 206 (Grading).

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.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P. Eng., a senior geotechnical engineer and Associate with Golder, and the technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., a senior geotechnical engineer and Principal of Golder. Mr. Jorge M. A. Costa, P. Eng., the Designated MTO Contact for this project and Principal of Golder, conducted an independent quality control review of the report.



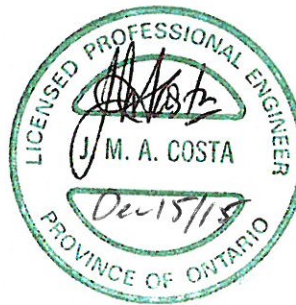
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REFERENCES

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd., British Columbia.

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

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

Geology of Ontario. 1991. Ontario Geological Society, 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.

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.

Unified Facilities Criteria. U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

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 D7102	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Commercial Software:

LPILE (Version 7.05) by Ensoft Inc.

Ministry of Transportation Ontario:

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 3101.150	Walls – Abutment, Backfill, Minimum Granular Requirement
OPSD 3101.200	Walls – Abutment, Backfill, Rock
OPSD 3102.100	Walls – Abutment, Backfill, Drain
OPSD 3190.100	Walls – Retaining and Abutment, Wall Drain
OPSD 3121.150	Walls – Retaining, Backfill, Minimum Granular Requirement



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GWP 5005-10-00; WP 5147-08-01

Ontario Provincial Standard Specification:

OPSS.PROV 120	General Specification for Use of Explosives
OPSS.PROV 206	Construction Specification for the Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 904	Construction Specification for Concrete Structures
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1010	Construction Specification for Aggregates - Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)
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Table 1: Evaluation of Foundation Alternatives – Abutments

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread Footings on Bedrock (semi-integral and conventional abutments)	1	<ul style="list-style-type: none">■ Relative ease of construction.■ Reduced bedrock excavation (as compared with pile option).■ Negligible post-construction settlement.	<ul style="list-style-type: none">■ Excavation in strong to very strong bedrock will be required to achieve a level bearing surface.■ Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break.■ Fully integral abutment design not achievable.	<ul style="list-style-type: none">■ Lower relative cost than piled foundation option.■ Additional costs for vertical dowels, if required to improve lateral resistance.■ Additional costs for dowling pre-support of rock face if footing is not set-back a sufficient distance from edge of rock face/rock cut.	<ul style="list-style-type: none">■ Controlled blasting techniques must be used to maintain integrity of rock below the footings or repair using mass concrete may be required during construction in areas of overbreak/overshatter.
Steel H-piles in Bedrock Trenches (integral abutments)	2	<ul style="list-style-type: none">■ Negligible post-construction settlement.■ Fully integral abutment design achievable.	<ul style="list-style-type: none">■ Excavation in strong to very strong bedrock will be required to form trench to achieve minimum required pile lengths.	<ul style="list-style-type: none">■ Higher relative cost than spread footings due to additional costs for excavating trenches in bedrock.	<ul style="list-style-type: none">■ Not recommended due to shallow depth to bedrock and the additional depth of excavation required in strong to very strong bedrock to accommodate minimum pile lengths.

Prepared By: AB
Checked By: JPD
Reviewed By: JMAC



Table 2: Evaluation of Foundation Alternatives – Piers

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Drilled Steel Casings (0.609 m to 0.760 m Ø) socketed into bedrock using DTH hammer drilling	1	<ul style="list-style-type: none">Very high axial capacity.Higher lateral capacity than small diameter pile elements.Smaller number of pile elements required per pier.DTH hammer drilling method offers best chance of seating casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.	<ul style="list-style-type: none">Installation of battered piles somewhat more difficult for larger diameter elements.Requirement for larger drill rig and equipment set-up on barge in lake may make over-water construction difficult.Requires specialty contractor to install piles.	<ul style="list-style-type: none">Higher cost per pile than driven piles and smaller diameter drilled steel casings due to more complex installation, but potentially fewer piles required as a result of higher capacity.Smaller number of pile elements may result in some cost savings.	<ul style="list-style-type: none">Complex subsurface conditions resulting in difficult construction and potential difficulties seating larger diameter steel casings on sloping, strong to very strong bedrock (especially for 0.760 m diameter pile) and drilling rock sockets which could increase costs and potentially affect schedule.If casings not adequately sealed, there is a potential of debris and materials impeding rock socket construction.High potential for variable pile lengths due to variable depth to bedrock.
Composite Foundation Elements (Drilled Shaft combined with Micropiles socketted into bedrock)	2	<ul style="list-style-type: none">Ease of socketing smaller diameter micropile elements into the very strong and sloping bedrock.Large diameter drilled shaft provides stiffer section through water column and increased lateral resistance in upper very soft organic soil stratum.	<ul style="list-style-type: none">Lateral loads on fewer, large diameter drilled shafts are significant and the overburden along the length of the drilled shafts offers low lateral resistance. Lateral forces from pile cap expected to transfer to great depth along piles at some critical locations. Designing the micropiles to transfer the high bending moments from the drilled shaft to the bedrock would be challenging at some locations.	<ul style="list-style-type: none">Probably highest cost per pile element given size and considering that composite pile requires construction of two different pile types per element.Additional costs for mobilizing two different types of equipment (over water) to install different pile types.Although a potentially smaller number of pile elements may result in some cost savings, total costs are expected to be higher than drilled steel casings option.	<ul style="list-style-type: none">Potential for base instability/heave when drilling/cleaning out large diameter liners in overburden if pressures not balanced.Large diameter drilled shafts (even as composite foundation elements) would require the use of large equipment installation. The use of large equipment would pose additional constructability challenges due to working over the water.
Small Diameter Drilled Steel Casings (0.406 m Ø) or Micropiles (0.273 m Ø) socketed into bedrock using DTH hammer drilling	3	<ul style="list-style-type: none">Relatively straight forward construction.Smaller diameter casings may be installed with relatively smaller drilling equipment (making it easier for over-water work).High axial capacity (limited by structural design).Can be battered to suit almost any angle typically used in similar type application.Smaller diameter DTH hammer drilling method offers best chance of seating casings in steeply sloping bedrock.Potentially structurally advantageous if installed in groups of 2 or 3 within 1.2 m to 1.5 m diameter upper steel casings backfilled with grout to provide large, stiff cross-section for higher lateral resistance in weak overburden.Could be designed as a larger group of smaller pile elements (consisting of many vertical and battered piles).	<ul style="list-style-type: none">Requires specialty contractor to install piles (especially if larger pile diameters used).Smaller diameter individual pile elements offer less axial and lateral resistance, therefore a larger number of pile elements are required.	<ul style="list-style-type: none">Higher cost per pile than driven piles due to more complex installation.Higher cost than large diameter drilled steel casing option as larger number of individual pile elements required per pier.	<ul style="list-style-type: none">High potential for variable pile lengths due to variable depth to bedrock.



Table 2: Evaluation of Foundation Alternatives – Piers

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles (HP310x110) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	NR	<ul style="list-style-type: none">■ Relatively straight forward construction.■ Structurally advantageous if installed in groups of 3 within 1.2 m diameter upper steel casings backfilled with grout to provide large, stiff cross-section for higher lateral resistance in weak overburden.	<ul style="list-style-type: none">■ Highly variable depths to refusal across river may result in near shore piles having insufficient length to support piers (since piles are not socketed into bedrock).■ Presence of steeply sloping bedrock (at angles up to about 50°) may lead to difficult conditions to seat piles, especially if piles are battered.■ Presence of cobbles and boulders may result in obstructions during driving and making proper seating of piles more difficult.■ Axial capacity will have to be reduced considering the steeply sloping nature of bedrock, the presence of cobbles and boulders and potential for pile damage to occur during driving and seating.■ Smaller diameter individual pile elements offer less lateral resistance.■ Large pile driving rig required for installation makes over-water work difficult.■ Long pile section extending through water column and very soft organic silt stratum does not provide for lateral resistance.	<ul style="list-style-type: none">■ Lower cost per pile than drilled steel casings, but more piles may be required as a result of lower axial capacity.■ Difficult construction conditions (i.e., large pile driving rig on barge in river) may result in higher equipment/ mobilization cost than drilled steel casing alternative.	<ul style="list-style-type: none">■ High potential for variable pile lengths due to variable depth to bedrock.■ Variable overburden thickness may result in insufficient length of piles to develop required lateral resistance (given unsupported, free length in water) considering piles are not socketed into bedrock.■ Difficulties seating driven piles on steeply sloping, strong to very strong bedrock which will raise costs and potentially affect schedule.
Steel H-piles (HP310x110) fixed into bedrock sockets backfilled with concrete	NR	<ul style="list-style-type: none">■ Fixing base of steel piles into bedrock sockets provides improved seating conditions over driven pile options.■ No concern over near shore piles having insufficient length since piles seated into bedrock.■ DTH hammer drilling method offers best chance of seating temporary casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.	<ul style="list-style-type: none">■ Removal of temporary liner after construction of rock socket will cause disturbance (softening/loosening) of overburden soils and significantly reduce lateral resistance.■ Installation of battered piles somewhat more difficult.■ Requirement for larger drill rig and equipment set-up on barge in lake for socket construction may make over-water construction difficult.■ Requires specialty contractor to drill rock sockets prior to installation of steel piles.■ Smaller diameter individual pile elements offer less lateral resistance.■ Large pile driving rig required for installation makes over-water work difficult.	<ul style="list-style-type: none">■ High cost per pile given that drilling of rock socket (with temporary liner) required before installation of steel pile.■ Higher overall cost given that more pile elements likely required to satisfy lateral resistances due to disturbances caused during installation.	<ul style="list-style-type: none">■ High potential for variable pile lengths due to variable depth to bedrock.■ Difficult construction and subsurface conditions and potential for difficulties seating larger diameter steel casings on sloping, strong to very strong bedrock and drilling rock sockets which could raise costs and potentially affect schedule■ Disturbance to overburden soils during removal of temporary drill casing will result in low lateral resistance for steel piles.
Concrete Caissons (Drilled Shafts) with permanent steel liners (0.9 m = Ø) socketed into bedrock	NR	<ul style="list-style-type: none">■ High axial capacity.■ High lateral capacity.■ Smaller number of pile elements required per pier.	<ul style="list-style-type: none">■ Presence of very strong to extremely strong sloping bedrock (at angles up to about 50°) will make socketing large diameter steel liners into bedrock very difficult.■ Cannot be easily battered.■ Requirement for large drill rig set-up on barge in river will make over-water construction difficult.■ The larger the caisson diameter the greater the likelihood of encountering difficulties to drill the caisson into the sloping surface and very strong bedrock.	<ul style="list-style-type: none">■ Difficult construction conditions (i.e., large caisson rig on barge in river) may result in higher equipment / mobilization cost than drilled steel casing alternatives.■ Smaller number of pile elements may result in some cost savings.	<ul style="list-style-type: none">■ Potential for variable caisson lengths due to variable depth to bedrock.■ Potential for base instability/heave when drilling/cleaning out large diameter liners in overburden if pressures not balanced.■ Potentially difficult construction conditions and difficulties seating steel liners on steeply sloping and very strong to extremely strong bedrock and drilling rock sockets which will raise costs and potentially affect schedule.■ If liners not adequately sealed, and the caisson base not properly cleaned, debris and materials will impede rock socket construction and tremie concreting operation.■ Large diameter drilled shafts (even as composite foundation elements) would require the use of large equipment installation. The use of large equipment would pose additional constructability challenges due to working over the water.



Table 2: Evaluation of Foundation Alternatives – Piers

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow Spread Footings on Overburden in Riverbed	NF		<ul style="list-style-type: none">■ Very low factored axial geotechnical resistance at ULS and geotechnical reaction at SLS.■ Relatively deep excavations into riverbed require to penetrate through organic silt deposit.		<ul style="list-style-type: none">■ Not feasible due to depth of river water and presence of very weak and compressible near surface overburden soils below riverbed.■ Likely require scour protection.

NR: Alternative not recommended for this site.
NF: indicates that the founding option is not feasible

Prepared By: AB
Checked By: JPD
Reviewed By: JMAC



FOUNDATION REPORT – KEY RIVER NBL BRIDGE – HIGHWAY 69
GWP 5005-10-00; WP 5147-08-01

Table 3 : Pier 1 (South Pier) – Stratigraphy based on Borehole B504-09 (towards Centre of River) and Borehole B504-01 (towards Shore)

Overburden Stratum	Pile Tip Direction	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ' (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k (kPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Water	Centre of River	176.3 - 172.5	-	-	-	-	-	-	-	-	-
	Shore	176.3 - 173.1									
Very Soft to Soft Organic Silt to Clayey Silt	Centre of River	172.5 – 165.0	API Soft Clay (Matlock)	14	10	-	0.04	-	1,000	330	0.5
	Shore	173.1 - 166.0									
Very Soft to Firm Clayey Silt	Centre of River	165.0 – 162.6	API Soft Clay (Matlock)	16	30	-	0.02	-	1,500	500	0.5
	Shore	166.0 - 162.3									
Loose to Compact Sand	Centre of River	162.6 – 154.0	API Sand (O'Neill)	19	-	32	-	14,000	10,000	3,700	0.35
Very Dense Gravelly Sand	Centre of River	154.0 – 153.0	API Sand (O'Neill)	22	-	34	-	19,200	50,000	19,200	0.3
Rock Type	Pile Tip Direction	Elevation (m)	Employed Model	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss	Centre of River	Below 153.0	User Defined	26	100*	31	0.7	90%	-	-	0.25
	Shore	Below 162.3									

* Note: If the location of the groundwater table/top of river is not explicitly modelled in the analysis, then effective (i.e., submerged) unit weights should be utilized ($\gamma' = \gamma_b - 9.81$ kN/m³).

Prepared By: ARV
Reviewed By: JPD



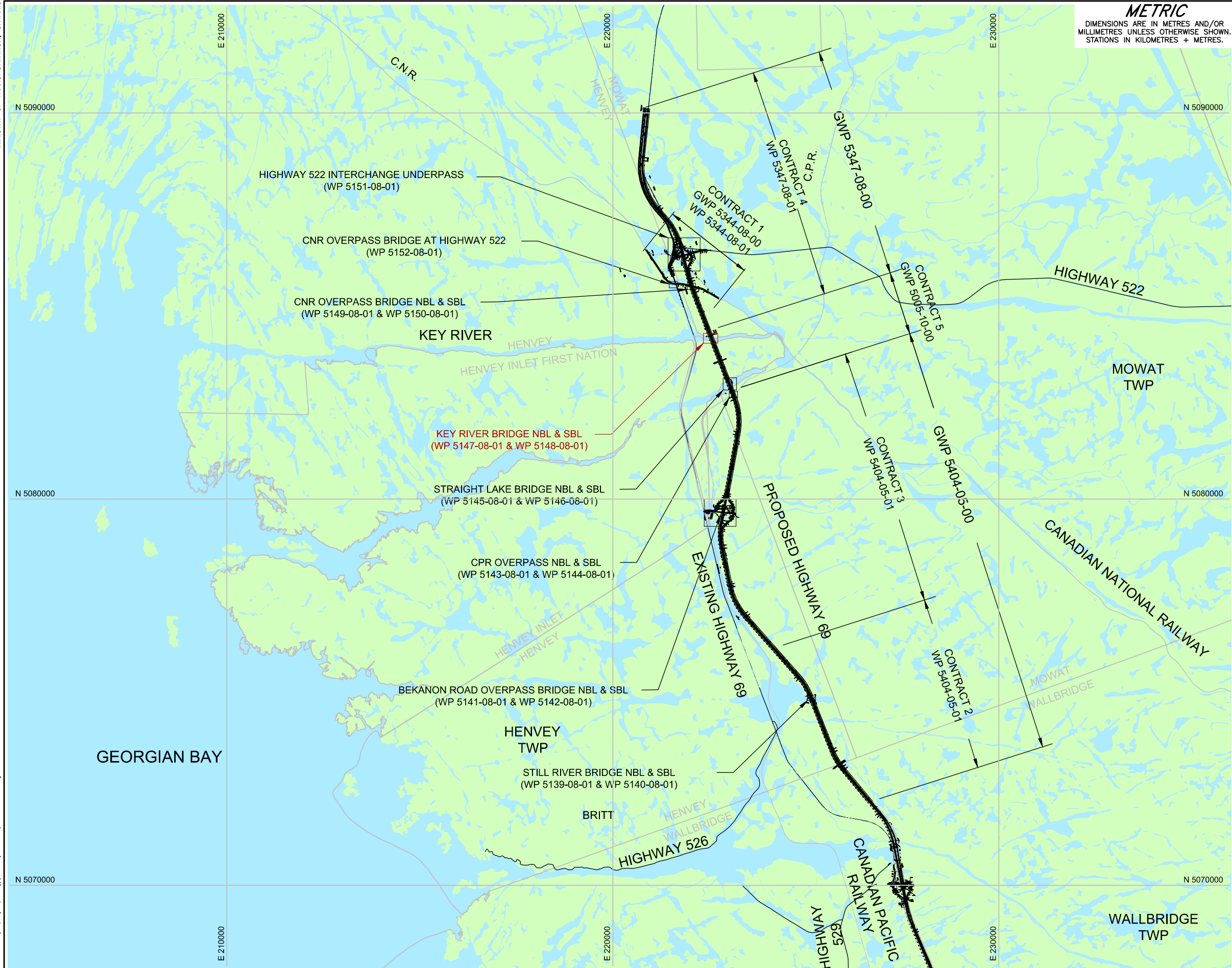
FOUNDATION REPORT – KEY RIVER NBL BRIDGE – HIGHWAY 69
GWP 5005-10-00; WP 5147-08-01

Table 4: Pier 2 (North Pier) – Stratigraphy based on Borehole B504-12 (towards Centre of River) and Borehole B504-02 (towards Shore)

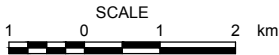
Overburden Stratum	Pile Tip Direction	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ' (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k (kPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Water	Centre of River	176.3 – 170.8	-	-	-	-	-	-	-	-	-
	Shore	176.3 - 171.1									
Very Soft to Soft Organic Silt to Clayey Silt	Centre of River	170.8 – 162.0	API Soft Clay (Matlock)	14	10	-	0.04	-	1,000	330	0.5
	Shore	171.1 – 162.5									
Loose to Compact Silt and Sand to Sand	Centre of River	162.0 – 150.2	API Sand (O'Neill)	19	-	32	-	14,000	10,000	3,700	0.35
Loose to Compact Sandy Silt to Sand	Shore	162.5 – 151.0	API Sand (O'Neill)	19	-	30	-	9,500	5,000	1,850	0.35
Cobbles / Boulders	Centre of River	150.2 – 147.7	API Sand (O'Neill)	23	-	34	-	19,200	50,000	19,200	0.3
Rock Type	Pile Tip Direction	Elevation (m)	Employed Model	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss	Centre of River	Below 147.7	User Defined	26	100*	31	0.7	90%	-	-	0.25
	Shore	Below 151.0									

* Note: If the location of the groundwater table/top of river is not explicitly modelled in the analysis, then effective (i.e., submerged) unit weights should be utilized ($\gamma' = \gamma_b - 9.81$ kN/m³).

Prepared By: ARV
Reviewed By: JPD



PLAN



METRIC
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CONT No. .
WP No. 5147-08-01

HIGHWAY 69
KEY RIVER NBL BRIDGE
INDEX PLAN

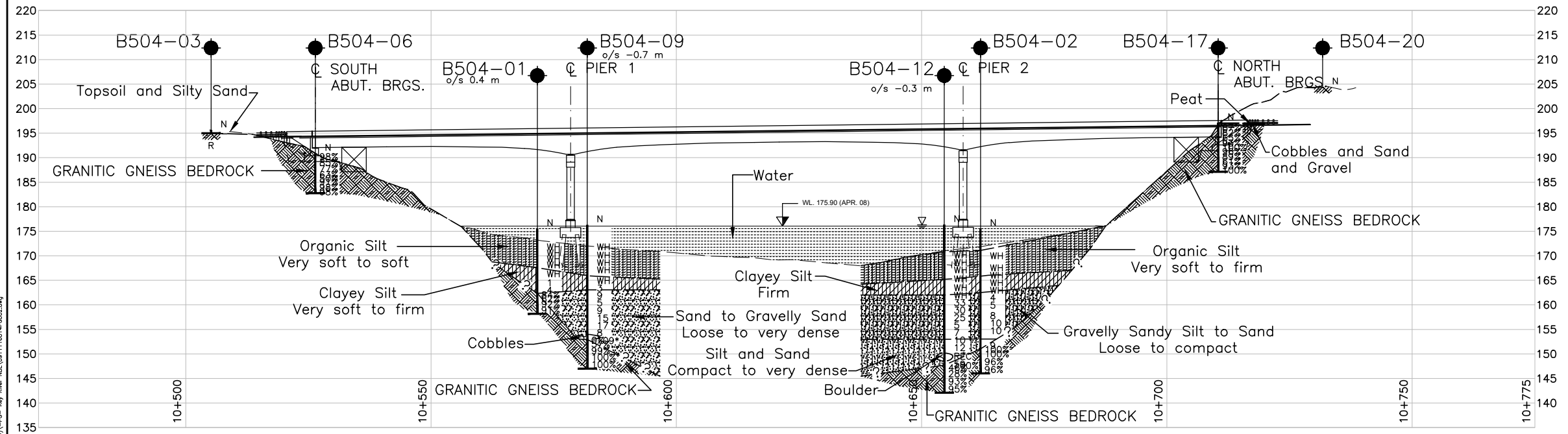
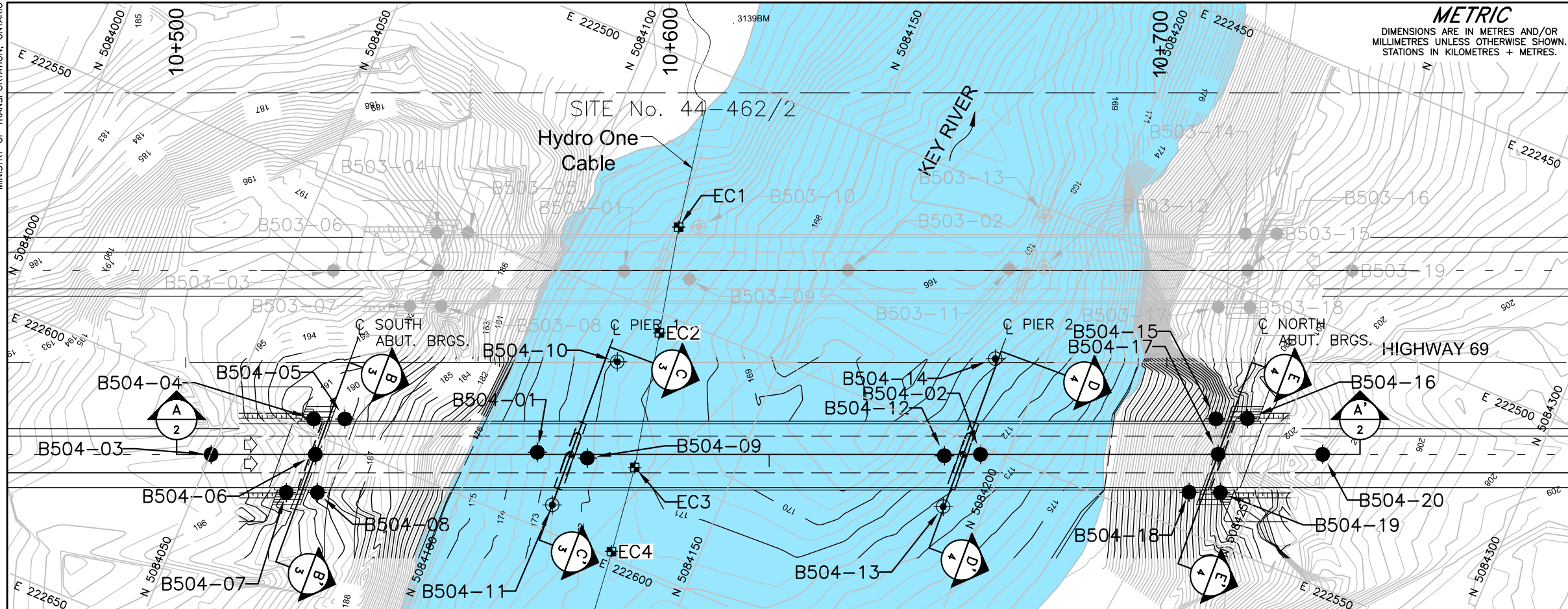


KEY PLAN
SCALE
6 0 6 12 km

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

NO.	DATE	BY	REVISION	
Geocres No. 41H-153				
HWY. 69		PROJECT NO. 09-1111-6014		DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: Dec. 2015	SITE: 44-462/1	
DRAWN: JFC	CHKD. AB	APPD. JPD/JMAC	DWG. 1	



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.

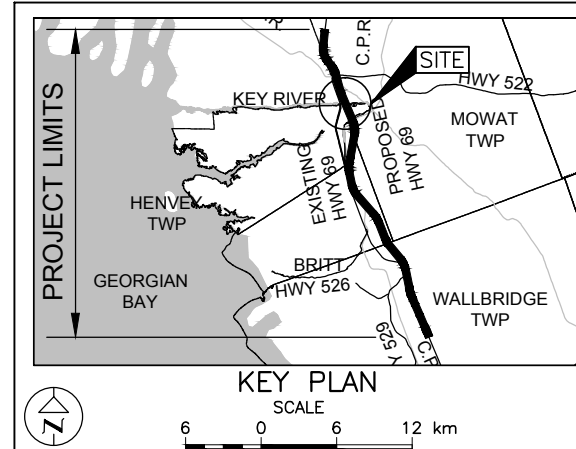
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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CONT No. 5147-08-01

HIGHWAY 69
KEY RIVER NBL BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA



- LEGEND**
- Borehole - Current Investigation
 - ⊕ Probeshole
 - Hydro One Cable
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - 100% Rock Quality Designation (RQD)
 - ≡ WL upon completion of drilling
 - R Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B504-01	175.5	5084111.4	222585.9
B504-02	175.5	5084194.9	222551.6
B504-03	195.6	5084050.1	222611.8
B504-04	191.1	5084066.7	222597.1
B504-05	189.7	5084072.5	222594.7
B504-06	190.8	5084069.8	222603.7
B504-07	192.4	5084067.3	222613.1
B504-08	190.1	5084073.2	222610.7
B504-09	176.3	5084121.2	222583.1
B504-10	176.3	5084119.3	222562.6
B504-11	176.3	5084118.3	222594.6
B504-12	176.3	5084188.2	222554.7
B504-13	176.3	5084192.0	222564.3
B504-14	176.3	5084190.2	222532.4
B504-15	196.6	5084236.5	222526.5
B504-16	199.6	5084242.4	222524.0
B504-17	197.1	5084239.6	222532.9
B504-18	193.6	5084237.1	222542.4
B504-19	195.4	5084243.1	222540.0
B504-20	204.4	5084259.3	222524.8

REFERENCE			
Base plans provided in digital format by URS, drawing file nos. Contours from Hwy69_Contour-Plan_CS.dwg, received August 31, 2012, KEY RIVER CROSSING OPTION B_Northbound_GA.dwg and KEY RIVER CROSSING OPTION B_Southbound_GA.dwg, received November 4, 2013.			
NO.	DATE	BY	REVISION
Geocres No. 41H-153			
HWY. 69		PROJECT NO. 09-1111-6014	
SUBM'D. MCK	CHKD. MCK	DATE: Dec. 2015	SITE: 44-462/1
DRAWN: JFC	CHKD. AB	APPD. JPD/JMAC	DWG. 2

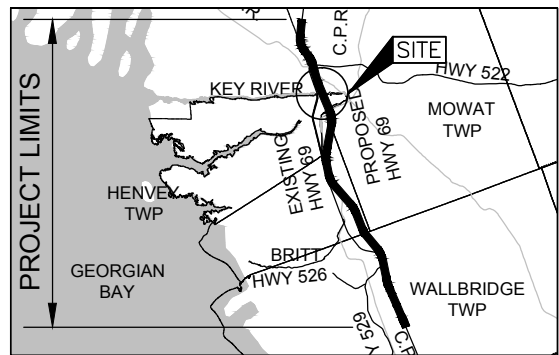
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STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.5147-08-01

HIGHWAY 69
KEY RIVER NBL BRIDGE

SOIL STRATA

SHEET



KEY PLAN
SCALE
6 0 6 12 km

LEGEND

- Borehole - Current Investigation
- ⊕ Probehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL upon completion of drilling

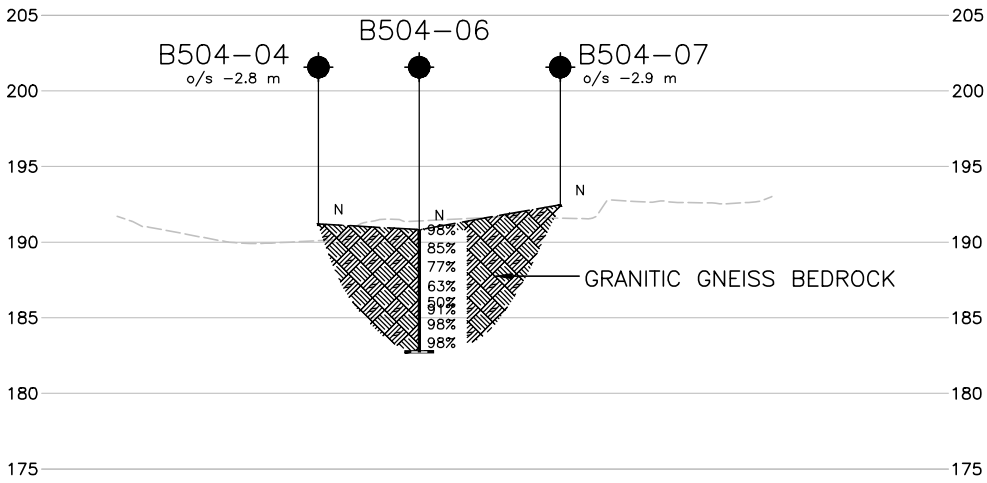
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B504-04	191.1	5084066.7	222597.1
B504-06	190.8	5084069.8	222603.7
B504-07	192.4	5084067.3	222613.1
B504-09	176.3	5084121.2	222583.1
B504-10	176.3	5084119.3	222562.6
B504-11	176.3	5084118.3	222594.6

NOTES

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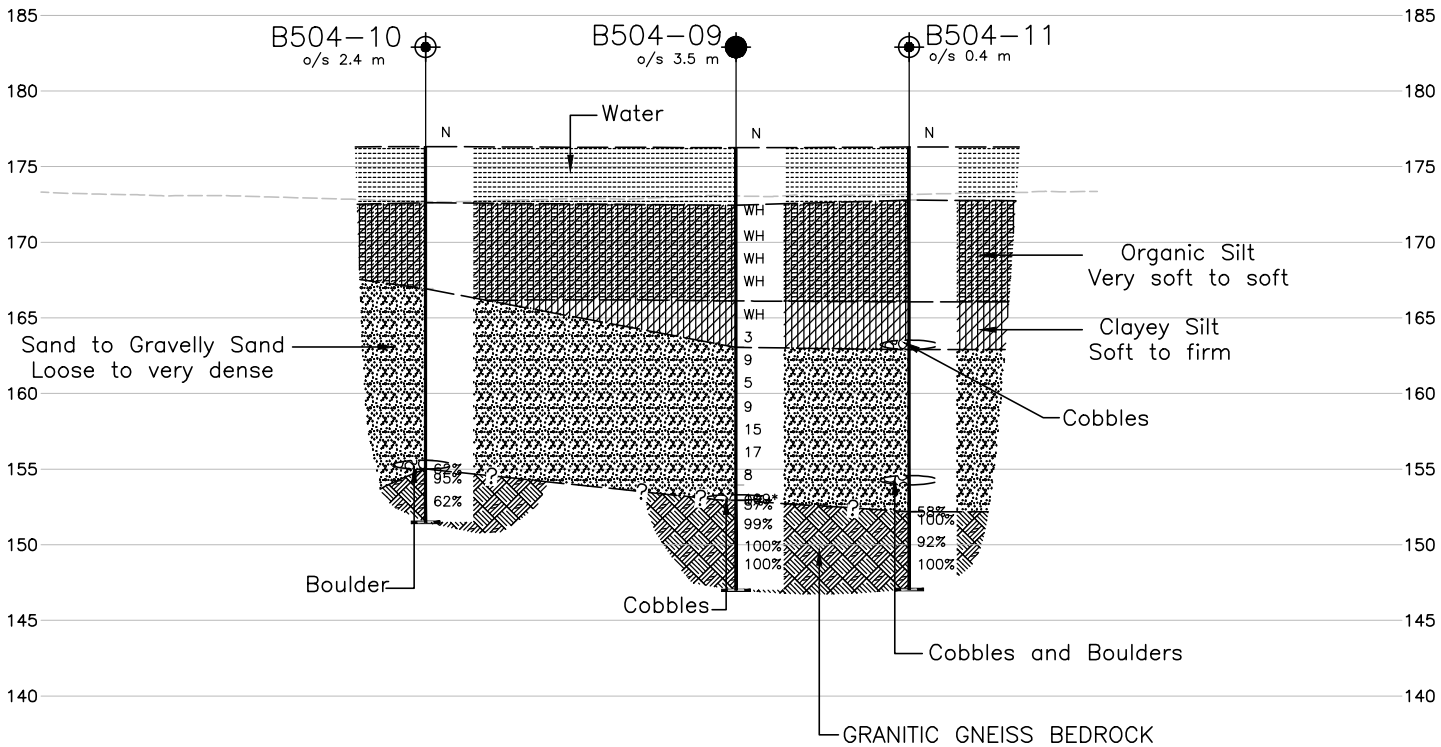
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B-B'
1 NBL SOUTH ABUTMENT CROSS-SECTION B-B'

SCALE
5 0 5 10 m

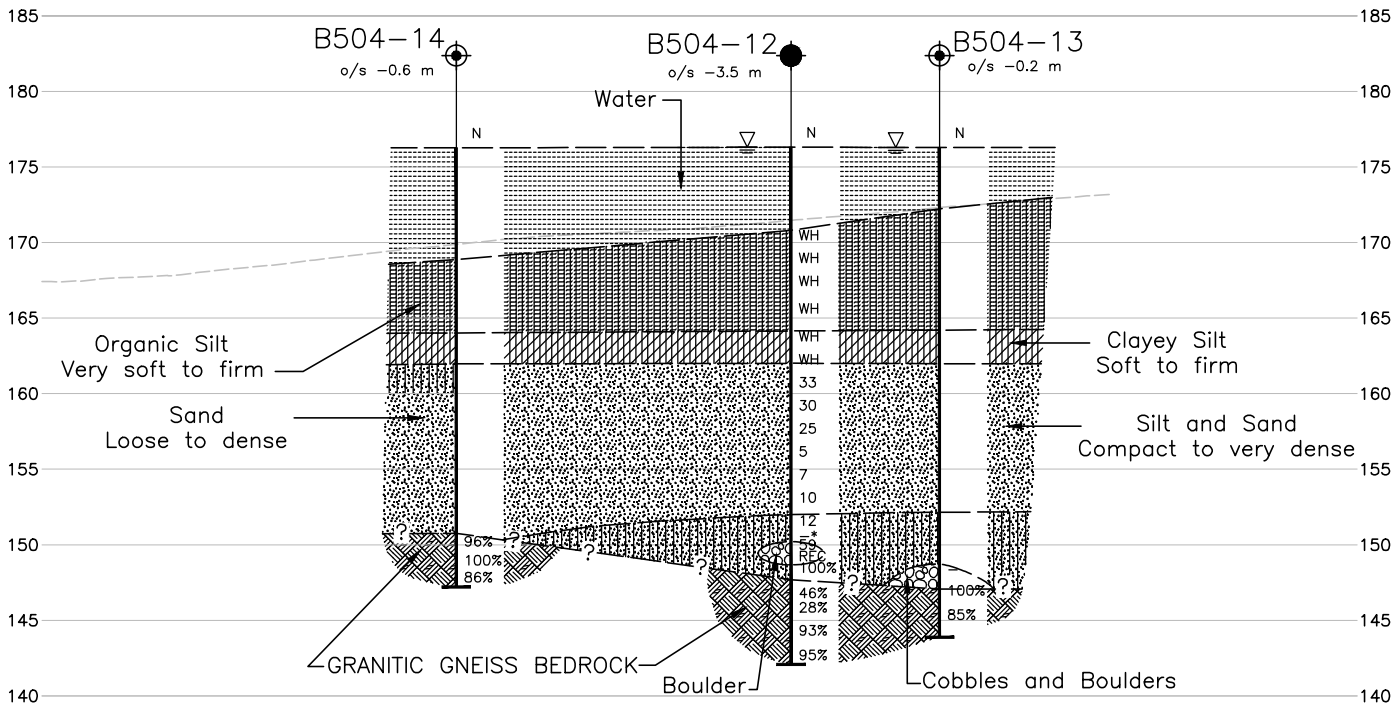


C-C'
1 NBL SOUTH PIER CROSS-SECTION C-C'

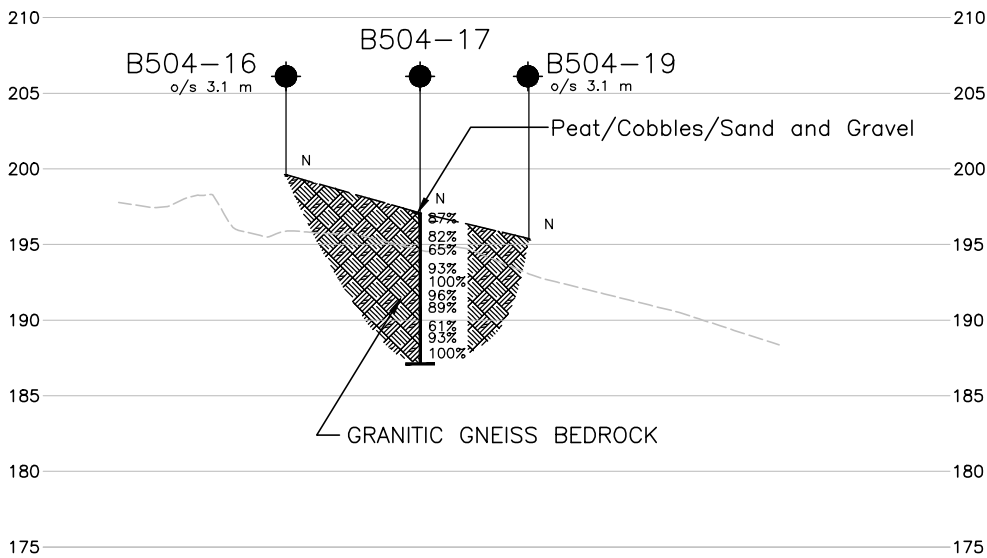
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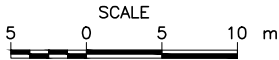
NO.	DATE	BY	REVISION
Geocres No. 41H-153			
HWY. 69	PROJECT NO. 09-1111-6014		DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: Dec. 2015	SITE: 44-462/1
DRAWN: JFC	CHKD. AB	APPD. JPD/JMAC	DWG. 3



D-D' NBL NORTH PIER CROSS-SECTION D-D'
1



E-E' NBL NORTH ABUTMENT CROSS-SECTION E-E'
1

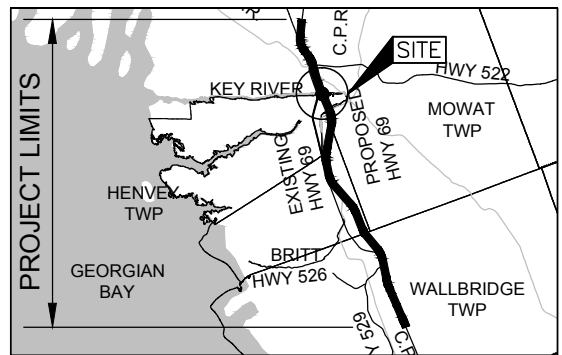


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STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.5147-08-01

HIGHWAY 69
KEY RIVER NBL BRIDGE
SOIL STRATA

SHEET



KEY PLAN

SCALE

6 0 6 12 km

LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B504-12	176.3	5084188.2	222554.7
B504-13	176.3	5084192.0	222564.3
B504-14	176.3	5084190.2	222532.4
B504-16	199.6	5084242.4	222524.0
B504-17	197.1	5084239.6	222532.9
B504-19	195.4	5084243.1	222540.0

NOTES

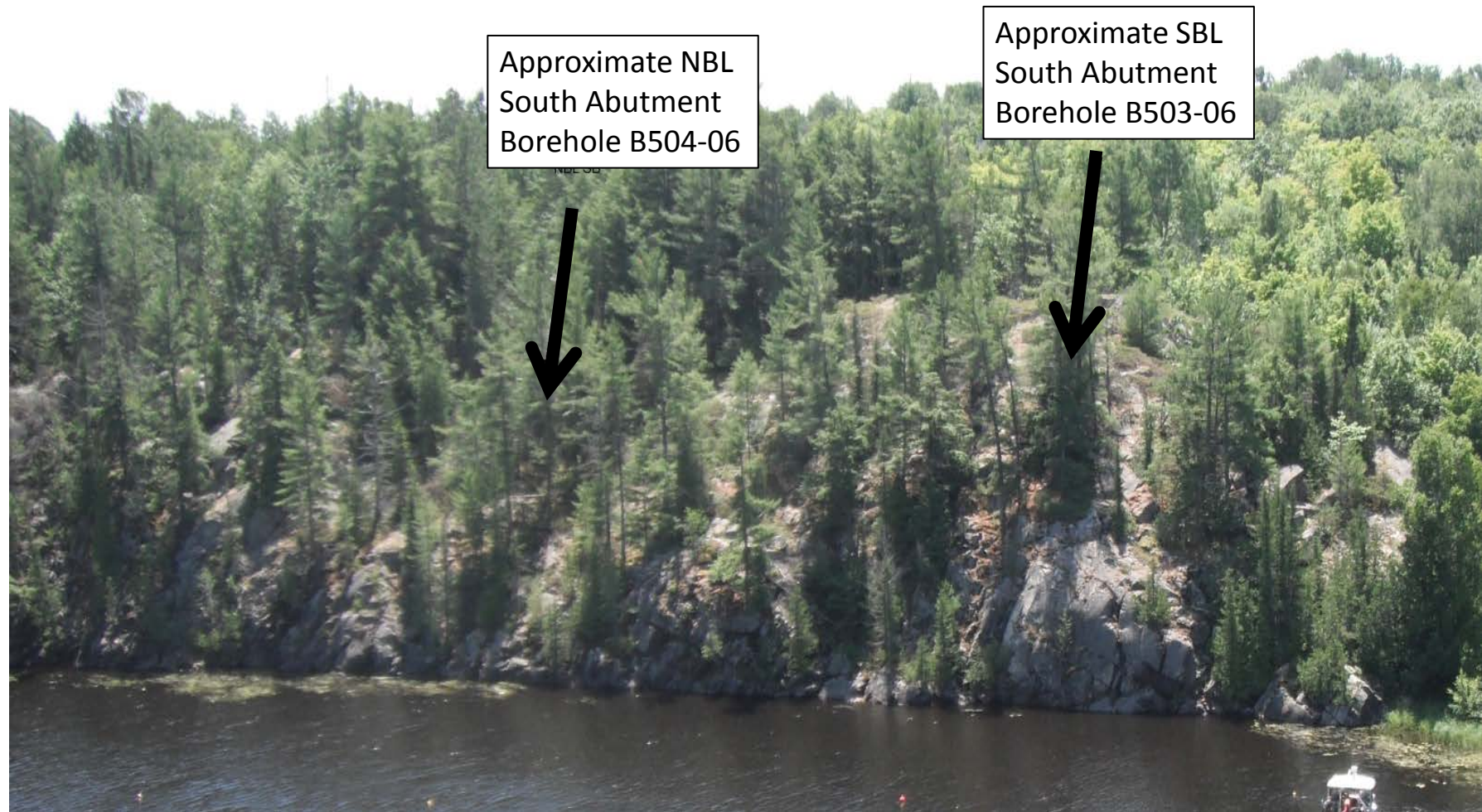
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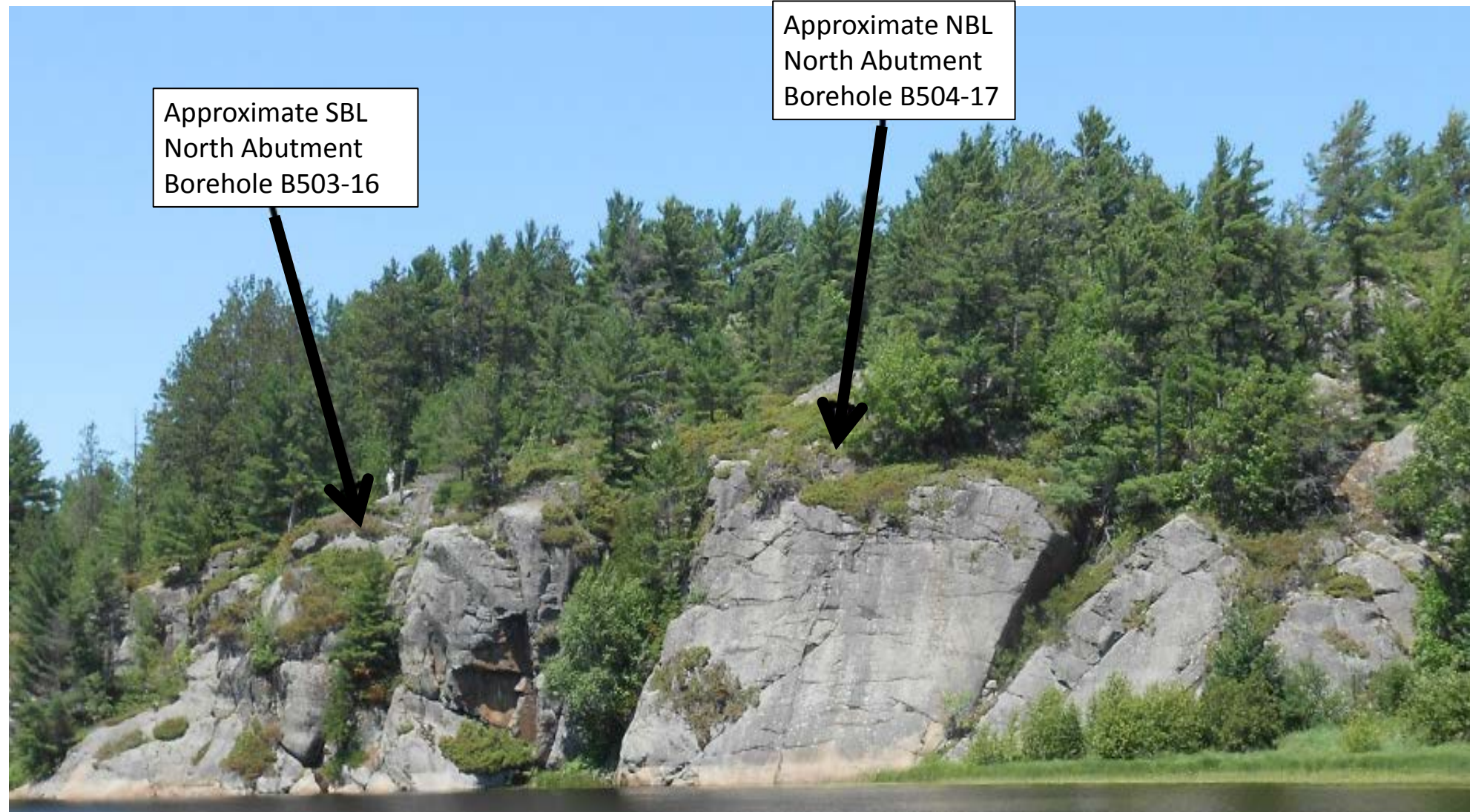
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NO.	DATE	BY	REVISION
Geocres No. 41H-153			
HWY. 69	PROJECT NO. 09-1111-6014		DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: Dec. 2015	SITE: 44-462/1
DRAWN: JFC	CHKD. AB	APPD. JPD/JMAC	DWG. 4



PROJECT					
Highway 69 - Key River NBL and SBL					
TITLE					
KEY RIVER NBL AND SBL PHOTOGRAPH OF SOUTH SHORE ROCK OUTCROP					
	PROJECT No. 09-1111-6014			FILE No. ----	
	DESIGN	ARB	April 2015	SCALE	AS SHOWN
	CADD	--			REV.
	CHECK	AB	April 2015	FIGURE 1	
	REVIEW				



Approximate SBL
North Abutment
Borehole B503-16

Approximate NBL
North Abutment
Borehole B504-17

PROJECT

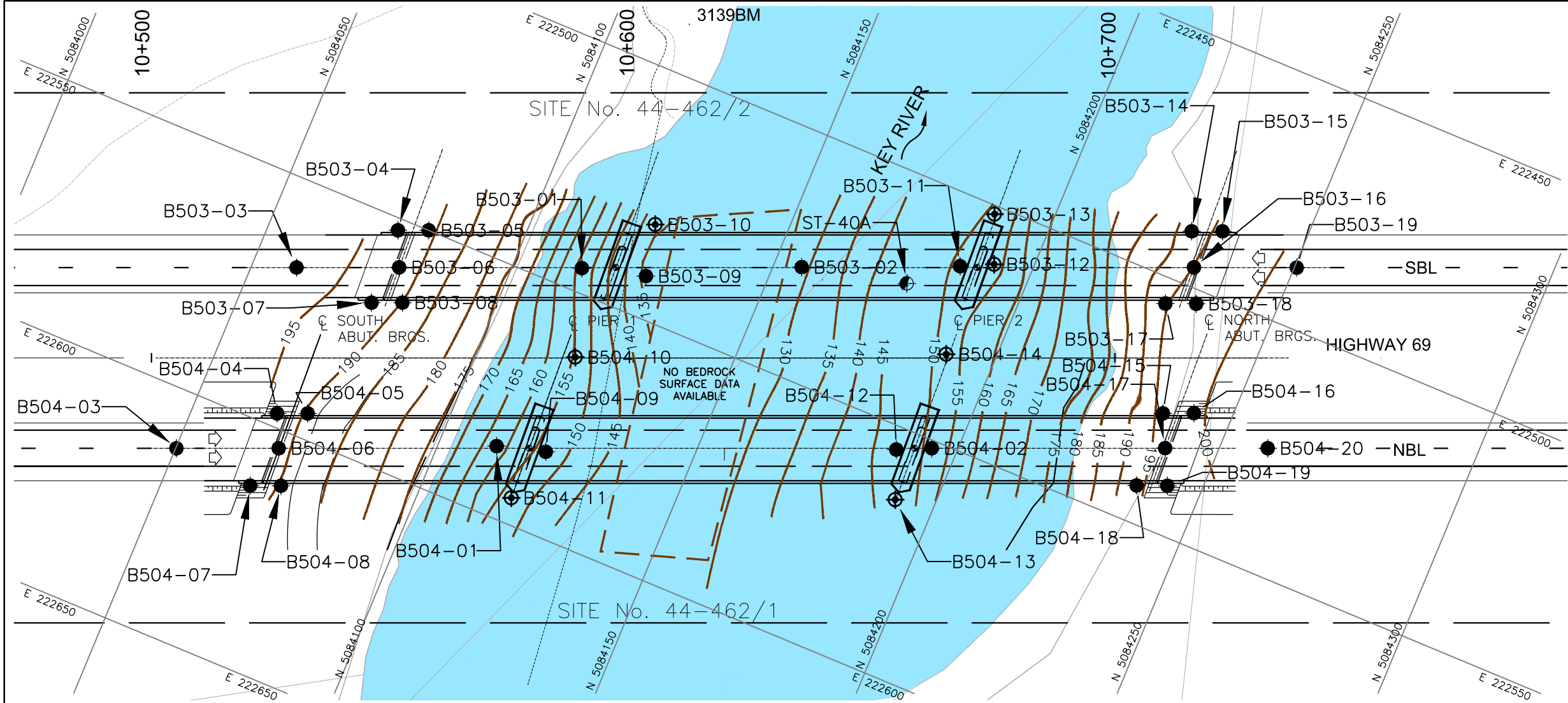
Highway 69 - Key River NBL and SBL

TITLE

**KEY RIVER NBL AND SBL
PHOTOGRAPH OF NORTH SHORE ROCK
OUTCROP**



PROJECT No. 09-1111-6014			FILE No. ----	
DESIGN	ARB	April 2015	SCALE	AS SHOWN
CADD	--		REV.	
CHECK	AB	April 2015	FIGURE 2	
REVIEW				

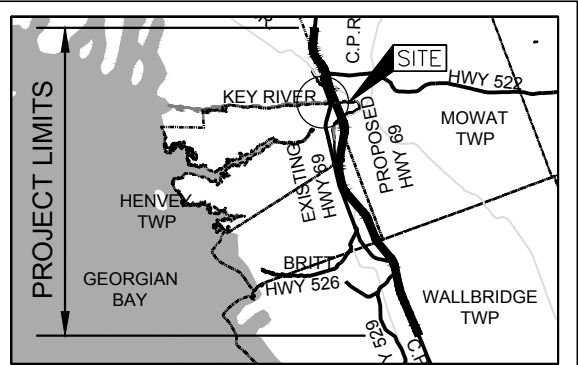


PLAN



BOREHOLE CO-ORDINATES			
No.	TOP OF BEDROCK ELEVATION	NORTHING	EASTING
B503-01	153.2	5084113.5	222545.0
B503-02	129.1	5084155.6	222527.3
B503-03	197.7	5084058.8	222567.6
B503-04	193.2	5084075.2	222552.4
B503-05	191.1	5084081.1	222550.0
B503-06	193.9	5084078.4	222559.4
B503-07	192.7	5084075.9	222568.4
B503-08	190.0	5084081.8	222565.9
B503-09	133.4	5084126.4	222541.4
B503-10	135.5	5084124.1	222530.8
B503-11	150.5	5084185.8	222514.4
B503-12	152.7	5084192.2	222511.4
B503-13	150.7	5084188.3	222501.8
B503-14	193.2	5084227.5	222489.3
B503-15	195.8	5084233.4	222486.8
B503-16	194.8	5084230.8	222496.0
B503-17	194.9	5084228.2	222505.2
B503-18	197.4	5084234.1	222502.8
B503-19	200.9	5084250.5	222487.9
ST-40A	145.0 (Inferred)	5084177.0	222522.0

BOREHOLE CO-ORDINATES			
No.	TOP OF BEDROCK ELEVATION	NORTHING	EASTING
B504-01	162.3	5084111.4	222585.9
B504-02	151.0	5084194.9	222551.6
B504-03	195.6	5084050.1	222611.8
B504-04	191.1	5084066.7	222597.1
B504-05	189.7	5084072.5	222594.7
B504-06	190.8	5084069.8	222603.7
B504-07	192.4	5084067.3	222613.1
B504-08	190.1	5084073.2	222610.7
B504-09	153.0	5084121.2	222583.1
B504-10	155.0	5084119.3	222562.6
B504-11	152.2	5084118.3	222594.6
B504-12	147.7	5084188.2	222554.7
B504-13	147.1	5084192.0	222564.3
B504-14	150.8	5084190.2	222532.4
B504-15	196.6	5084236.5	222526.5
B504-16	199.6	5084242.4	222524.0
B504-17	196.8	5084239.6	222532.9
B504-18	193.6	5084237.1	222542.4
B504-19	195.4	5084243.1	222540.0
B504-20	204.4	5084259.3	222524.8



KEY PLAN

SCALE
12 0 12 24 km

LEGEND

- Borehole - Current Investigation (Golder)
- Probehole - Current Investigation (Golder)
- Borehole - Previous Investigation (AMEC)
- Interpreted Estimated Bedrock Contour Interval (5 m)

NOTE:
Bedrock surface contours are approximate only and based on limited subsurface information.

Actual Bedrock surface elevations are unknown between borehole locations where coring was carried out and will likely vary from that shown.

Interpreted Bedrock surface contours should not be relied upon for design, construction or quantity estimates.

NOTES

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

REFERENCE

Base plans provided in digital format by URS, drawing file nos. Contours from Hwy69_Contour-Plan_C5.dwg, received August 31, 2012, KEY RIVER CROSSING OPTION B_Northbound_GA.dwg and KEY RIVER CROSSING OPTION B_Southbound_GA.dwg, received November 4, 2013.

CLIENT
MTO

CONSULTANT



YYYY-MM-DD 2015-04-08
DESIGNED MCK
PREPARED JFC
REVIEWED AB
APPROVED JPD/JMAC

PROJECT
HIGHWAY 69
KEY RIVER NBL AND SBL BRIDGES

TITLE
INTERPRETED APPROXIMATE BEDROCK
SURFACE CONTOURS

PROJECT NO. 09-1111-6014

A 0001

REV. A

FIGURE 3



APPENDIX A

Record of Boreholes and Drillholes



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

III. SOIL DESCRIPTION

(a) Non-Cohesive 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

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.)

(b) Cohesive Soils Consistency

	<u>kPa</u>	<u>Cu, Su</u>	<u>psf</u>
Very soft	0 to 12		0 to 250
Soft	12 to 25		250 to 500
Firm	25 to 50		500 to 1,000
Stiff	50 to 100		1,000 to 2,000
Very stiff	100 to 200		2,000 to 4,000
Hard	over 200		over 4,000

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial 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) 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 <u>09-1111-6014</u>		RECORD OF BOREHOLE No B504-01		SHEET 1 OF 2		METRIC	
W.P. <u>5147-08-01</u>		LOCATION <u>N 5084111.4 ; E 222585.9</u>		ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>100 mm I.D. HW Casing, Wash Boring</u>		COMPILED BY <u>JFC</u>			
DATUM <u>Geodetic</u>		DATE <u>November 14 and 16, 2012</u>		CHECKED BY <u>MCK/AB</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	20 40 60	W _p W W _L						
175.5 0.0	WATER SURFACE WATER														GR SA SI CL		
173.1 2.4	ORGANIC SILT, trace sand Very soft to soft Brown Wet																
			1	SS	WH									OC = 4.6%			
							3 +										
			2	SS	WH												
							2 +										
			3	SS	WH												
			4	SS	WH												
							2 +							OC = 11.4%			
167.3 8.2	CLAYEY SILT, trace sand Very soft to firm Grey Wet Sandy silt to silty sand seams below a depth of 8.2 m (Elev. 167.3 m)																
			5	TO	WH												
							2 +										
			6	SS	1												
			7	SS	4												
162.3 13.2	Granite Gneiss (BEDROCK) Bedrock cored from depths 13.2 m to 17.3 m. For bedrock coring details refer to Record of Drillhole B504-1.																
			1	RC	REC 90%										RQD = 84%		
			2	RC	REC 96%										RQD = 62%		
			3	RC													

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-01				SHEET 2 OF 2		METRIC										
W.P. 5147-08-01		LOCATION N 5084111.4 ; E 222585.9				ORIGINATED BY LK												
DIST _____ HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring				COMPILED BY JFC												
DATUM Geodetic		DATE November 14 and 16, 2012				CHECKED BY MCK/AB												
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 (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60						
158.2	Granite Gneiss (BEDROCK) Bedrock cored from depths 13.2 m to 17.3 m. For bedrock coring details refer to Record of Drillhole B504-1.		3	RC	REC 100%	160												RQD = 92%
17.3			4	RC	REC 97%	159												RQD = 91%
17.3	END OF BOREHOLE																	

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-01

SHEET 1 OF 1

LOCATION: N 5084111.4 ; E 222585.9

DRILLING DATE: November 15 to 16, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D55

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.25	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES
		Continued from Record of Borehole B504-1		162.34															
	NW Casing	Fresh, foliated, medium grained, slightly porous, very strong, grey and black GRANITIC GNEISS		13.16	1							JN,PL,RO	1.5	1					14.68 MPa (Axial)
14					2							JN,CU,RO JN,PL,VR JN,PL,RO	3 1.5 1.5	1 1 1					
15					3							JN,PL JN,PL,SM JN,PL,SM	1 1 1	2 2 2					
16	NORC November 15 to 16, 2012				4							JN,UN,RO JN,PL,RO JN,PL,RO	3 1.5 1.5	1 1 1					23.88 MPa
17												JN,PL,RO	1.5	1					7.20 MPa (Axial)
		END OF DRILLHOLE		158.19								JN,PL,RO	1.5	1					
18				17.31															
19																			
20																			
21																			
22																			
23																			

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15



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

PROJECT		RECORD OF BOREHOLE		No B504-02		SHEET 2 OF 2		METRIC									
W.P. 09-1111-6014		LOCATION		N 5084194.9 ; E 222551.6		ORIGINATED BY		LK									
DIST		HWY 69		BOREHOLE TYPE		100 mm I.D. HW Casing, Wash Boring		COMPILED BY									
JFC		DATE		November 7 to 9, 2012		CHECKED BY		MCK/AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
157.2	Gravelly Sandy SILT, trace clay Loose Grey Wet		7	SS	5												
159																	
158			8	SS	8												
157	SAND, trace to some silt, trace gravel and clay Loose to compact Grey Wet		9	SS	10												
156																	
155			10	SS	10												0 93 6 1
154																	
153																	
152			11	SS	9												
151	Granitic Gneiss (BEDROCK)		1	RC	REC 100%												RQD = 100%
150	Bedrock cored from depths of 24.5 m to 29.5 m. For bedrock coring details refer to Record of Drillhole B504-02.		2	RC	REC 100%												RQD = 94%
149																	
148			3	RC	REC 96%												RQD = 96%
147			4	RC	REC 96%												RQD = 96%
146.0	END OF BOREHOLE																
29.5																	

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-02

SHEET 1 OF 1

LOCATION: N 5084194.9 ; E 222551.6

DRILLING DATE: November 8 to 9, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D55

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY K, cm/sec			Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES	
					DEPTH (m)	TOTAL CORE %				SOLID CORE %	B Angle			DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴				10 ⁻³
		Continued from Record of Borehole B504-2		151.03																					
	HQ Casing	Fresh, foliated, medium grained, slightly porous, strong to very strong, pink, grey and black GRANITIC GNEISS		24.47	1																				11.77 MPa (Axial)
25				2																					UC = 94.1 MPa
	HQRC November 8 to 9, 2012																								
26																									
27																									8.69 MPa
28																									
29																									
		END OF DRILLHOLE			146.03																				
					29.47																				
30																									
31																									
32																									
33																									
34																									

DEPTH SCALE

1 : 50

Golder

Associates

LOGGED: LK

CHECKED: MCK/AB

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-03				SHEET 1 OF 1		METRIC									
W.P. 5147-08-01		LOCATION N 5084050.1 ; E 222611.8				ORIGINATED BY TM											
DIST _____ HWY 69		BOREHOLE TYPE Portable Equipment				COMPILED BY JFC											
DATUM Geodetic		DATE August 6, 2014				CHECKED BY MCK/AB											
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									
195.6	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL																
0.1	SILTY SAND, trace rootlets		1	AS	-												
195.0	Brown																
0.6	Moist																
	END OF BOREHOLE AUGER REFUSAL						195										
NOTE: 1. Borehole dry upon completion of drilling.																	



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

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

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-06

SHEET 1 OF 1

LOCATION: N 5084069.8 ; E 222603.7

DRILLING DATE: August 6, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	RECOVERY TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	B Angle °	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm/sec 10 ⁻⁹ 10 ⁻⁸ 10 ⁻⁷ 10 ⁻⁶	Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES
0		Continued from Record of Borehole B504-06		190.82																
1		Slightly weathered, foliated, grey and pink, coarse grained, faintly porous, strong to very strong GRANITIC GNEISS		0.00	1								JN,PL,RO SO	1.5	1					
2					2								JN,CU,RO BC	3	1					7.67 MPa
3					3								JN,PL,RO SO	1.5	1					
4	HORC August 6, 2014				4								JN,PL,RO SA	1.5	2					
5					5								JN,UN,RO	3	1					
6					6								JN,PL,RO BC	1.5	1					
7					7								FO,PL,RO SO	1.5	1					8.34 MPa (Axial)
8					8								JN,PL,RO	1.5	1					UC = 96.6 MPa
9		END OF DRILLHOLE		182.74																6.19 MPa (Axial)
10				8.08																7.35 MPa

DEPTH SCALE

1 : 50



LOGGED: TM/SP

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/26/15



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



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B504-09		SHEET 1 OF 2		METRIC	
W.P. <u>5147-08-01</u>		LOCATION <u>N 5084121.2 ; E 222583.1</u>		ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>100 mm I.D. HW Casing, Wash Boring</u>		COMPILED BY <u>JFC</u>			
DATUM <u>Geodetic</u>		DATE <u>August 5 to 7, 2014</u>		CHECKED BY <u>MCK/AB</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE 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		W _p	W	W _L		
							20 40 60 80 100							
176.3 0.0	WATER SURFACE WATER													
172.5 3.8	ORGANIC SILT Very soft to soft Grey Wet		1	SS	WH									
			2	SS	WH									
			3	SS	WH									
			4	SS	WH									
166.0 10.3	CLAYEY SILT, trace to some sand, trace gravel Soft to firm Grey Wet		5	SS	WH									
			6A 6B	SS	3									
162.6 13.7	SAND, trace so some silt, trace clay, trace gravel Loose to compact Grey Wet		7	SS	9									

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-09		SHEET 2 OF 2		METRIC						
W.P. 5147-08-01		LOCATION N 5084121.2 ; E 222583.1		ORIGINATED BY LK								
DIST _____ HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring		COMPILED BY JFC								
DATUM Geodetic		DATE August 5 to 7, 2014		CHECKED BY MCK/AB								
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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
	--- CONTINUED FROM PREVIOUS PAGE ---						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					
			8	SS	5	161						1 83 15 1
						160						
			9	SS	9	159						
						158						1 86 12 1
			10	SS	15	157						
						156						
			11	SS	17	155						
						154						
154.0	22.3											
	Gravelly SAND, some silt, trace clay, trace gravel											
	Loose to compact											
	Grey											
	Wet											
	Containing cobbles below a depth of 22.9 m (Elev. 153.4 m)		13	SS	109*	153						26 56 16 2
153.0	23.3											RQD = 0%
	Granitic Gneiss (BEDROCK)		1	RC	REC 100%							
	Bedrock cored from depths of 23.3 m to 29.3 m.		2	RC	REC 100%	152						RQD = 57%
	For bedrock coring details refer to Record of Drillhole B504-09.					151						RQD = 99%
	NOTE:		3	RC	REC 100%	150						RQD = 100%
	* Split-Spoon Sampler bouncing.					149						
			4	RC	REC 100%	148						RQD = 100%
			5	RC	REC 100%							
147.0	29.3					147						
	END OF BOREHOLE											

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-09

SHEET 1 OF 1

LOCATION: N 5084121.2 ; E 222583.1

DRILLING DATE: August 6 to 7, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D120

DRILLING CONTRACTOR: WALKER 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										RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diameter Point Load Index (MPa)	RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
				(m)													TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn			10	5	1	0.1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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		Continued from Record of Borehole B504-09		152.93																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
	HW Casing	Slightly weathered to fresh, massive, grey to dark grey, fine to medium grained, faintly porous, strong to very strong GRANITIC GNEISS		23.32	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE		No B504-10		SHEET 2 OF 2		METRIC									
W.P. 09-1111-6014		LOCATION		N 5084119.3 ; E 222562.6		ORIGINATED BY		LK									
DIST 5147-08-01 HWY 69		BOREHOLE TYPE		100 mm I.D. HW Casing, Wash Boring		COMPILED BY		JFC									
DATUM Geodetic		DATE		August 7, 2014		CHECKED BY		MCK/AB									
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	Inferred SAND to GRAVELLY SAND No sample taken between depths of 3.7 m and 21.3 m.																
155.6																	
20.7	Inferred BOULDERS																
155.0																	
21.3	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 21.3 m to 24.8 m. For bedrock coring details refer to Record of Drillhole B504-10.		1	RC	REC 100%											RQD = 62%	
			2	RC	REC 100%											RQD = 95%	
			3	RC	REC 100%											RQD = 62%	
151.5																	
24.8	END OF BOREHOLE NOTE: 1. Soil stratigraphy inferred from field observations during drilling and from information in adjacent boreholes.																

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: WALKER DRILLING

LOGGED: LK
CHECKED: MCK/AB

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-11		SHEET 1 OF 3		METRIC											
W.P. 5147-08-01		LOCATION N 5084118.3 ; E 222594.6		ORIGINATED BY LK													
DIST HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring		COMPILED BY JFC													
DATUM Geodetic		DATE July 31 to August 1, 2014		CHECKED BY MCK/AB													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p W W _L	20 40 60	kN/m ³					
176.3 0.0	WATER SURFACE WATER						176										
							175										
							174										
							173										
172.8 3.5	Inferred ORGANIC SILT No sample taken between depths of 3.5 m and 24.1 m.						172										
							171										
							170										
							169										
							168										
							167										
166.0 10.3	Inferred CLAYEY SILT						166										
							165										
							164										
162.9 13.4	Cobbles encountered at a depth of 12.8 m (Elev. 163.5 m)						163										
							162										

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT		RECORD OF BOREHOLE		No B504-11		SHEET 2 OF 3		METRIC																
W.P. 09-1111-6014		LOCATION		N 5084118.3 ; E 222594.6		ORIGINATED BY		LK																
DIST		HWY 69		BOREHOLE TYPE		100 mm I.D. HW Casing, Wash Boring		COMPILED BY																
JFC		DATE		July 31 to August 1, 2014		CHECKED BY		MCK/AB																
DATUM		Geodetic																						
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																			
<div style="display: flex; justify-content: space-between;"> <div> <p>--- CONTINUED FROM PREVIOUS PAGE ---</p> <p>No sample taken between depths of 3.5 m and 24.1 m.</p> <p>Inferred SAND to GRAVELLY SAND</p> </div> <div> <p>Cobbles and boulders inferred from a depth of 21.7 m (Elev. 154.6 m) to 24.1 m (Elev. 152.2 m)</p> </div> </div>																								
152.2	Granitic Gneiss (BEDROCK)		1	RC	REC 100%																			
24.1	Bedrock cored from depths of 24.1 m to 29.3 m.		2	RC	REC 100%																			
	For bedrock coring details refer to Record of Drillhole B504-11.		3	RC	REC 100%																			
			4	RC	REC 100%																			
147.0	END OF BOREHOLE																							
29.3																								

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-11				SHEET 3 OF 3		METRIC															
W.P. 5147-08-01		LOCATION N 5084118.3 ; E 222594.6				ORIGINATED BY LK																	
DIST HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring				COMPILED BY JFC																	
DATUM Geodetic		DATE July 31 to August 1, 2014				CHECKED BY MCK/AB																	
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															
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>																
NOTE: 1. Soil stratigraphy inferred from field observations during drilling and from information in adjacent boreholes.																							

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-11

SHEET 1 OF 1

LOCATION: N 5084118.3 ; E 222594.6

DRILLING DATE: August 1, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D120

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES		
				DEPTH (m)					TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 cm/sec	10 cm/sec				10 cm/sec	10 cm/sec
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.																	
		Continued from Record of Borehole B504-11		152.19																					
	HW Casing	Slightly weathered, massive, grey, medium to coarse grained, faintly porous, very strong GRANITIC GNEISS		24.11		1									JN,UN,RO,Br IN		3	4							
25															JN,PL,RO SA		1.5	2	4						
															JN,UN,RO,Br IN		3	4							
						2									JN,UN,RO		3	1		15					
															JN,UN,RO SA		3	2							
26															VN,PL,RO		1.5	1							
															JN,PL,RO		1.5	1							
															JN,PL,RO		1.5	1							
27	HQRC August 1, 2014					3									VN,UN,RO SA		3	2							
															VN,UN,RO SA		3	2	15						
															JN,UN,RO		3	1							
															JN,PL,RO JN,UN,RO SA		1.5	1							
28															JN,UN,RO SA		3	2							
															JN,UN,SM SA		2	2							
						4												4							
29															JN,UN,RO		3	1							
		END OF DRILLHOLE		147.04 29.26																					
30																									
31																									
32																									
33																									
34																									

8.11 MPa

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B504-12		SHEET 1 OF 3	METRIC
W.P. <u>5147-08-01</u>	LOCATION <u>N 5084188.2 ; E 222554.7</u>	ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>69</u>	BOREHOLE TYPE <u>100 mm I.D. HW Casing, Wash Boring</u>	COMPILED BY <u>JFC</u>			
DATUM <u>Geodetic</u>	DATE <u>July 16 to 18, 2014</u>	CHECKED BY <u>MCK/AB</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE 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 (%)				
								20 40 60 80 100	20 40 60					
176.3 0.0	WATER SURFACE WATER													
170.8 5.5	ORGANIC SILT Very soft to soft Brown to grey Wet		1	SS	WH									
			2	SS	WH									
			3	SS	WH									
			4	SS	WH									
163.5 12.8	CLAYEY SILT, trace sand Firm Grey Wet		5	SS	WH									
162.0 14.3			6	SS	WH									

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-12		SHEET 2 OF 3		METRIC	
W.P. 5147-08-01		LOCATION N 5084188.2 ; E 222554.7		ORIGINATED BY LK			
DIST _____ HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring		COMPILED BY JFC			
DATUM Geodetic		DATE July 16 to 18, 2014		CHECKED BY MCK/AB			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED			w _p		w	w _L					
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100												
152.5 23.8	SAND, trace to some silt, trace to some gravel Loose to dense Brown to grey Moist to wet Silt pockets encountered at a depth of 21.6 m (Elev. 154.7 m).		7	SS	33															
			8	SS	30															
			9	SS	25															
			10	SS	5															
			11	SS	7															
			12	SS	10															
147.7 28.6	SILT and SAND, trace to some gravel, trace clay Compact to very dense Grey Wet Clay lenses encountered at a depth of 24.7 m (Elev. 151.6 m). Boulder encountered at a depth of 26.1 m (Elev. 150.2 m).		13	SS	12															
			14	SS	-*															
			15	RC	REC 100%															
147.7 28.6	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 28.6 m to 34.2 m. For bedrock coring details refer to Record of Drillhole B504-12.																			
			1	RC	REC 100%															
			2	RC	REC 100%															

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-12				SHEET 3 OF 3		METRIC									
W.P. 5147-08-01		LOCATION N 5084188.2 ; E 222554.7				ORIGINATED BY LK											
DIST _____ HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring				COMPILED BY JFC											
DATUM Geodetic		DATE July 16 to 18, 2014				CHECKED BY MCK/AB											
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 (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100	W _p	W	W _L			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
							20	40	60	80	100	20	40	60			
142.1	Granitic Gneiss (BEDROCK)		2	RC	REC 100%												RQD = 28%
	Bedrock cored from depths of 28.6 m to 34.2 m.		3	RC	REC 100%												RQD = 93%
	For bedrock coring details refer to Record of Drillhole B504-12.		4	RC	REC 100%												RQD = 95%
143																	
144																	
145																	
146																	
147																	
148																	
149																	
150																	
151																	
152																	
153																	
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262																	
263																	
264																	
265																	
266																	

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-12

SHEET 1 OF 1

LOCATION: N 5084188.2 ;E 222554.7

DRILLING DATE: July 17 to 18, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D120

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	RECOVERY				R.Q.D. %	FRACT INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES																							
								TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10°	20°	30°	45°				60°																						
																										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
	HW Casing	Continued from Record of Borehole B504-12		147.69																																											
29		Slightly weathered to fresh, foliated, grey and pink, medium grained, faintly porous GRANITIC GNEISS		28.62	1																																										

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B504-13		SHEET 1 OF 3		METRIC	
W.P. <u>5147-08-01</u>		LOCATION <u>N 5084192.0 ; E 222564.3</u>		ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>100 mm I.D. HW Casing, Wash Boring</u>		COMPILED BY <u>JFC</u>			
DATUM <u>Geodetic</u>		DATE <u>July 18 to 19, 2014</u>		CHECKED BY <u>MCK/AB</u>			

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 (%)		
								20	40	60	80	100			w _p	w	w _L
176.3 0.0	WATER SURFACE WATER																
172.2 4.1	Inferred Organic Silt No sample taken between depths of 4.1 m and 27.6 m.																


Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15



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

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-13				SHEET 3 OF 3		METRIC										
W.P. 5147-08-01		LOCATION N 5084192.0 ; E 222564.3				ORIGINATED BY LK												
DIST _____ HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring				COMPILED BY JFC												
DATUM Geodetic		DATE July 18 to 19, 2014				CHECKED BY MCK/AB												
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									WATER CONTENT (%)	
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60						
	Granitic Gneiss (BEDROCK)		2	RC	REC 100%		146											RQD = 100%
	Bedrock cored from depths of 29.2 m to 32.4 m. For bedrock coring details refer to Record of Drillhole B504-13.		3	RC	REC 98%		145											RQD = 85%
143.9 32.4	END OF BOREHOLE						144											
	NOTE: 1. Soil stratigraphy inferred from field observations during drilling and from information in adjacent boreholes.																	

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-13

SHEET 1 OF 1

LOCATION: N 5084192.0 ; E 222564.3

DRILLING DATE: July 19, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D120

DRILLING CONTRACTOR: WALKER 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)	RECOVERY										R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
																	TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn			T ₁	T ₂	T ₃	T ₄																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
		Continued from Record of Borehole B504-13		147.08																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

Continued Next Page

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

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-14				SHEET 3 OF 3		METRIC													
W.P. 5147-08-01		LOCATION N 5084190.2 ; E 222532.4				ORIGINATED BY LK															
DIST HWY 69		BOREHOLE TYPE 100 mm I.D. HW Casing, Wash Boring				COMPILED BY JFC															
DATUM Geodetic		DATE July 25 to 26, 2014				CHECKED BY MCK/AB															
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													
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 20 40 60 20 40 60 </div>									
NOTE: 1. Soil stratigraphy inferred from field observations during drilling and from information in adjacent boreholes.																					

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B504-14

SHEET 1 OF 1

LOCATION: N 5084190.2 ; E 222532.4

DRILLING DATE: July 25 to 26, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Diedrich D120

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	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
		Continued from Record of Borehole B504-14		150.76									
				25.51									
26	HW Casing	Fresh, foliated, grey and pink, medium to coarse grained, faintly porous, strong to very strong GRANITIC GNEISS			1								7.89 MPa
27	HORC				2								
28					3								5.94 MPa 8.31 MPa (Axial)
29		END OF DRILLHOLE		147.22									
30				29.05									
31													
32													
33													
34													
35													

DEPTH SCALE

1 : 50



LOGGED: LK

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15



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



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

PROJECT		RECORD OF BOREHOLE		No B504-17		SHEET 1 OF 1		METRIC					
W.P.		LOCATION		ORIGINATED BY		TM							
DIST		BOREHOLE TYPE		COMPILED BY		JFC							
DATUM		DATE		CHECKED BY		MCK/AB							
09-1111-6014		N 5084239.6 ; E 222532.9											
5147-08-01		Portable Equipment											
HWY 69		July 24 to 26, 2014											
Geodetic													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
197.1	GROUND SURFACE												
0.0	PEAT (Fibrous)		1	GRAB	-		197						
0.3	COBBLE												
	SAND and GRAVEL, trace silt, trace rootlets and organics		1	RC	REC 100%								RQD = 87%
	Dark brown Moist												
	Granitic Gneiss (BEDROCK)		2	RC	REC 100%		196						RQD = 82%
	Bedrock cored from depths of 0.3 m to 10.0 m.												
	For bedrock coring details refer to Record of Drillhole B504-17.		3	RC	REC 100%		195						RQD = 65%
			4	RC	REC 100%		194						RQD = 93%
			5	RC	REC 100%		193						RQD = 100%
			6	RC	REC 100%		192						RQD = 96%
			7	RC	REC 100%		191						RQD = 89%
			8	RC	REC 100%		190						RQD = 61%
			9	RC	REC 100%		189						RQD = 93%
			10	RC	REC 100%		188						RQD = 100%
187.1	END OF BOREHOLE												
10.0													
NOTE: 1. Water level in open corehole at a depth of 7.3 m below ground surface (Elev. 189.8 m) on the morning of July 27, 2014.													

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: OGS Inc

[illegible]

CHECKED: MCK/AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 11/24/15

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B504-18				SHEET 1 OF 1		METRIC							
W.P. 5147-08-01		LOCATION N 5084237.1 ; E 222542.4				ORIGINATED BY TM									
DIST HWY 69		BOREHOLE TYPE Portable Equipment				COMPILED BY JFC									
DATUM Geodetic		DATE July 24, 2014				CHECKED BY MCK/AB									
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	<div style="display: flex; justify-content: space-between; width: 100%;"> 20 40 60 80 100 </div>		W _p	W	W _L	γ	GR SA SI CL	
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%)					
								20 40 60 80 100		20 40 60					
193.6 0.0	GROUND SURFACE Bedrock Outcrop														

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 11/24/15



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



APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

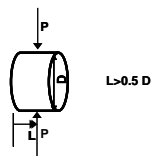
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Foundation Element	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)
South Abutment	B504-06	2	1.6	189.2	Granitic Gneiss	Diametral	84.00	63.00	7.67	107
	B504-06	7	6.3	184.5	Granitic Gneiss	Axial	47.00	63.00	8.34	117
	B504-06	8	7.5	183.3	Granitic Gneiss	Axial	66.00	63.00	6.19	87
	B504-06	8	8.0	182.8	Granitic Gneiss	Diametral	84.00	63.00	7.35	103
South Pier	B504-01	1	13.5	162.0	Granitic Gneiss	Axial	22.97	47.27	14.68	206
	B504-01	3	15.5	160.0	Granitic Gneiss	Diametral	84.70	39.03	23.88	334
	B504-01	4	16.8	158.7	Granitic Gneiss	Axial	22.73	46.79	7.20	101
	B504-09	3	25.1	151.2	Granitic Gneiss	Diametral	120.00	63.00	7.13	100
	B504-09	5	28.8	147.5	Granitic Gneiss	Diametral	105.00	63.00	7.89	110
	B504-10	2	23.3	153.0	Granitic Gneiss	Diametral	69.00	63.00	8.58	120
	B504-11	3	26.1	150.2	Granitic Gneiss	Diametral	90.00	63.00	8.11	114
North Pier	B504-02	2	24.8	150.7	Granitic Gneiss	Axial	23.63	47.52	11.77	165
	B504-02	3	26.7	148.8	Granitic Gneiss	Diametral	60.99	40.19	8.69	122
	B504-02	4	28.4	147.1	Granitic Gneiss	Axial	22.02	47.39	8.94	125
	B504-13	2	29.3	147.0	Granitic Gneiss	Diametral	115.00	63.00	5.48	77
	B504-13	2	29.3	147.0	Granitic Gneiss	Axial	61.00	63.00	7.12	100
	B504-13	2	29.6	146.7	Granitic Gneiss	Diametral	102.00	63.00	5.68	80
	B504-13	2	29.6	146.7	Granitic Gneiss	Axial	63.00	63.00	7.70	108
	B504-13	3	31.7	144.6	Granitic Gneiss	Axial	60.00	63.00	8.27	116
	B504-13	3	31.7	144.6	Granitic Gneiss	Diametral	120.00	63.00	7.60	106
	B504-14	1	25.8	150.5	Granitic Gneiss	Diametral	115.00	63.00	7.89	110
	B504-14	2	27.7	148.6	Granitic Gneiss	Diametral	96.00	63.00	5.94	83
	B504-14	2	27.7	148.6	Granitic Gneiss	Axial	52.00	63.00	8.31	116
North Abutment	B504-17	1	1.2	195.9	Granitic Gneiss	Diametral	100.00	56.00	2.47	35
	B504-17	1	1.4	195.7	Granitic Gneiss	Axial	41.00	56.00	9.01	126
	B504-17	3	2.8	194.3	Granitic Gneiss	Axial	40.00	56.00	9.23	129
	B504-17	5	5.1	192.0	Granitic Gneiss	Diametral	107.00	56.00	6.26	88

⁽¹⁾ $Is_{90} \times K$, 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 $K = 14$ has been determined based on UCS tests for both SBL and NBL bridges.

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)

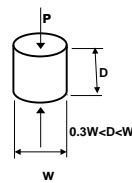


TABLE B2-1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
KEY RIVER NBL BRIDGE
HIGHWAY 69 GWP 5404-05-00; WP 5147-08-01

Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B504-02	25.6	149.9	Granitic Gneiss	47.2	94.1
B504-06	6.9	183.9	Granitic Gneiss	63.1	96.6
B504-09	25.5	150.8	Granitic Gneiss	63.1	109.4
B504-09	28.7	147.6	Granitic Gneiss	63.1	177.7
B504-17	1.6	195.5	Granitic Gneiss	56.4	88.1
B504-17	6.4	190.7	Granitic Gneiss	56.4	101.6

Compiled By: ABReviewed By: JMAC

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

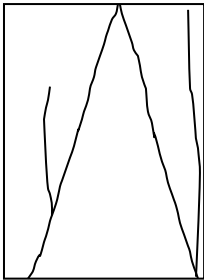
SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	2
BOREHOLE NUMBER	B504-02	SAMPLE DEPTH, m	25.52-25.69
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.33
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	11.02	WATER CONTENT, (specimen) %	0.24
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m ³	26.16
SAMPLE AREA, cm ²	17.50	DRY UNIT WT., kN/m ³	26.09
SAMPLE VOLUME, cm ³	192.82	SPECIFIC GRAVITY	-
WET WEIGHT, g	514.46	VOID RATIO	-
DRY WEIGHT, g	513.23		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	94.1
REMARKS:	DATE:		12/13/2012
PREPARED BY: AB	REVIEWED BY: JPD/JMAC		

TABLE B2-3
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

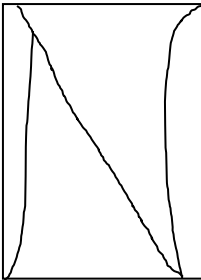
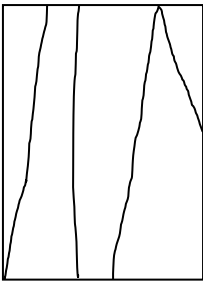
SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	7
BOREHOLE NUMBER	B504-06	SAMPLE DEPTH, m	6.88-7.10
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.29
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	14.44	WATER CONTENT, (specimen) %	0.11
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m ³	26.30
SAMPLE AREA, cm ²	31.30	DRY UNIT WT., kN/m ³	26.27
SAMPLE VOLUME, cm ³	451.83	SPECIFIC GRAVITY	-
WET WEIGHT, g	1212.10	VOID RATIO	-
DRY WEIGHT, g	1210.77		
VISUAL INSPECTION	FAILURE SKETCH		
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	96.6
REMARKS:	DATE:		04/22/15
PREPARED BY:	AB	REVIEWED BY:	JMAC

TABLE B2-4
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	3
BOREHOLE NUMBER	B504-09	SAMPLE DEPTH, m	25.40-25.65

TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	14.23	WATER CONTENT, (specimen) %	0.20
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m ³	29.17
SAMPLE AREA, cm ²	31.30	DRY UNIT WT., kN/m ³	29.11
SAMPLE VOLUME, cm ³	445.48	SPECIFIC GRAVITY	-
WET WEIGHT, g	1325.50	VOID RATIO	-
DRY WEIGHT, g	1322.85		

VISUAL INSPECTION	FAILURE SKETCH
	

TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	109.4

REMARKS: _____ DATE: 04/23/15

PREPARED BY: AB REVIEWED BY: JMAC

TABLE B2-5
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1111-6014	RUN NUMBER	5
BOREHOLE NUMBER	B504-09	SAMPLE DEPTH, m	28.57-28.78

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.30

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.52	WATER CONTENT, (specimen) %	0.15
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m ³	29.21
SAMPLE AREA, cm ²	31.29	DRY UNIT WT., kN/m ³	29.17
SAMPLE VOLUME, cm ³	454.19	SPECIFIC GRAVITY	-
WET WEIGHT, g	1353.30	VOID RATIO	-
DRY WEIGHT, g	1351.27		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	177.7
----------------------	---	---------------------------	-------

REMARKS:	DATE:	04/22/15
----------	-------	----------

PREPARED BY:	AB	REVIEWED BY: JMAC
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TABLE B2-6
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

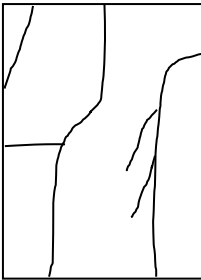
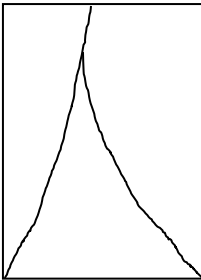
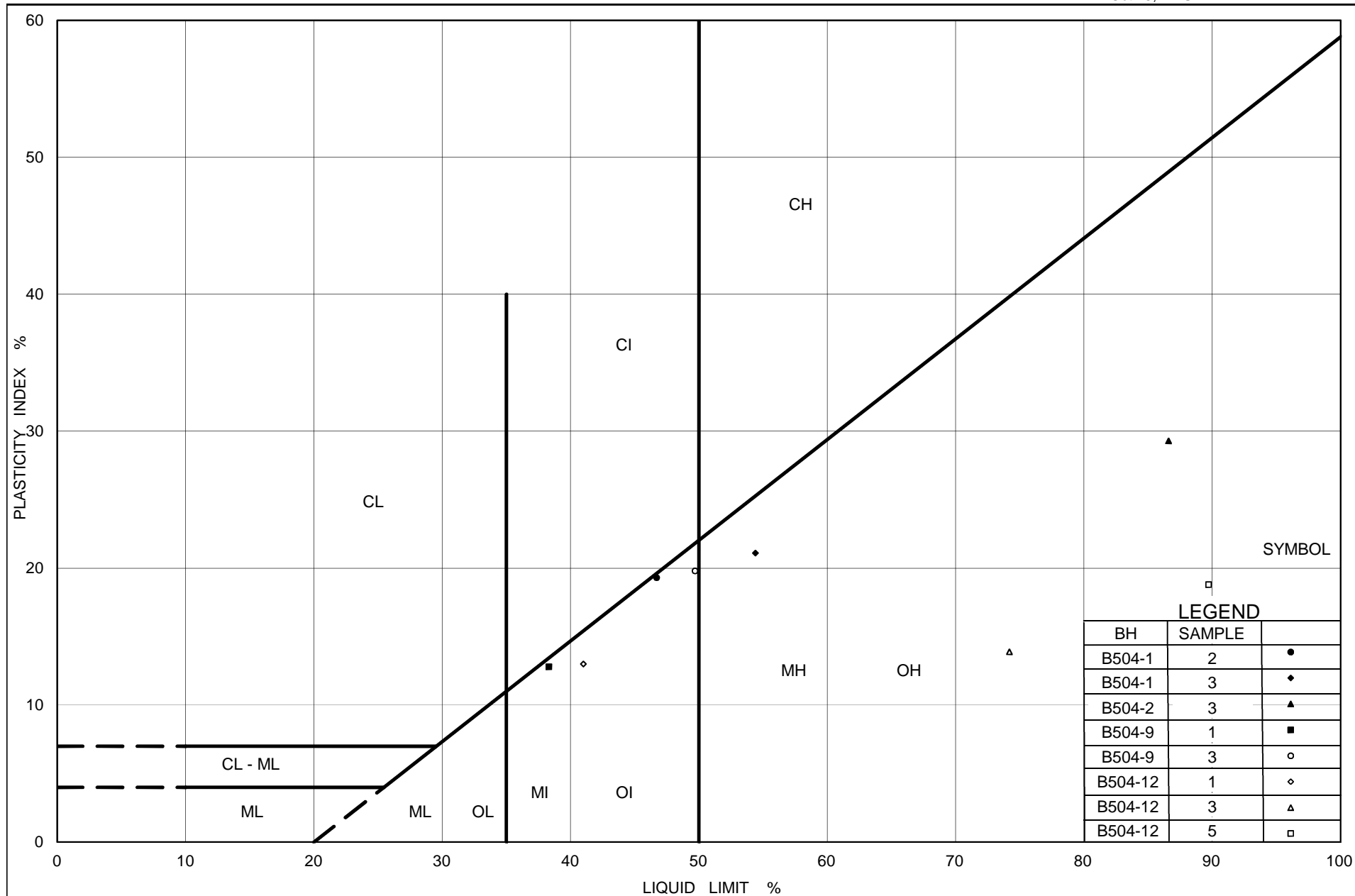
SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	2
BOREHOLE NUMBER	B504-17	SAMPLE DEPTH, m	1.54-1.75
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	12.49	WATER CONTENT, (specimen) %	0.06
SAMPLE DIAMETER, cm	5.64	UNIT WEIGHT, kN/m ³	26.67
SAMPLE AREA, cm ²	25.00	DRY UNIT WT., kN/m ³	26.66
SAMPLE VOLUME, cm ³	312.34	SPECIFIC GRAVITY	-
WET WEIGHT, g	849.80	VOID RATIO	-
DRY WEIGHT, g	849.29		
VISUAL INSPECTION	FAILURE SKETCH		
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	88.1
REMARKS:	DATE:		04/22/15
PREPARED BY:	AB	REVIEWED BY:	JMAC

TABLE B2-7
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	7
BOREHOLE NUMBER	B504-17	SAMPLE DEPTH, m	6.32-6.50
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	12.67	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	5.64	UNIT WEIGHT, kN/m ³	27.15
SAMPLE AREA, cm ²	24.95	DRY UNIT WT., kN/m ³	27.12
SAMPLE VOLUME, cm ³	316.09	SPECIFIC GRAVITY	-
WET WEIGHT, g	875.40	VOID RATIO	-
DRY WEIGHT, g	874.61		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	101.6
REMARKS:		DATE: 04/22/15	
PREPARED BY:	AB	REVIEWED BY:	JMAC



Ministry of Transportation

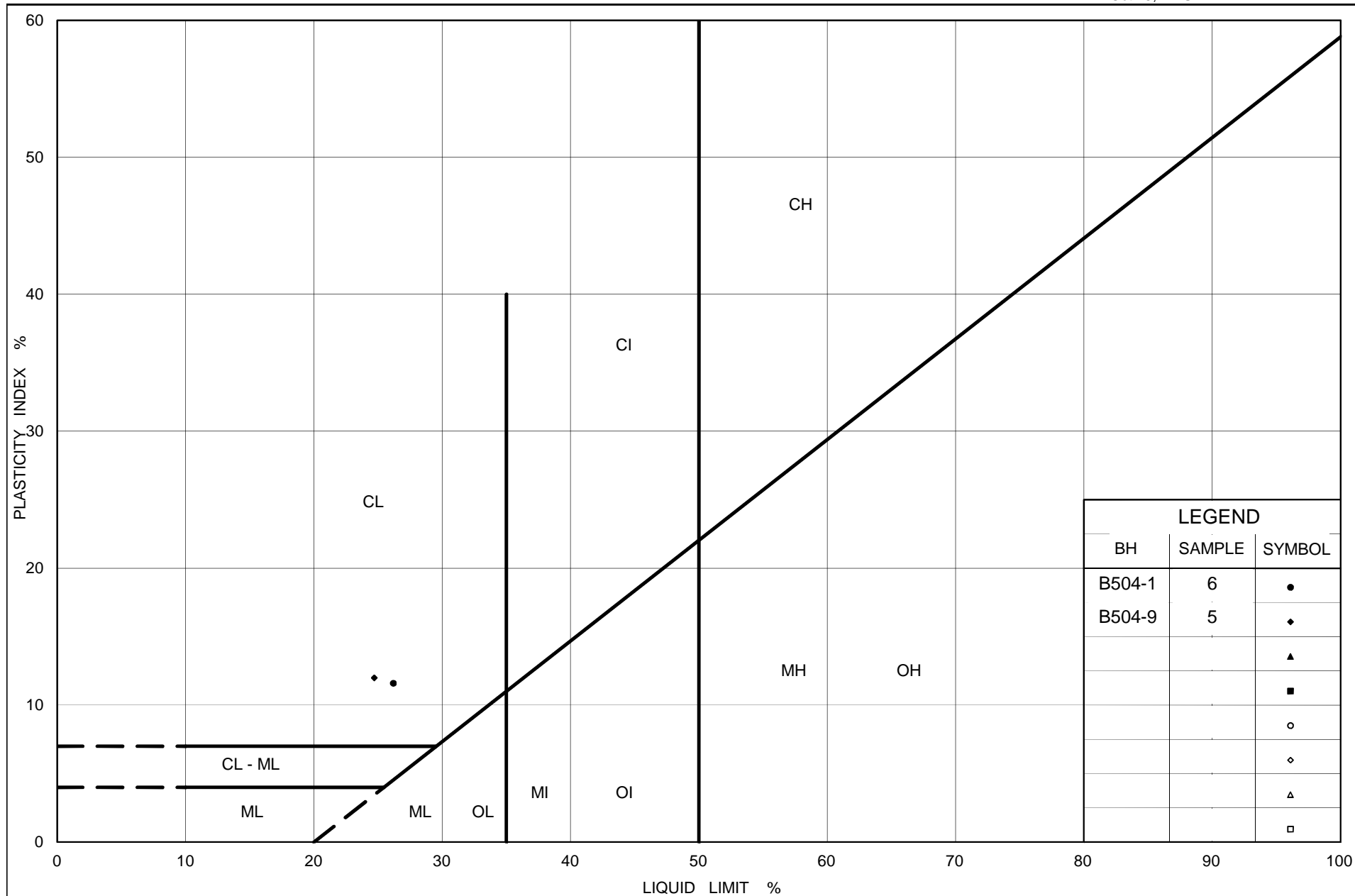
Ontario

PLASTICITY CHART Organic Silt

Figure No. B1A

Project No. 09-1111-6014

Checked By: AB



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt

Figure No. B1B

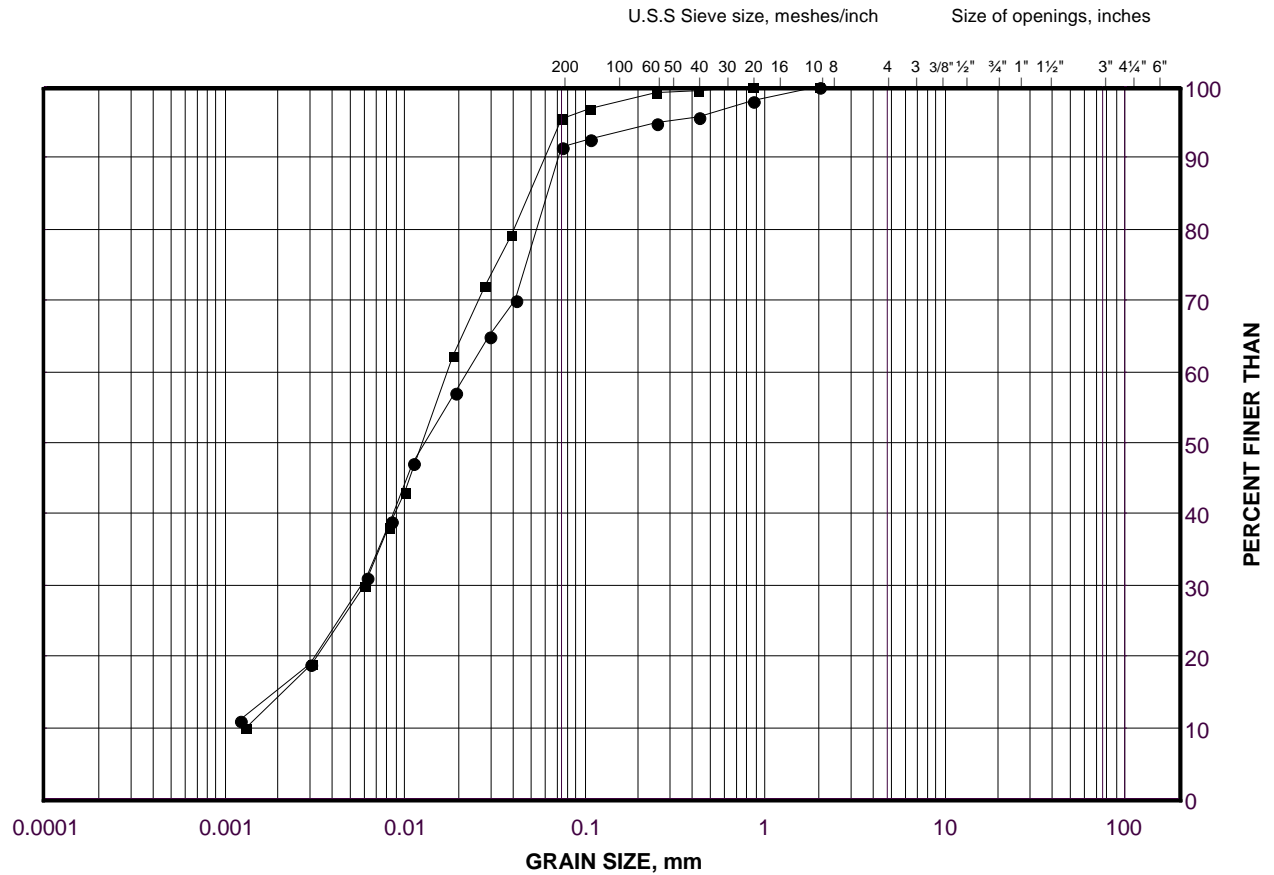
Project No. 09-1111-6014

Checked By: AB

GRAIN SIZE DISTRIBUTION

Organic Silt

FIGURE B2A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B504-02	3	166.4
■	B504-01	3	169.7

Project Number: 09-1111-6014

Checked By: AB

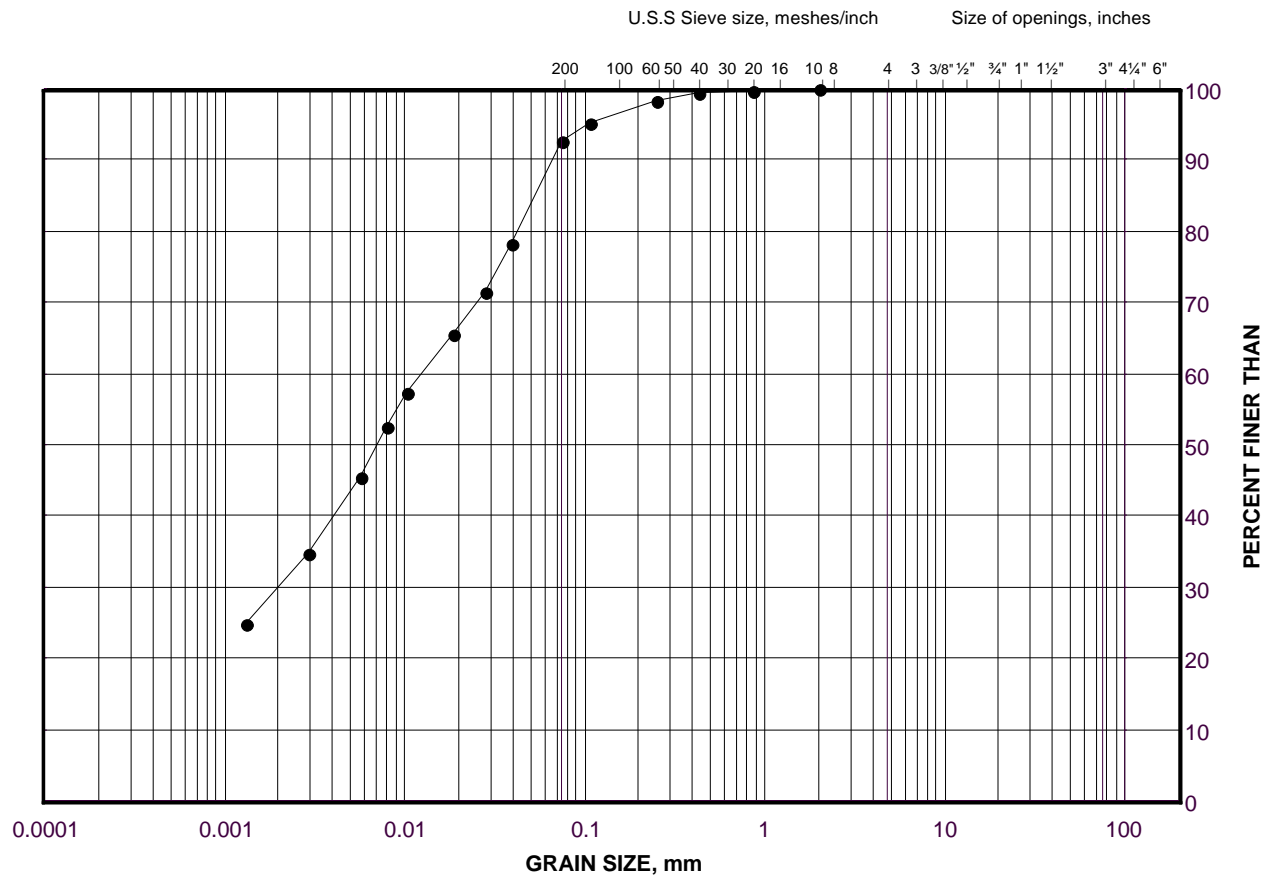
Golder Associates

Date: 30-Apr-15

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B2B



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B504-01	6	164.8

Project Number: 09-1111-6014

Checked By: AB

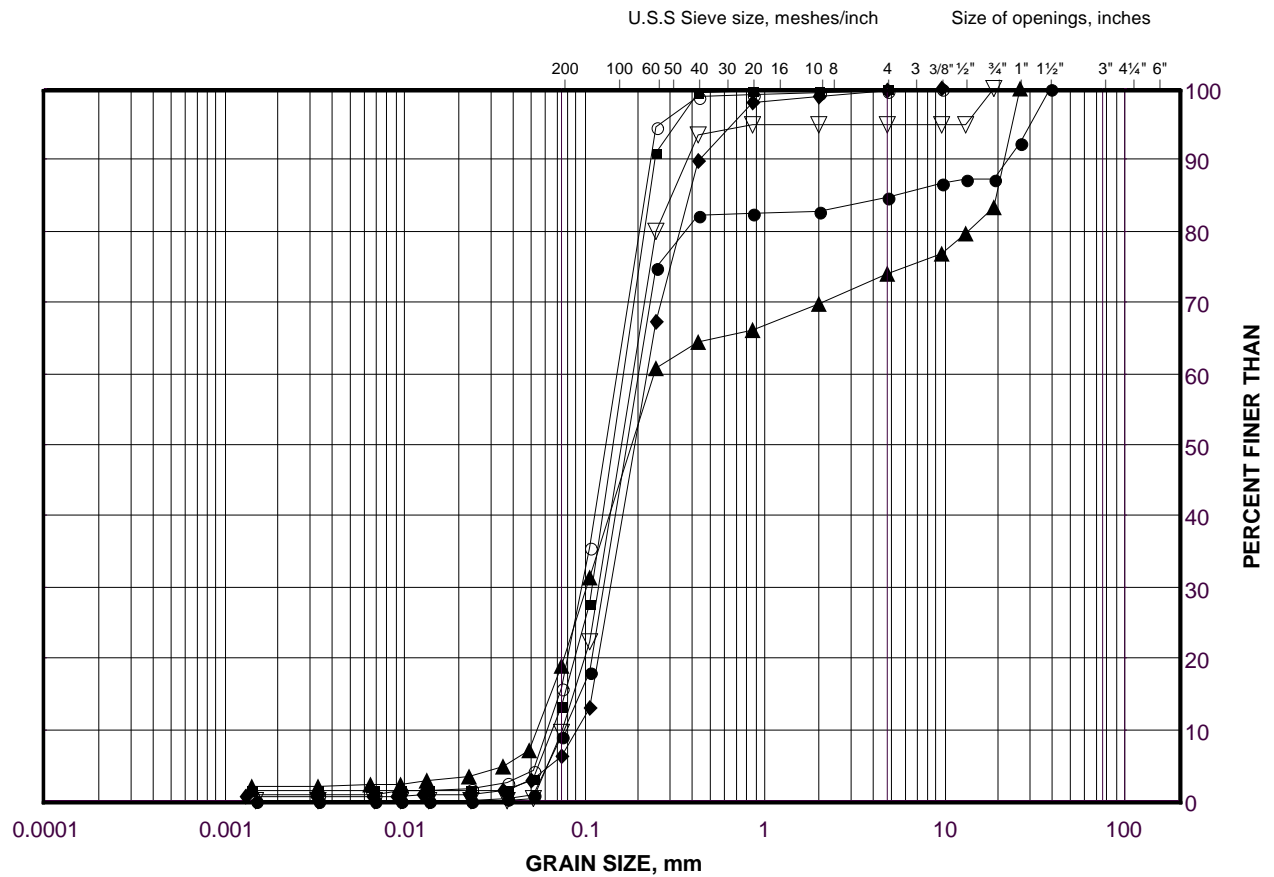
Golder Associates

Date: 30-Apr-15

GRAIN SIZE DISTRIBUTION

Sand to Gravelly Sand

FIGURE B3A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B504-12	10	156.2
■	B504-09	10	157.8
◆	B504-02	10	155.1
▲	B504-09	13	153.2
▽	B504-12	8	159.3
○	B504-09	8	160.8

Project Number: 09-1111-6014

Checked By: AB

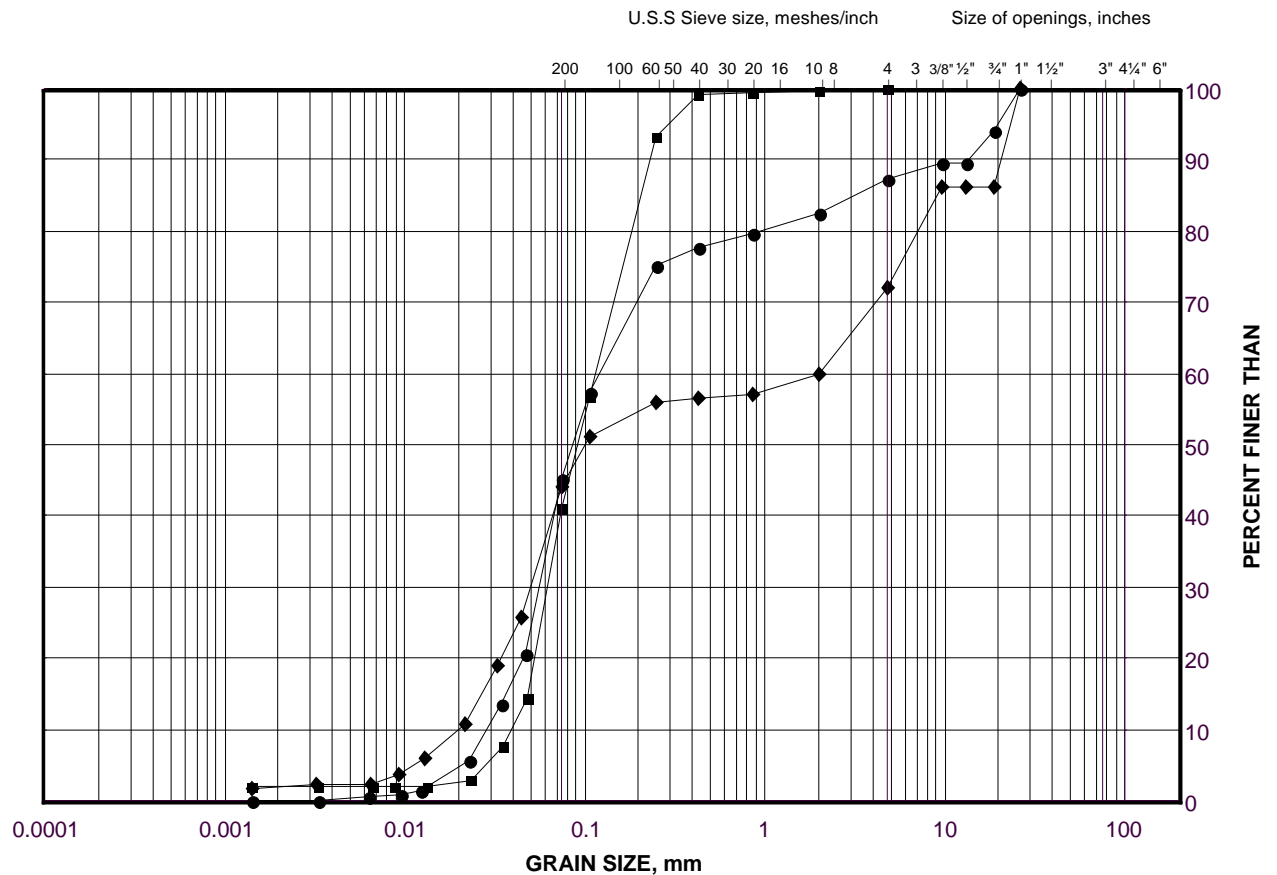
Golder Associates

Date: 30-Apr-15

GRAIN SIZE DISTRIBUTION

Gravelly Sandy Silt to Silt and Sand

FIGURE B3B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B504-12	13	151.7
■	B504-12	16	148.2
◆	B504-02	6	161.8

Project Number: 09-1111-6014

Checked By: AB

Golder Associates

Date: 30-Apr-15

Borehole B504-01



Box 1: 13.16 m – 17.31 m

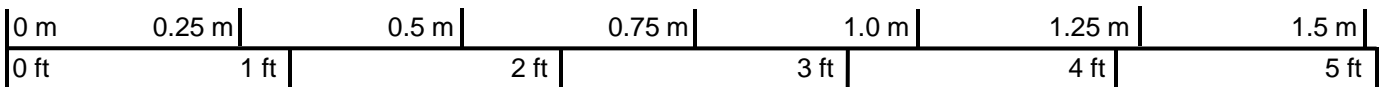
Borehole B504-02




Box 1: 24.47 m – 27.89 m



Box 2: 27.89 m – 29.47 m



Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01			
TITLE		Bedrock Core Photograph – Borehole B504–01 & Borehole B504–02			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	MCK	MAR 15	SCALE NTS REV.
		CADD	-- --		
		CHECK	AB	APR 15	FIGURE B4
		REVIEW	JMAC	APR 15	

Borehole B504-06



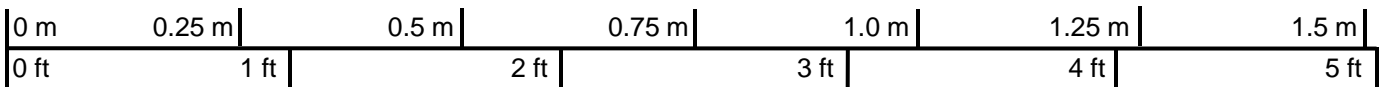
Box 1: 0.00 m – 2.45m




Box 2: 2.45 m – 5.26 m



Box 3: 5.26 m – 8.08 m



Scale

PROJECT			KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01		
TITLE			Bedrock Core Photograph – Borehole B504-06		
			PROJECT No. 09-1111-6014		FILE No. ----
			DESIGN	MCK	MAR 15
			CADD	-- --	
			CHECK	AB	APR 15
			REVIEW	JMAC	APR 15
			SCALE NTS REV.		
			FIGURE B5		

Borehole B504-09



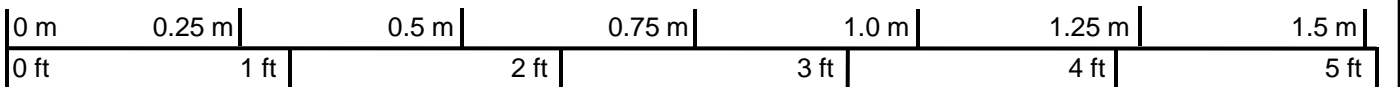
Box 1: 23.32 m – 25.60 m




Box 2: 25.60 m – 27.51 m



Box 3: 27.51 m – 29.26 m



Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01			
TITLE		Bedrock Core Photograph – Borehole B504-09			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	MCK	MAR 15	SCALE NTS REV.
		CADD	--		
		CHECK	AB	APR 15	FIGURE B6
		REVIEW	JMAC	APR 15	

Borehole B504-10



Box 1: 21.31 m – 23.57 m



Box 2: 23.57 m – 24.84 m

Borehole B504-11




Box 1: 24.11 m – 26.70 m



Box 2: 26.70 m – 29.26 m

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	

Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01			
TITLE		Bedrock Core Photograph – Borehole B504–10 & Borehole B504–11			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	MCK	MAR 15	SCALE NTS REV.
		CADD	--		
		CHECK	AB	APR 15	FIGURE B7
		REVIEW	JMAC	APR 15	

Borehole B504-12



Box 1: 26.12 m – 28.78 m
(Boulder: 26.12 to 27.19 m, Silt and Sand: 27.19 m to 28.62 m, Top of Bedrock at 28.62 m)



Box 2: 28.78 m – 31.09 m




Box 3: 31.09 m – 33.04 m



Box 4: 33.04 m – 34.23 m

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	

Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01		
TITLE		Bedrock Core Photograph – Borehole B504-12		
		PROJECT No. 09-1111-6014		FILE No. ----
		DESIGN	MCK	MAR 15
		CADD	-- --	
		CHECK	AB	APR 15
		REVIEW	JMAC	APR 15
		SCALE NTS REV.		
		FIGURE B8		

Borehole B504-13

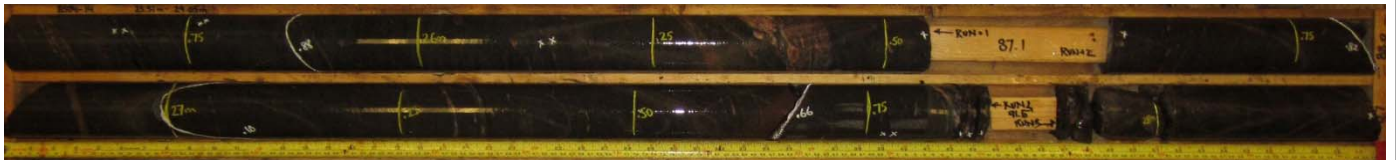


Box 1: 27.60 m – 30.04 m
(Cobbles and Boulders: 27.60, Top of Bedrock: 29.20 m)



Box 2: 30.04 m – 32.40 m

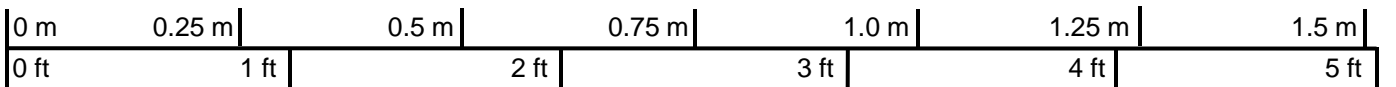
Borehole B504-14




Box 1: 25.51 m – 28.22 m



Box 2: 28.22 m – 29.05 m



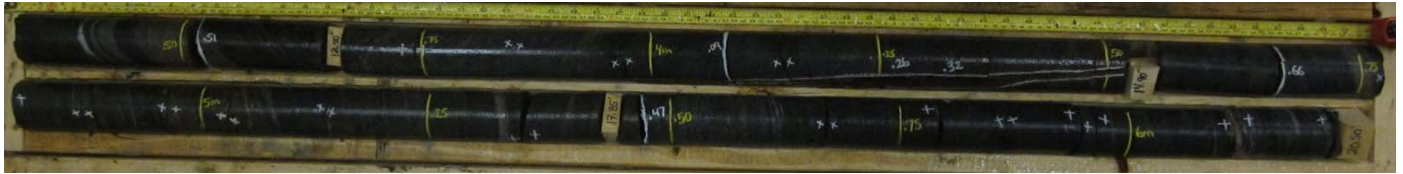
Scale

PROJECT			KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01		
TITLE			Bedrock Core Photograph – Borehole B504–13 & Borehole B504–14		
			PROJECT No. 09-1111-6014		FILE No. ----
			DESIGN	MCK	MAR 15
			CADD	--	
			CHECK	AB	APR 15
			REVIEW	JMAC	APR 15
			SCALE NTS REV.		
			FIGURE B9		

Borehole B504-17



Box 1: 0.34 m – 3.29 m



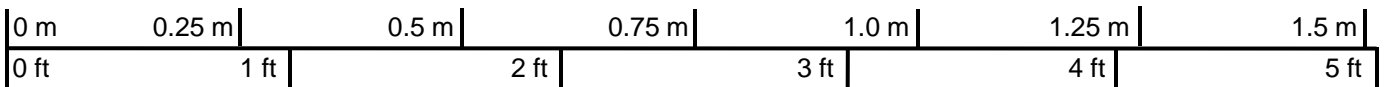
Box 2: 3.29 m – 6.25 m




Box 3: 6.25 m – 9.22 m



Box 4: 9.22 m – 9.98 m



Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 5404-05-00; WP 5147-08-01		
TITLE		Bedrock Core Photograph – Borehole B504-17		
		PROJECT No. 09-1111-6014		FILE No. ----
		DESIGN	MCK	MAR 15
		CADD	-- --	
		CHECK	AB	APR 15
		REVIEW	JMAC	APR 15
		SCALE NTS REV.		
		FIGURE B10		



APPENDIX C

Drilling Photographs and Underwater Locates



Photograph 1: Drilling with portable equipment at SBL north abutment



Photograph 2: Drilling with portable equipment at SBL south abutment

PROJECT					
HIGHWAY 69 - KEY RIVER NBL AND SBL					
TITLE					
Drill Setup Photographs					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	MCK	MAR 15	SCALE	NTS	REV.
CADD	-- --		FIGURE C1		
CHECK	AB	APR 15			
REVIEW	JMAC	APR 15			



20140728 - Key River - Drilling
over BH 503-10 SBL.



Photograph 1: Drilling from barge at SBL south pier

20140824 - Key River - steel posts
welded and rope installed around
the barge.



Photograph 2: Drilling from barge at SBL north pier

PROJECT

HIGHWAY 69 - KEY RIVER NBL AND SBL

TITLE

Drill Setup Photographs



**Golder
Associates**

PROJECT No. 09-1111-6014

FILE No. ----

DESIGN

MCK

MAR 15

SCALE

NTS

REV.

CADD

-- --

CHECK

AB

APR 15

REVIEW

JMAC

APR 15


FIGURE C2



Photograph 1: Key River ASI Group Diver Locates first buoy on west side of river



Photograph 2: Key River ASI Group - Diver out of Water

PROJECT						
HIGHWAY 69 - KEY RIVER NBL AND SBL						
TITLE						
Underwater Locates Photographs						
	PROJECT No. 09-1111-6014			FILE No. ----		
	DESIGN	MCK	MAR 15	SCALE	NTS	REV.
	CADD	-- --				
	CHECK	AB	APR 15	FIGURE C3		
	REVIEW	JMAC	APR 15			



APPENDIX D

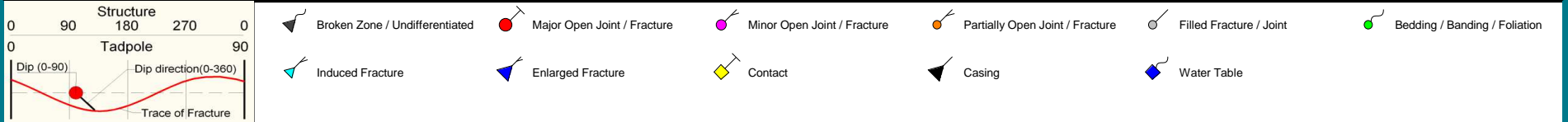
Geophysical Logs, Bedrock Outcrop Photographs and Structural Analysis



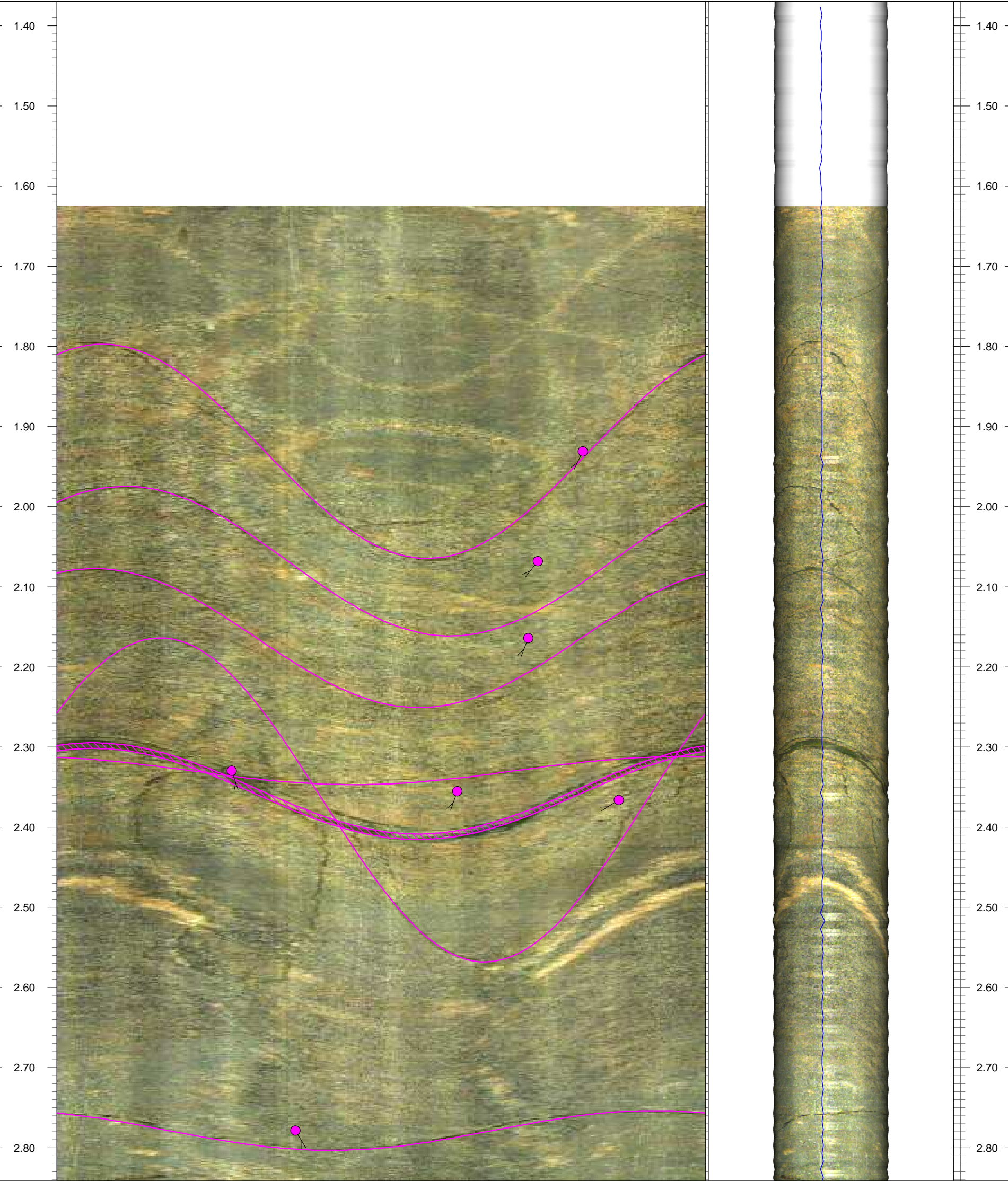
GEOPHYSICAL RECORD OF BOREHOLE: B504-06

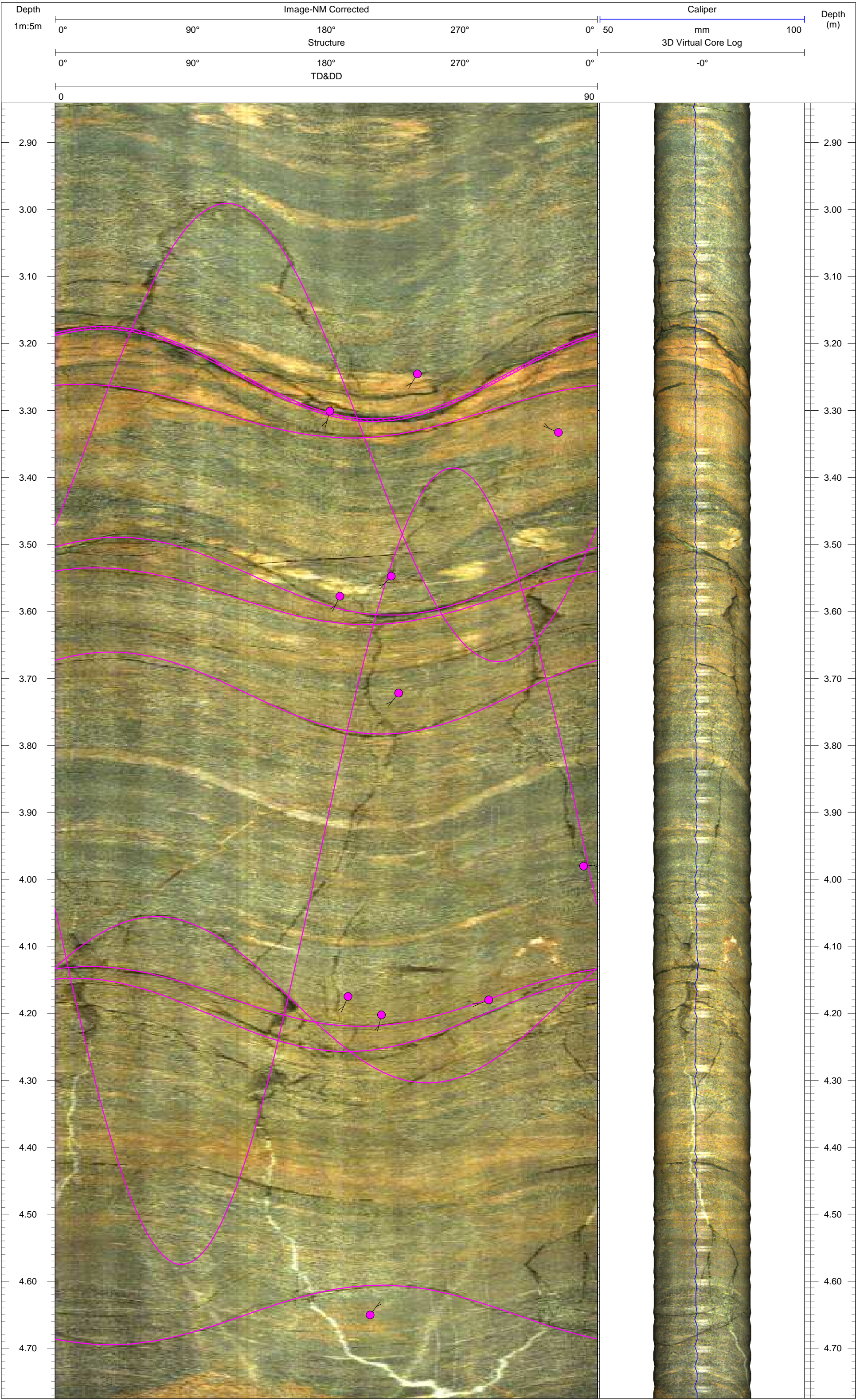
Project Number: 09-1111-6014
Client: MTO
Date: August 2014

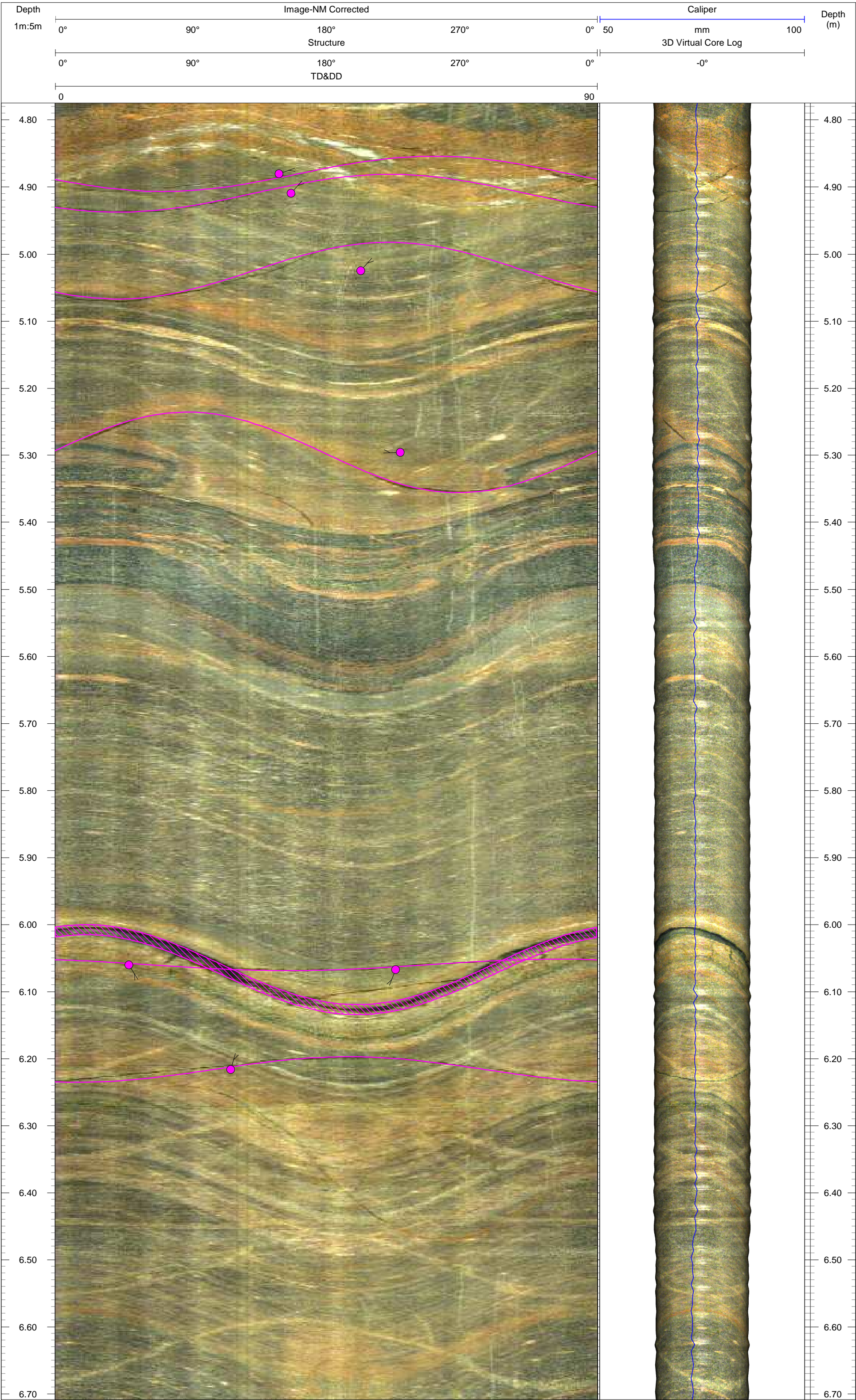
Datum:	WGS84, UTM Zone 17N	Elevation:	190.822 m asl	Borehole Diameter:	73 mm	Water Level:	N/A	Location:	South Abuttment, NBL
Easting:	222,603.680 m	Depth Reference:	"0" at Ground	Casing Diameter:	N/A	Borehole Inclination:	Vertical	Log Date:	7-Aug-14
Northing:	5,084,069.756 m	Drilled Depth:	7.97 m bgs	Casing Depth:	N/A	Borehole Azimuth:	N/A	Logged By:	AR

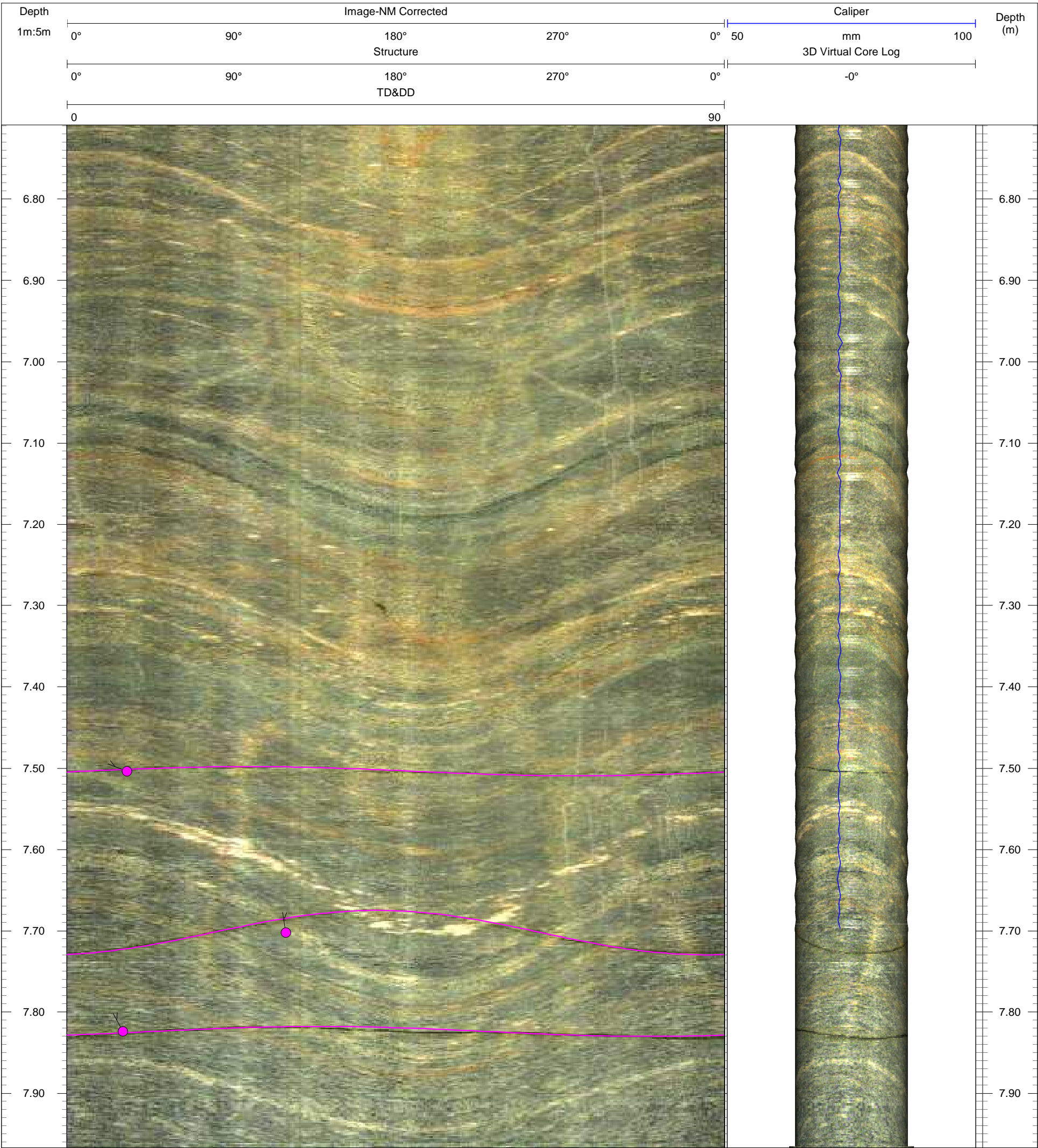


Notes:











GEOPHYSICAL RECORD OF BOREHOLE: B504-17

Project Number: 09-1111-6014
Client: MTO
Date: August 2014

Datum:	WGS84, UTM Zone 17N	Elevation:	197.085 m asl	Borehole Diameter:	67 mm	Water Level:	N/A	Location:	North Abuttment, NBL
Easting:	222,532.942 m	Depth Reference:	"0" at Ground	Casing Diameter:	N/A	Borehole Inclination:	Vertical	Log Date:	8-Aug-14
Northing:	5,084,239.648 m	Drilled Depth:	8.80 m bgs	Casing Depth:	N/A	Borehole Azimuth:	N/A	Logged By:	AR

0901802700

090

Dip (0-90)

Dip direction(0-360)

Trace of Fracture

Broken Zone / Undifferentiated

Major Open Joint / Fracture

Minor Open Joint / Fracture

Partially Open Joint / Fracture

Filled Fracture / Joint

Bedding / Banding / Foliation

Induced Fracture

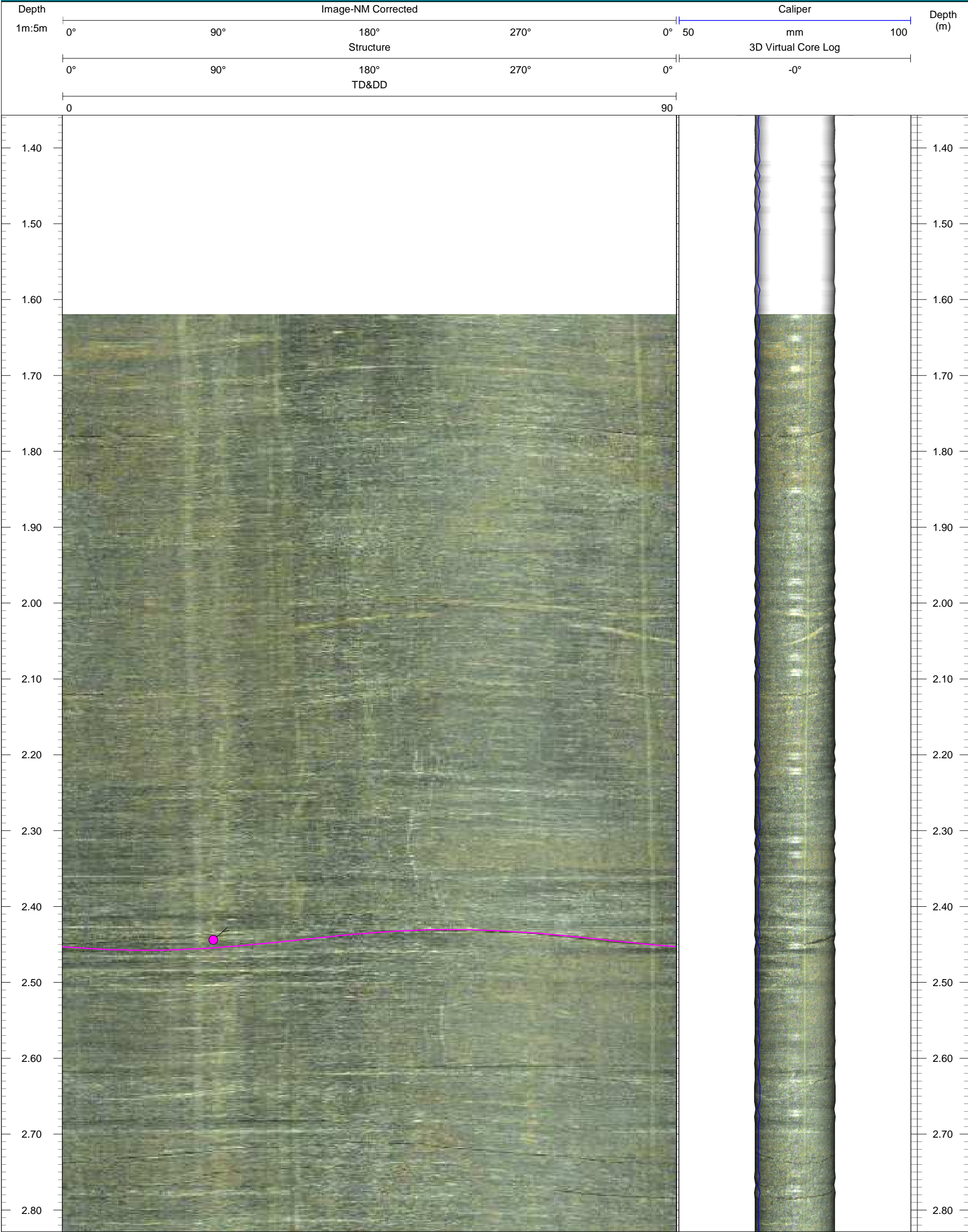
Enlarged Fracture

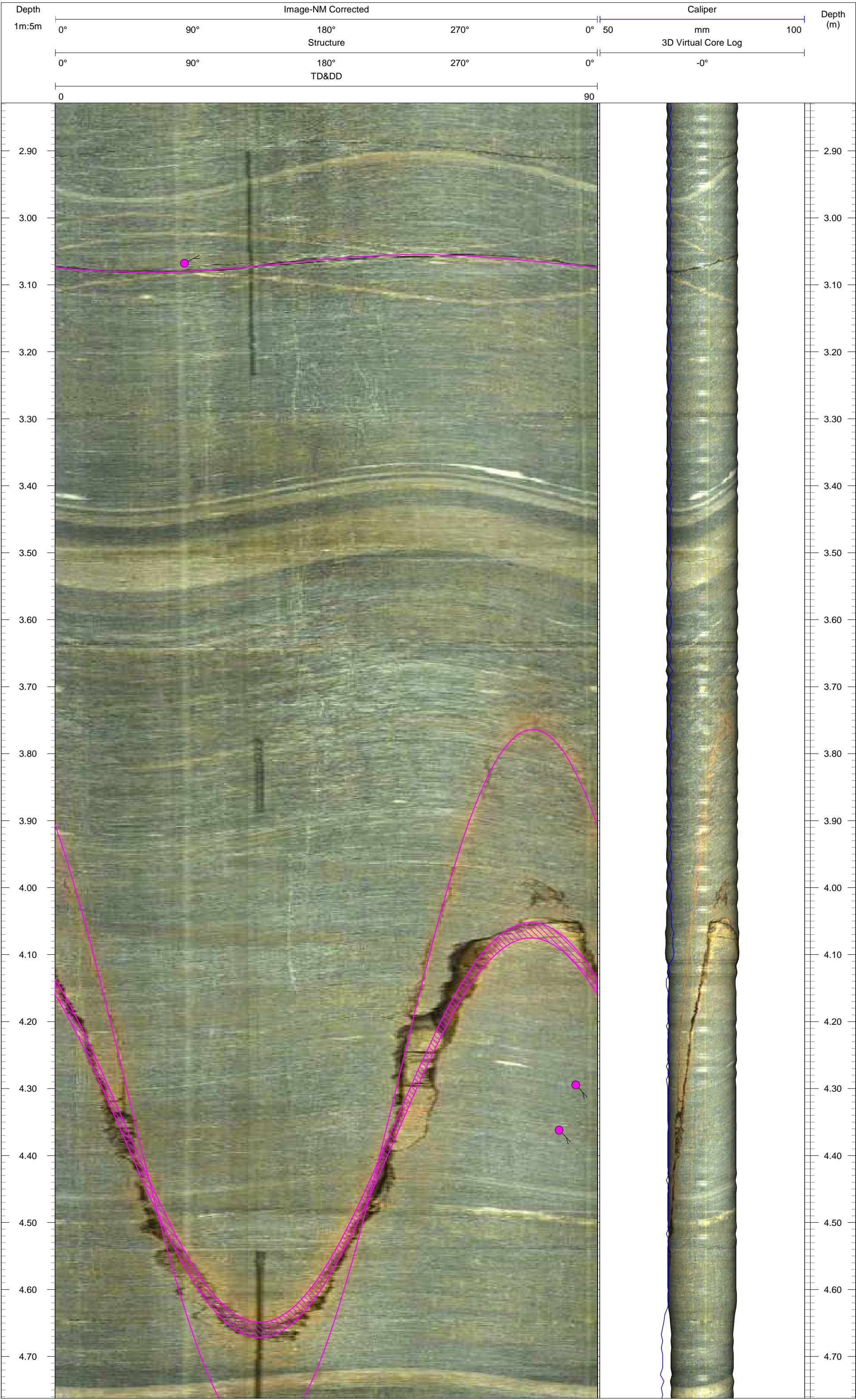
Contact

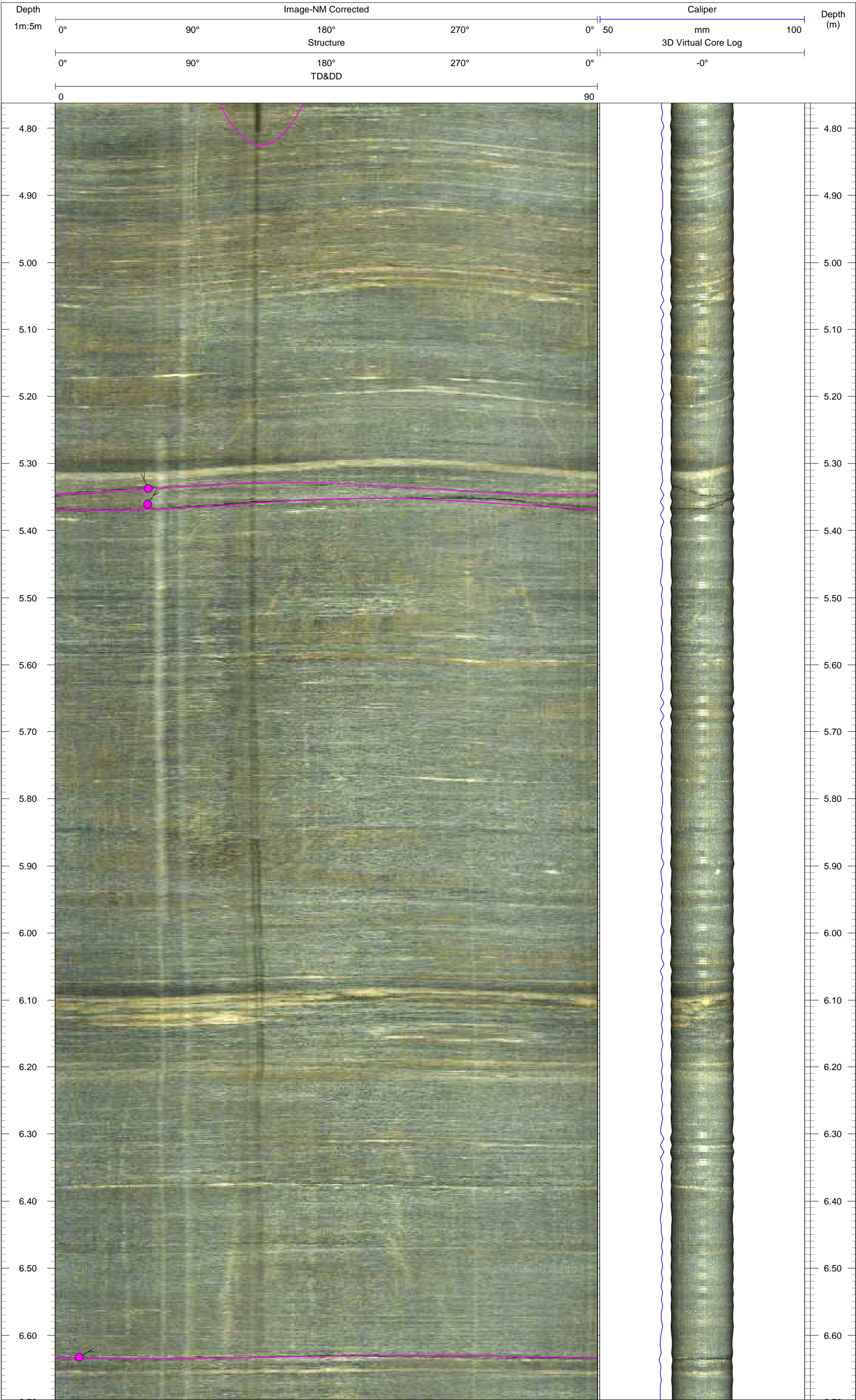
Casing

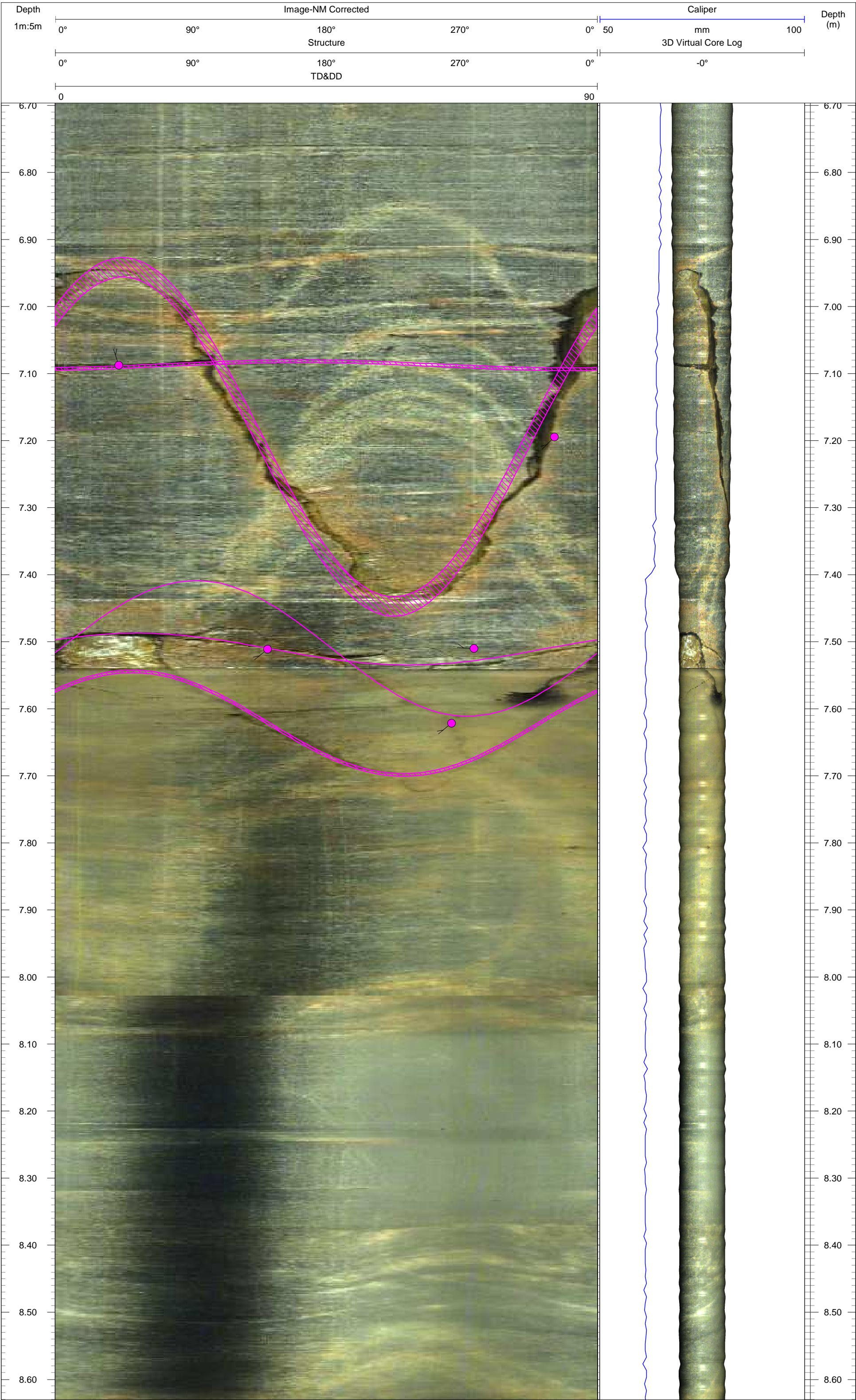
Water Table

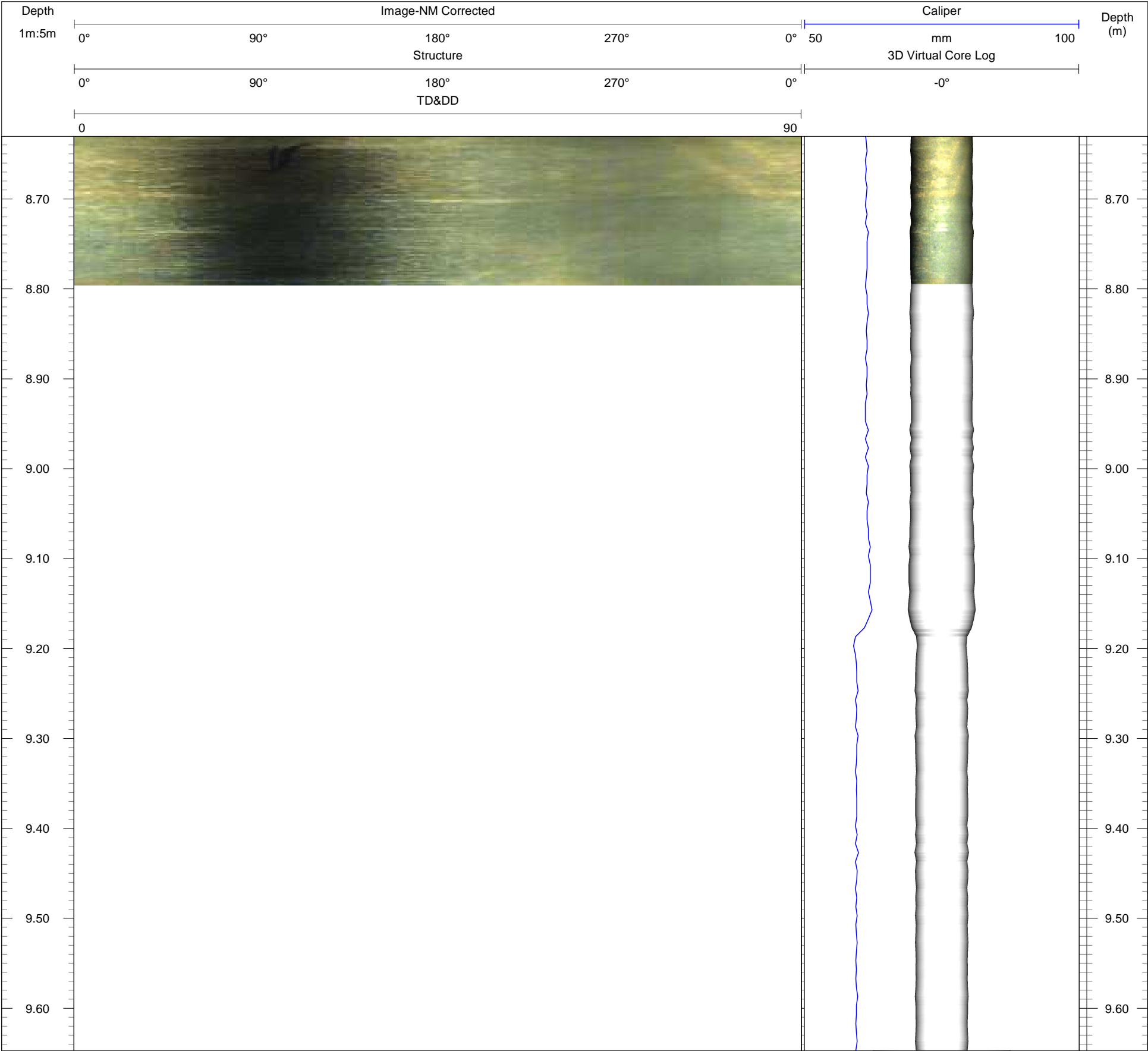
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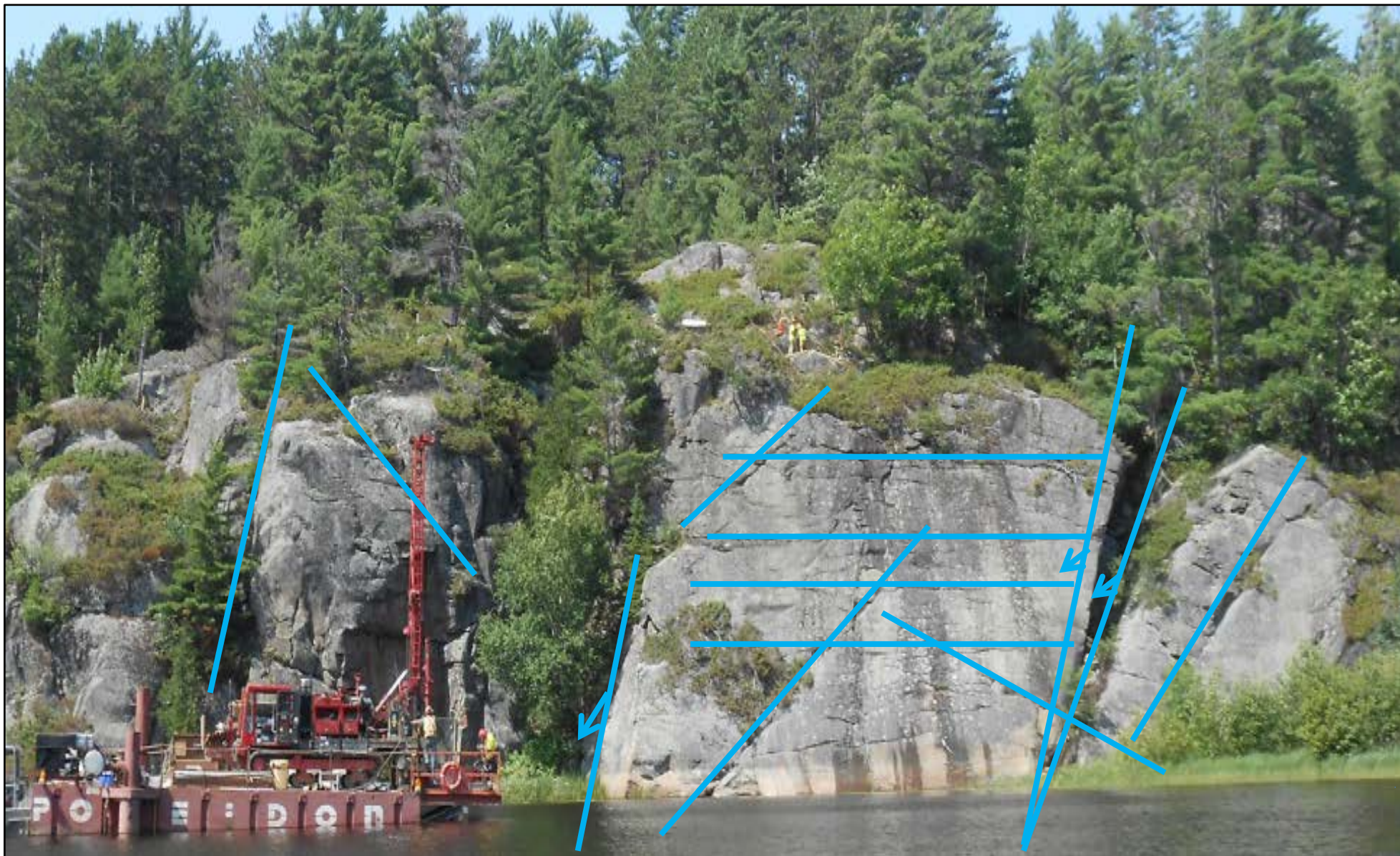












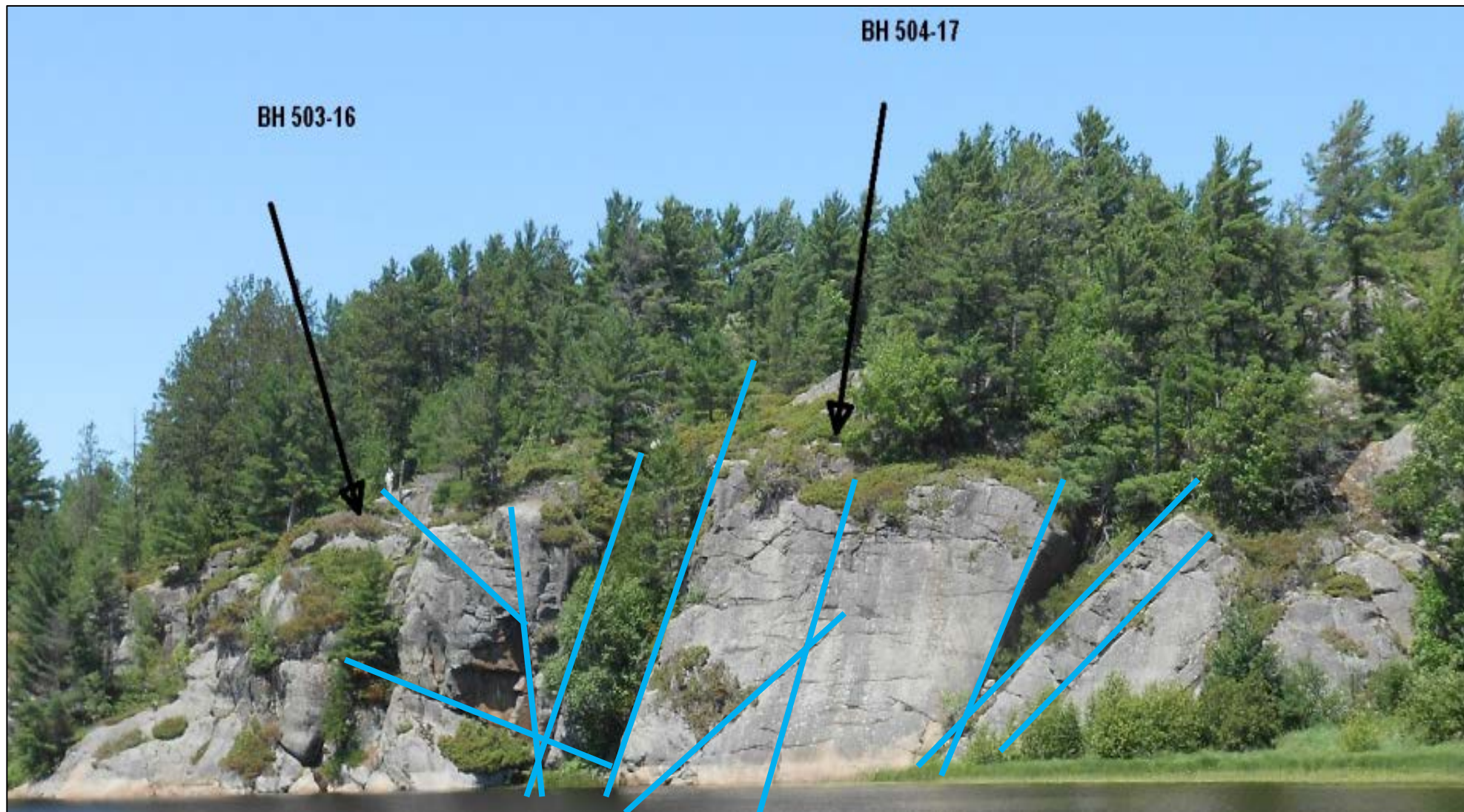
North Wall of Bedrock Outcrop at NBL and SBL Structures

TITLE

**KEY RIVER BRIDGE – ABUTMENT AREAS
JOINTS AND FEATURES
NORTH ABUTMENTS**



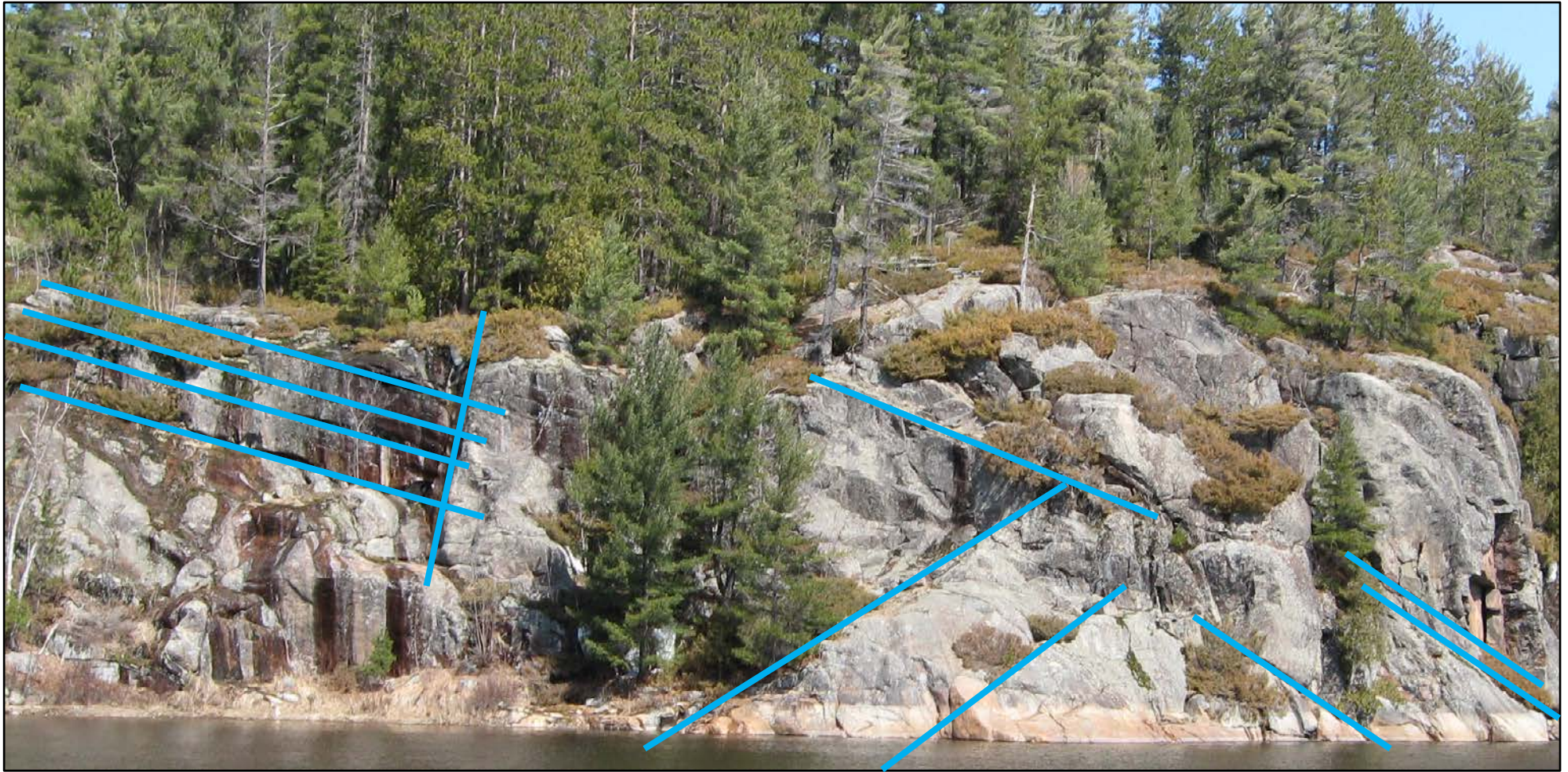
PROJECT No. 09-1111-6014			FILE No. ----	
DESIGN	AB	April 2015	SCALE AS SHOWN	REV.
CADD	--			
CHECK	MT/JPD	April 2015	FIGURE D1	
REVIEW				



North Wall of Bedrock Outcrop at NBL and SBL Structures

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES NORTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D2		
CHECK	MT/JPD	April 2015			
REVIEW					





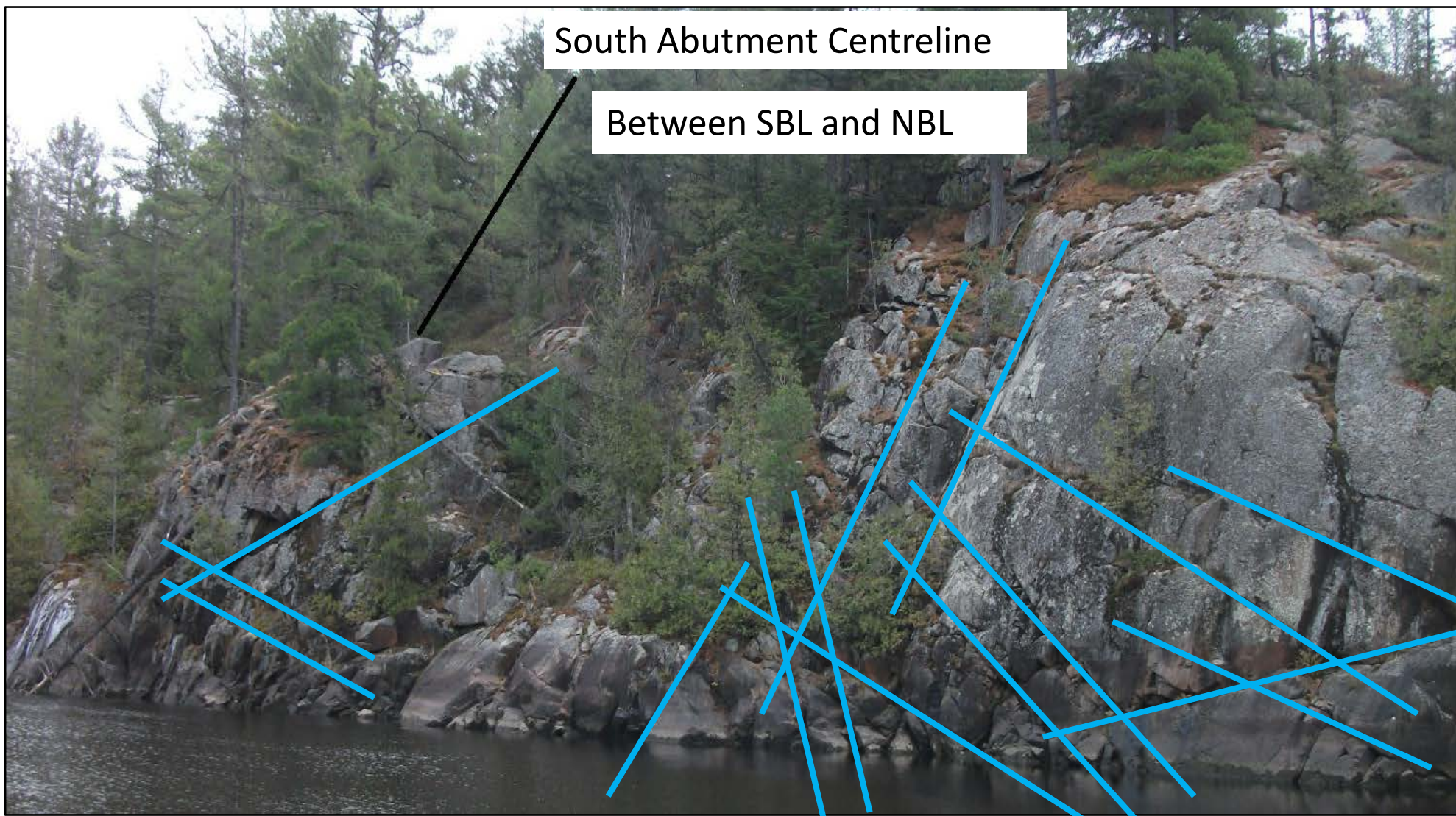
North Wall of Bedrock Outcrop

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES NORTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D3		
CHECK	MT/JPD	April 2015			
REVIEW					



South Abutment Centreline

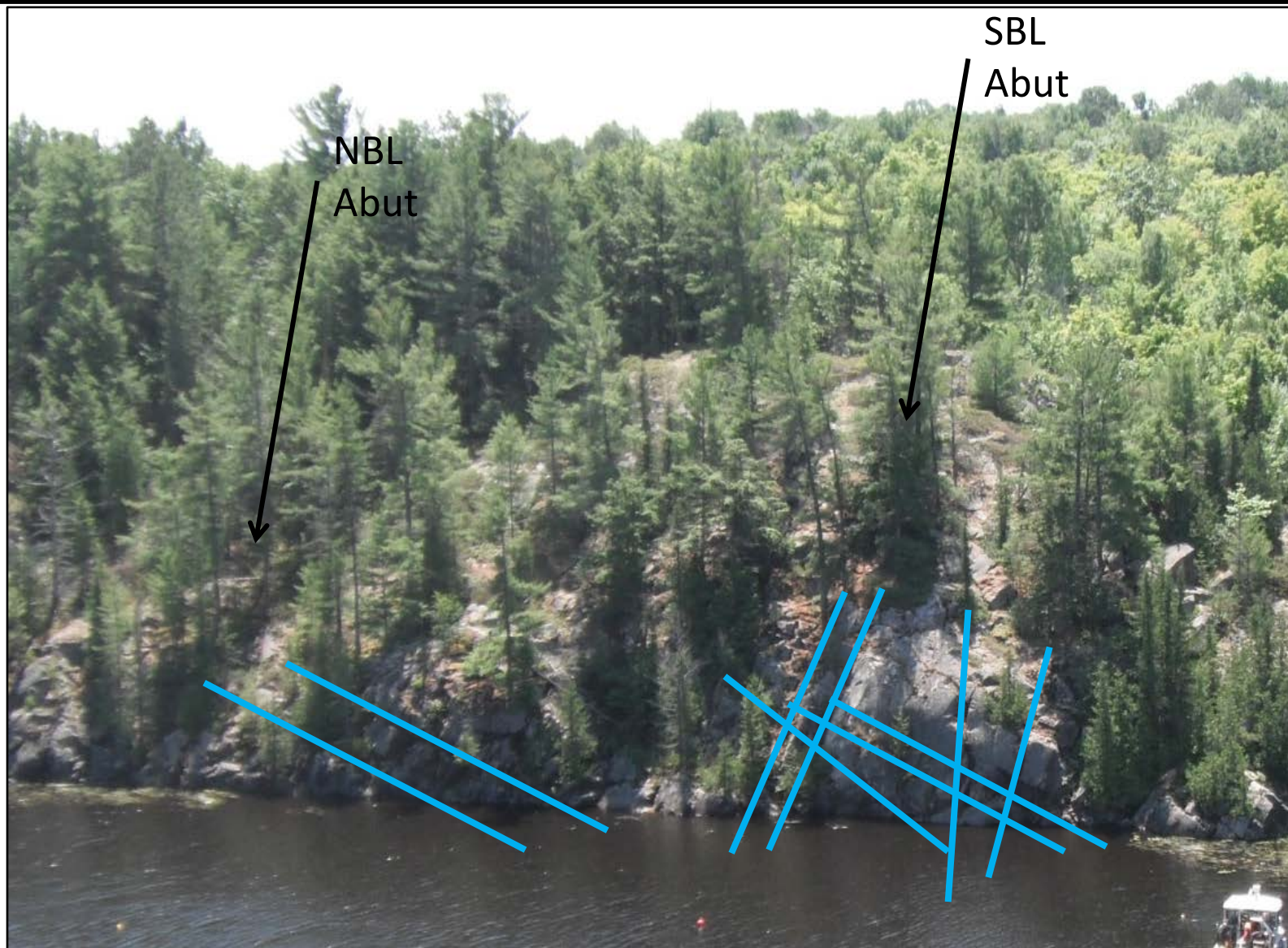
Between SBL and NBL



South Wall of Bedrock Outcrop at NBL and SBL Structures

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES SOUTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D4		
CHECK	MT/JPD	April 2015			
REVIEW					





South Wall of Bedrock Outcrop at NBL and SBL Structures

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES SOUTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D5		
CHECK	MT/JPD	April 2015			
REVIEW					



South Abutments Approximate Locations



South Wall of Bedrock Outcrop at NBL and SBL Structures

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES SOUTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D6		
CHECK	MT/JPD	April 2015			
REVIEW					

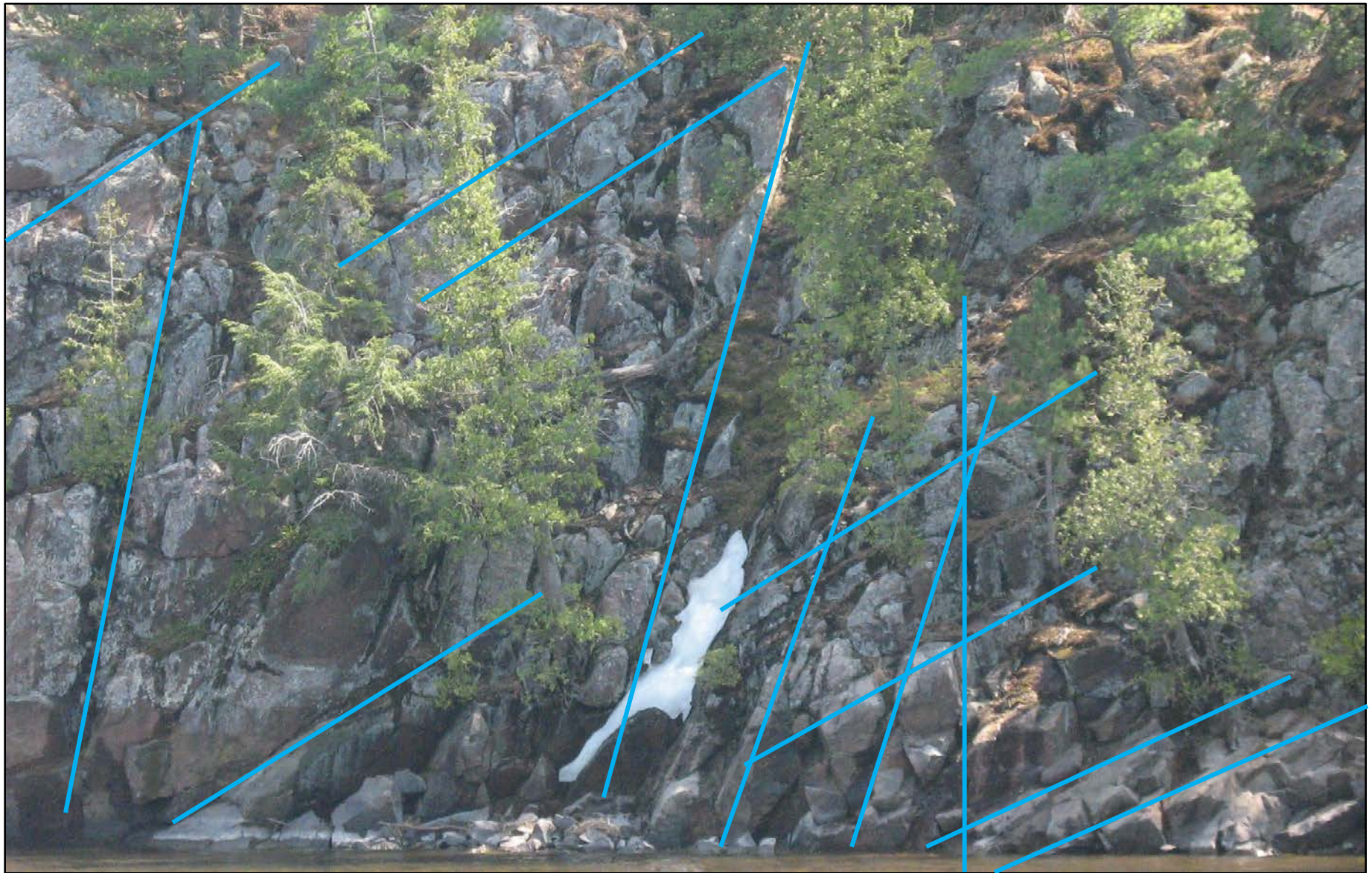




South Wall of Bedrock Outcrop

TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES SOUTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	MT/JPD	April 2015	FIGURE D7		
REVIEW					



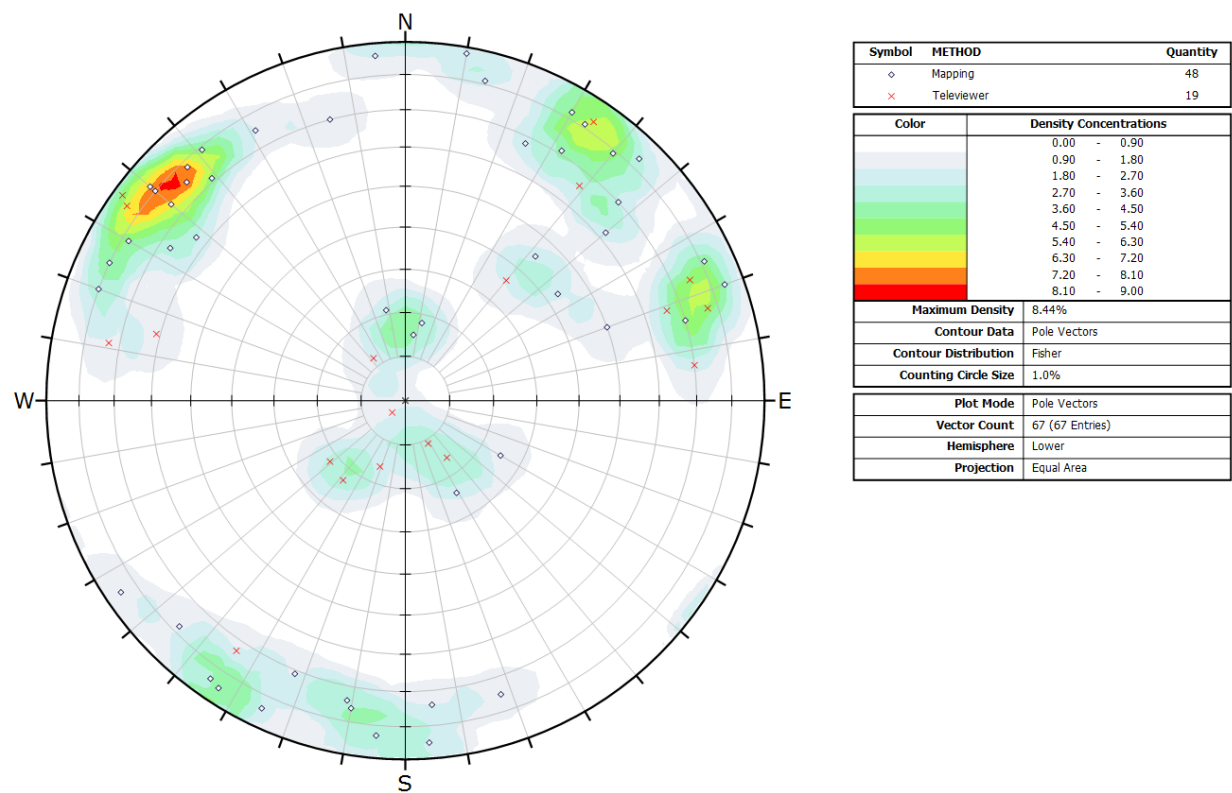


South Wall of Bedrock Outcrop

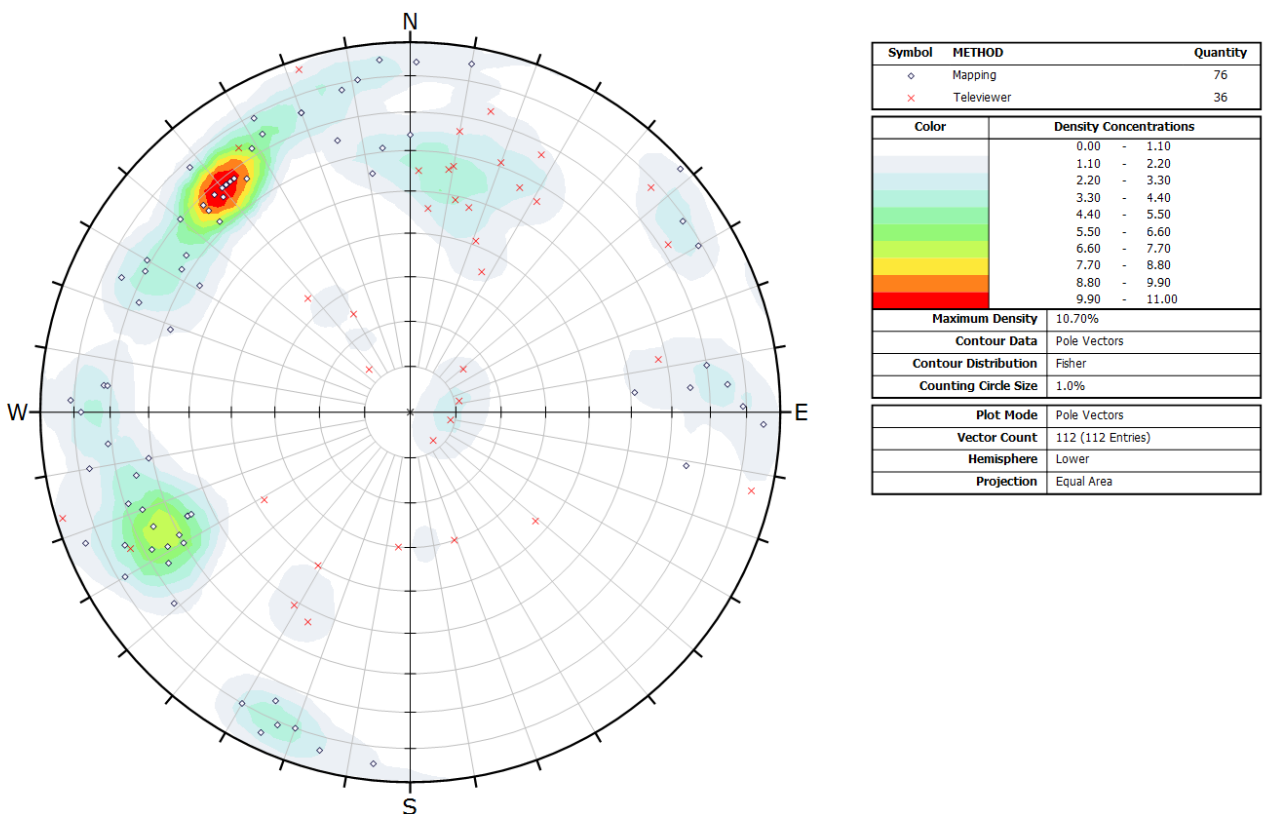
TITLE					
KEY RIVER BRIDGE – ABUTMENT AREAS JOINTS AND FEATURES SOUTH ABUTMENTS					
PROJECT No. 09-1111-6014			FILE No. ----		
DESIGN	AB	April 2015	SCALE	AS SHOWN	REV.
CADD	--		FIGURE D8		
CHECK	MT/JPD	April 2015			
REVIEW					




A) North Abutment – Boreholes B503-16 & B504-17 and Field Mapping

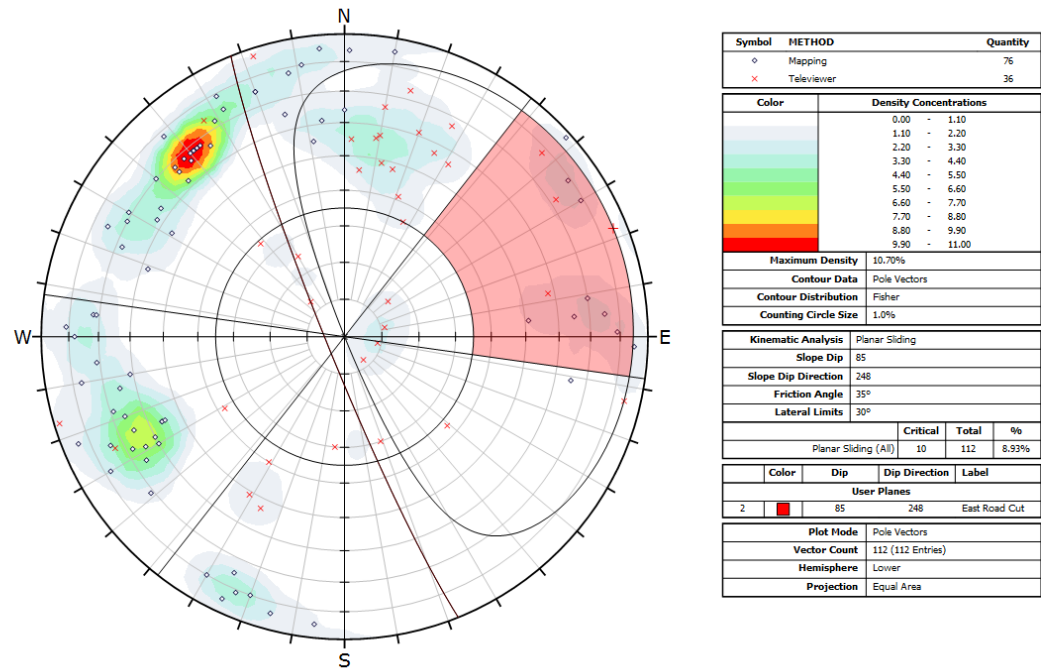


B)) South Abutment – Boreholes B503-06 & B504-06 and Field Mapping

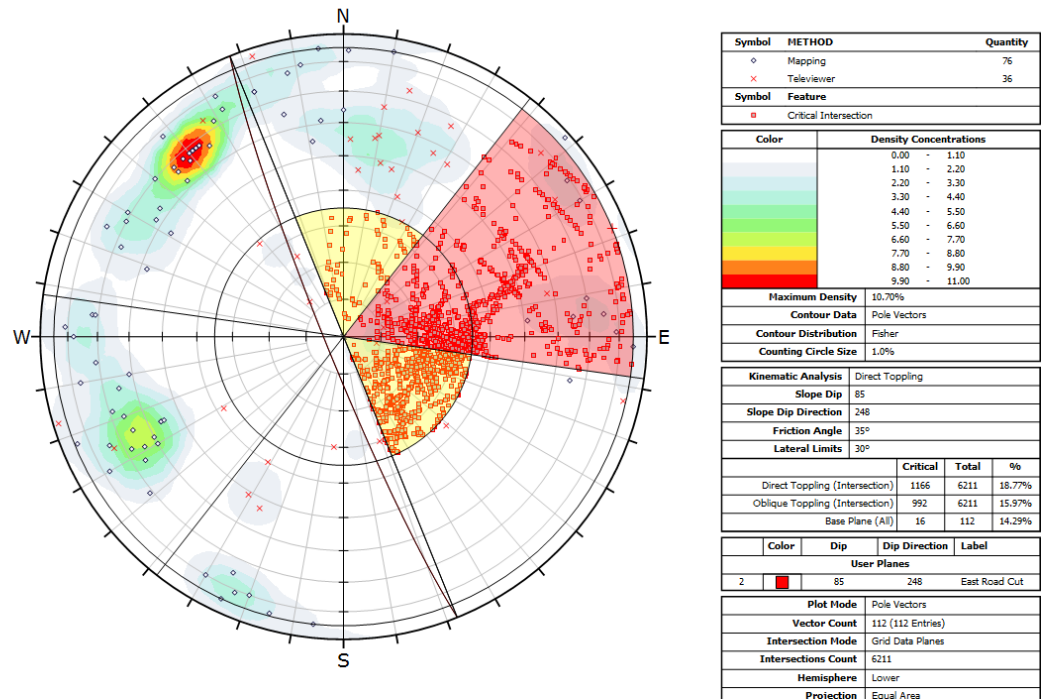


TITLE				
KEY RIVER STRUCTURAL ANALYSIS OF BEDROCK NBL AND SBL APPROACH CUTS				
	PROJECT No. 09-1111-6014		FILE No. ----	
	DESIGN	GK	June 2015	SCALE AS SHOWN
	CADD	--		REV.
	CHECK	GM	April 2015	FIGURE D9
	REVIEW			

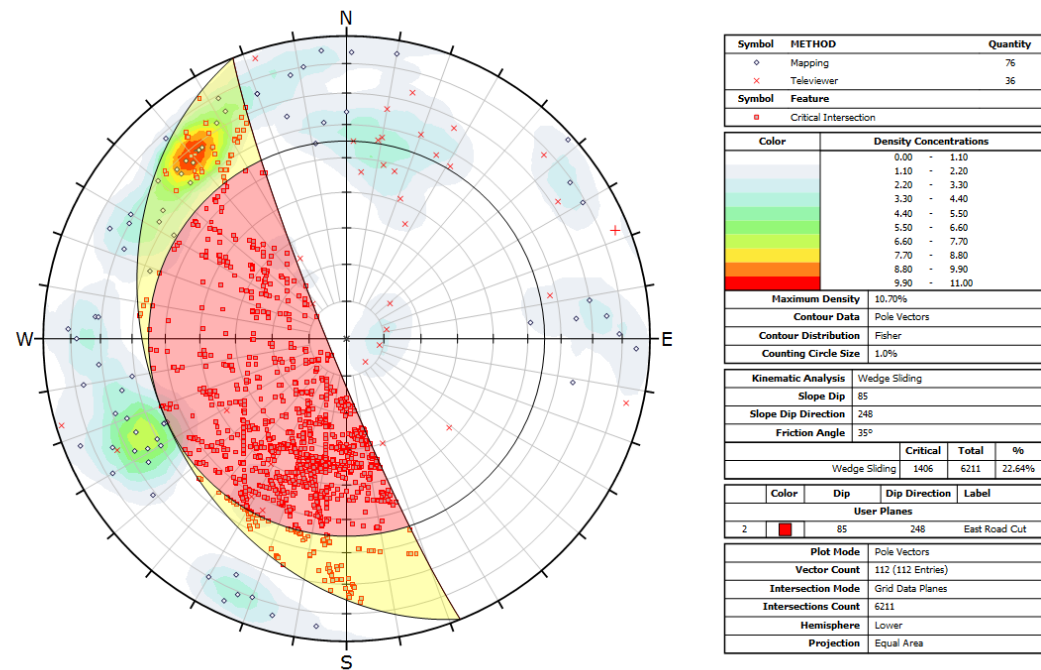
A) South Approach – Rock Cut on East Side – Planar Failure



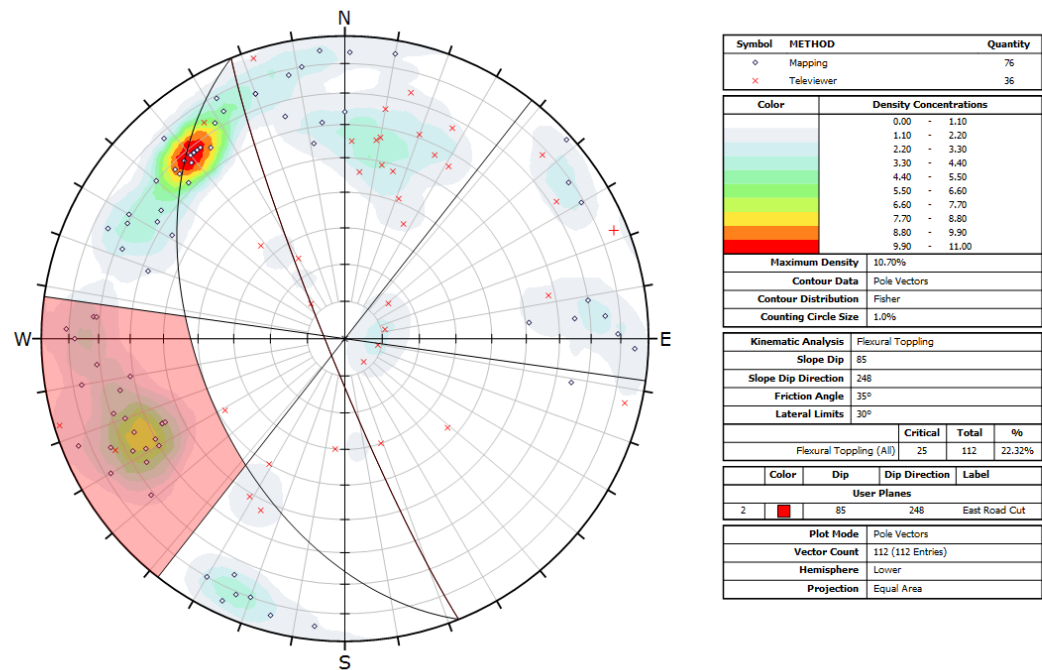
C) South Approach – Rock Cut on East Side – Direct Toppling Failure



B) South Approach – Rock Cut on East Side – Wedge Failure




D) South Approach – Rock Cut on East Side– Flexural Toppling Failure



TITLE

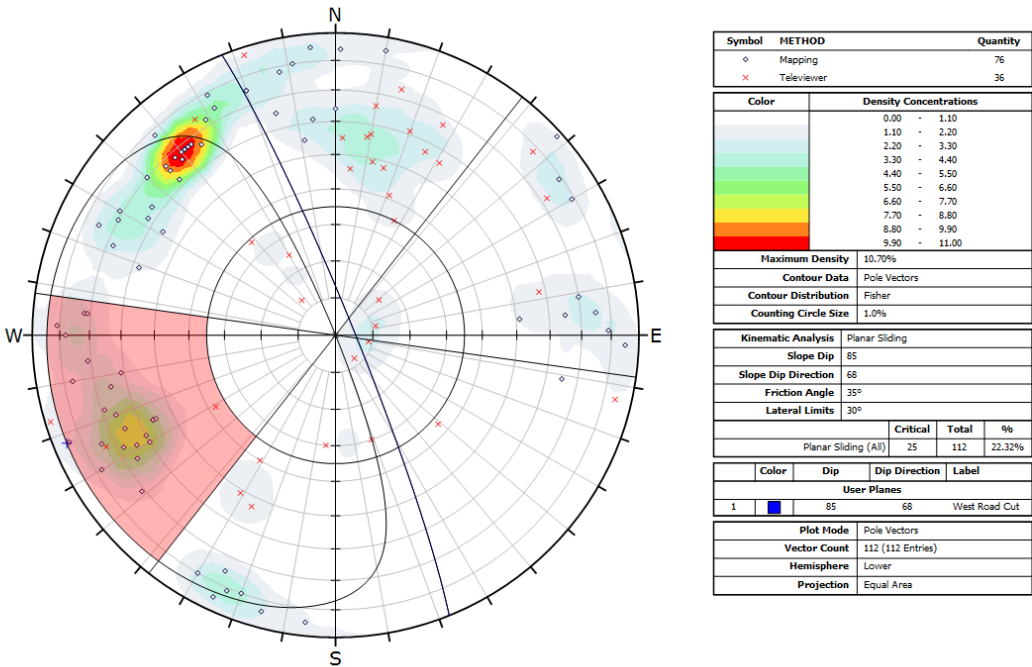
KEY RIVER
STRUCTURAL ANALYSIS OF BEDROCK
NBL AND SBL SOUTH APPROACH CUTS – EAST SIDE



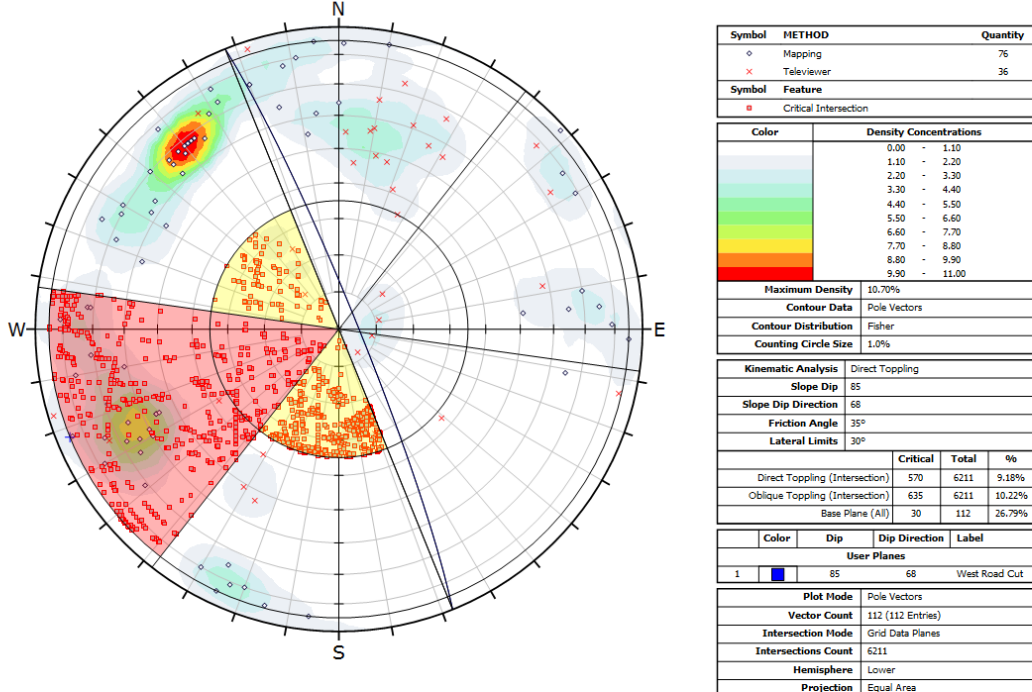
PROJECT No.	09-1111-6014	FILE No.	----
DESIGN	GK	June 2015	SCALE AS SHOWN
CADD	--		REV.
CHECK	GM	June 2015	
REVIEW			

FIGURE D10

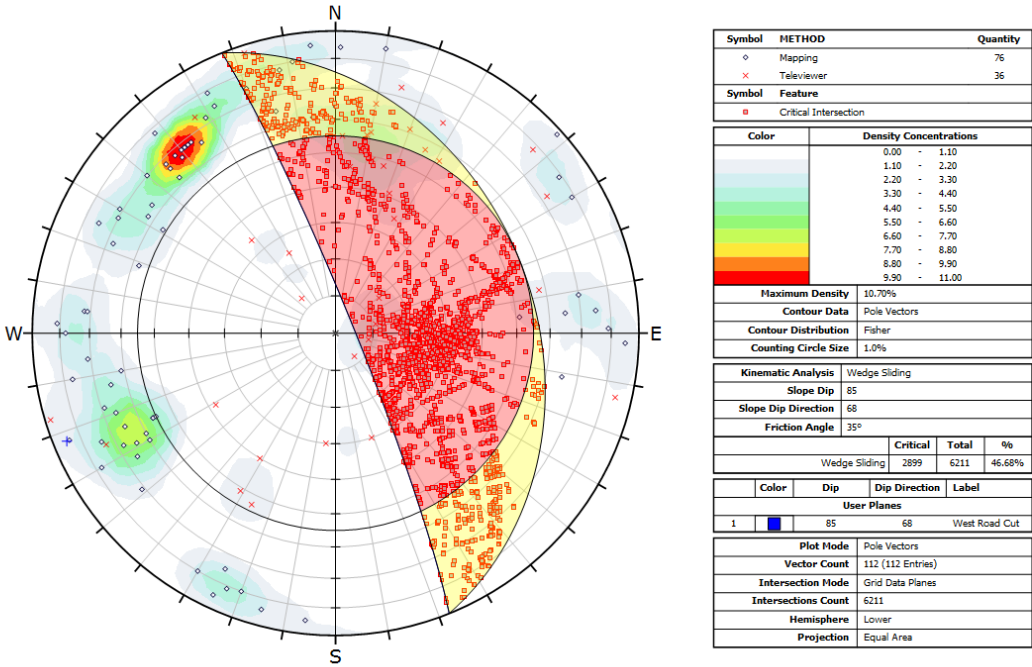
A) South Approach – Rock Cut on West Side – Planar Failure



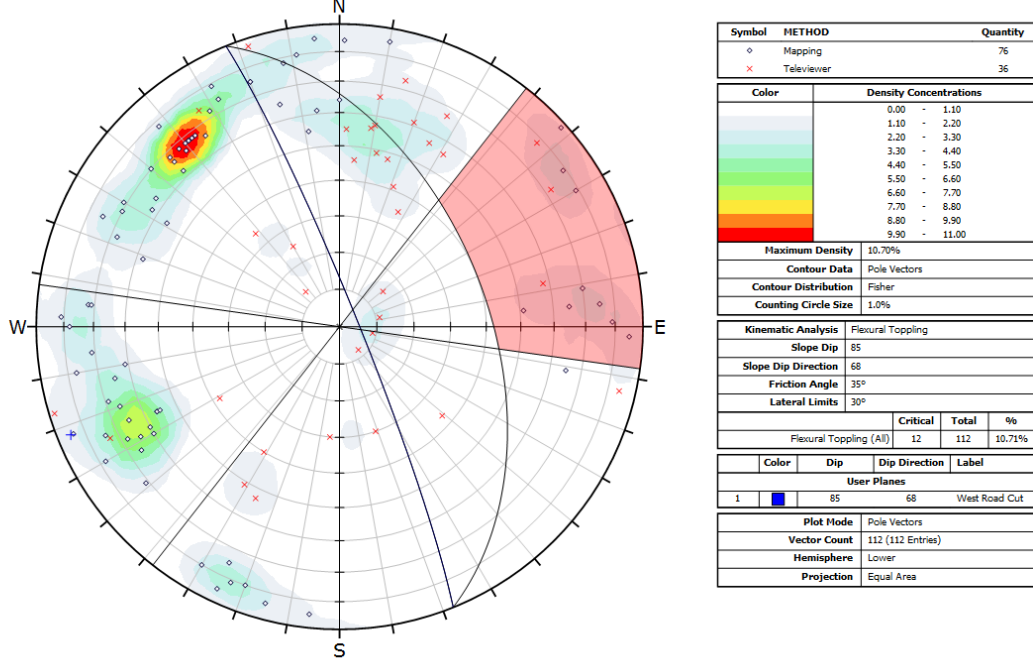
C) South Approach – Rock Cut on West Side – Direct Toppling Failure



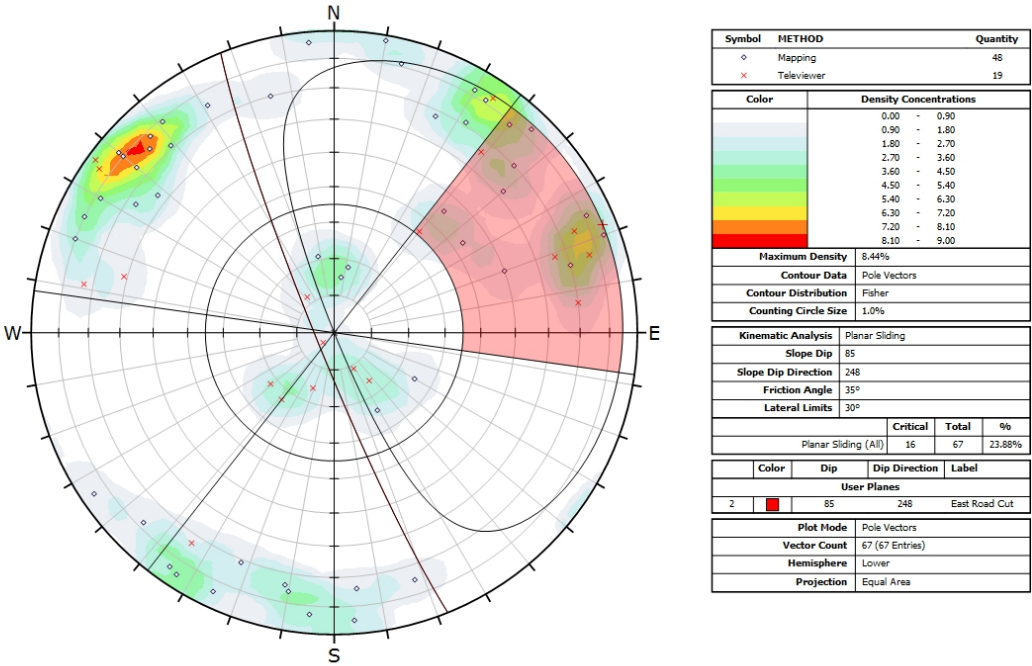
B) South Approach – Rock Cut on West Side – Wedge Failure



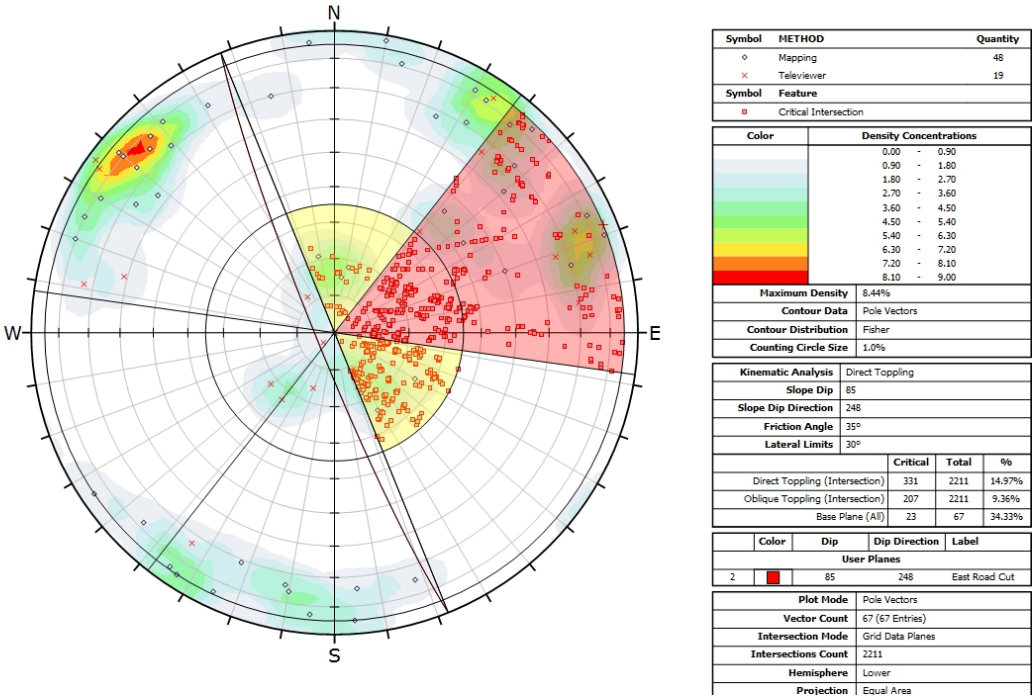
D) South Approach – Rock Cut on West Side – Flexural Toppling Failure



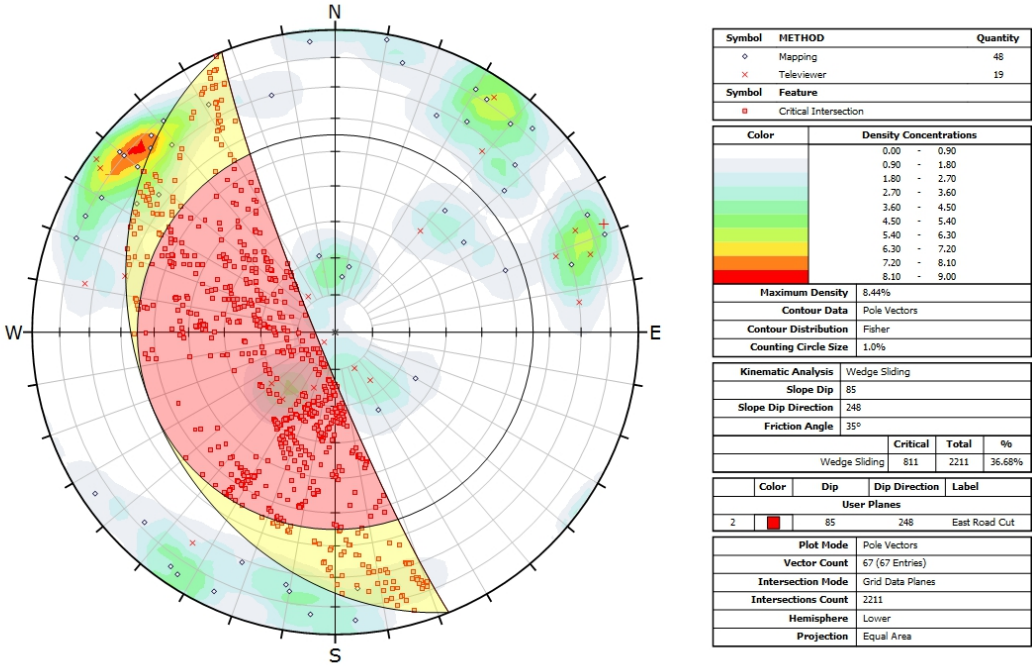
A) North Approach – Rock Cut on East Side – Planar Failure



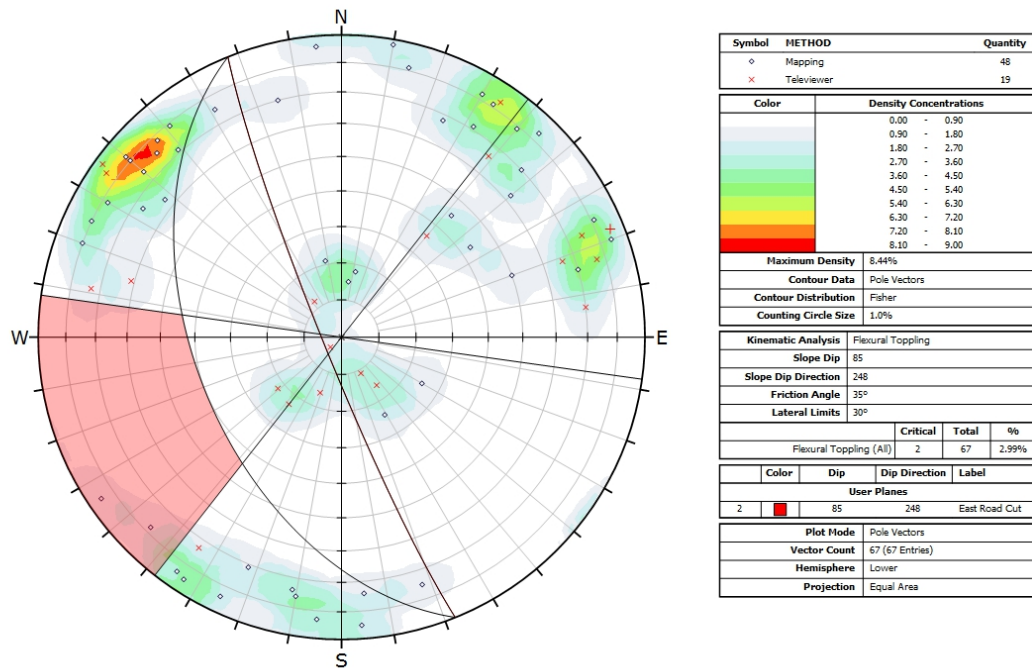
C) North Approach – Rock Cut on East Side – Direct Toppling Failure



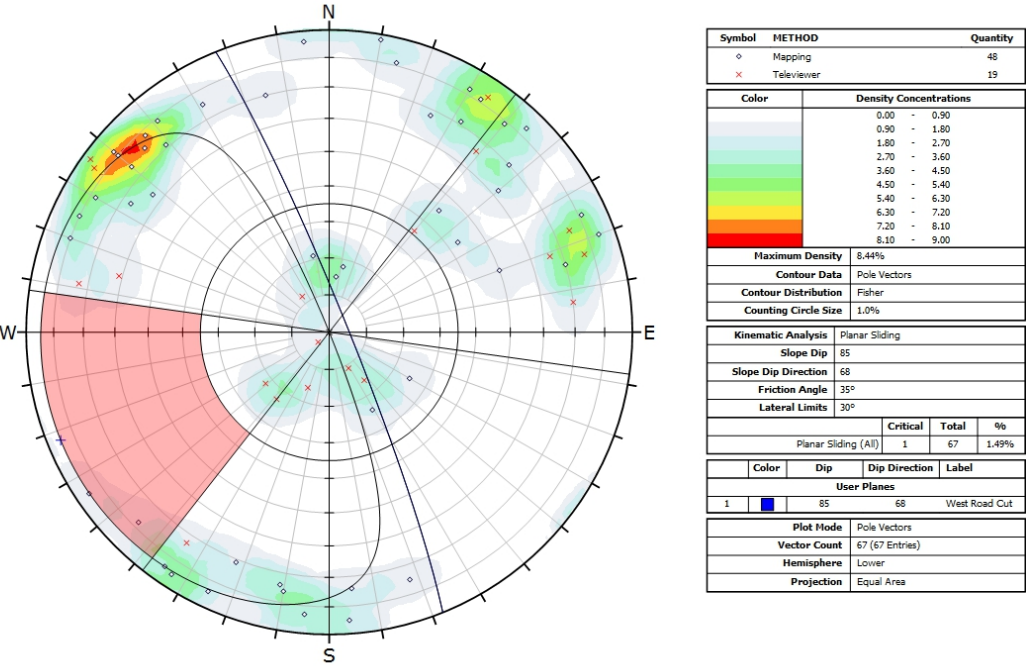
B) North Approach – Rock Cut on East Side – Wedge Failure



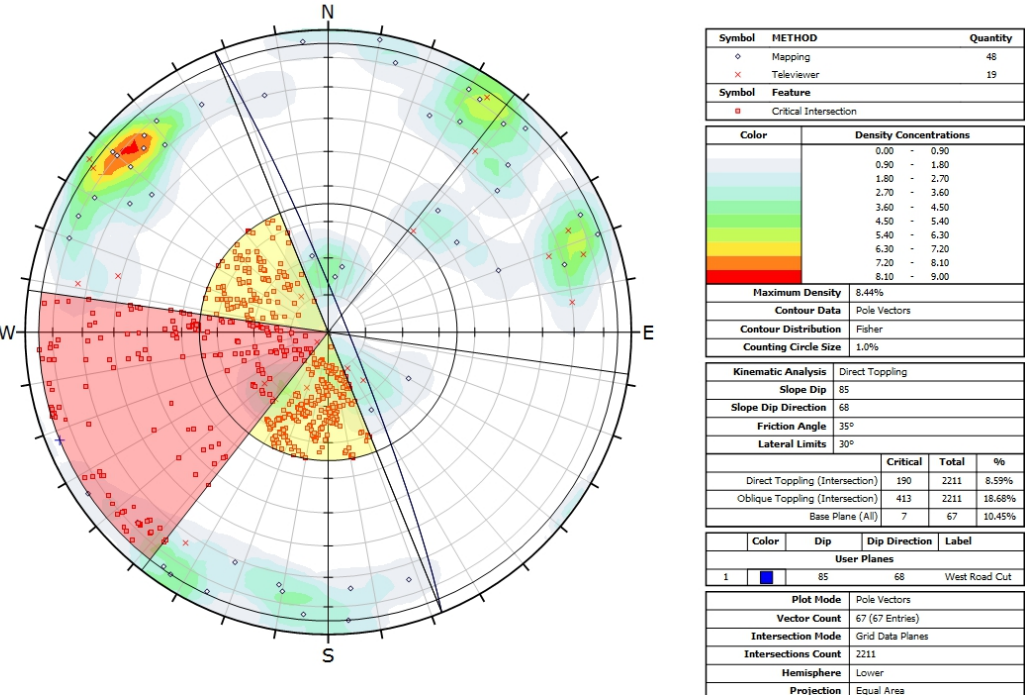
D) North Approach – Rock Cut on East Side – Flexural Toppling Failure



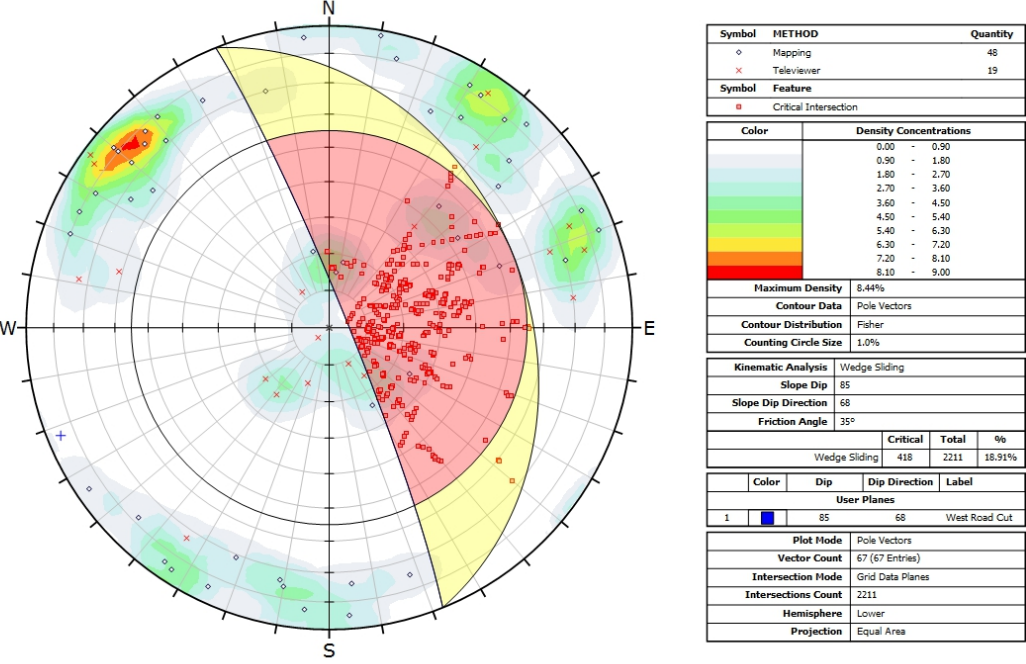
A) North Approach – Rock Cut on West Side – Planar Failure



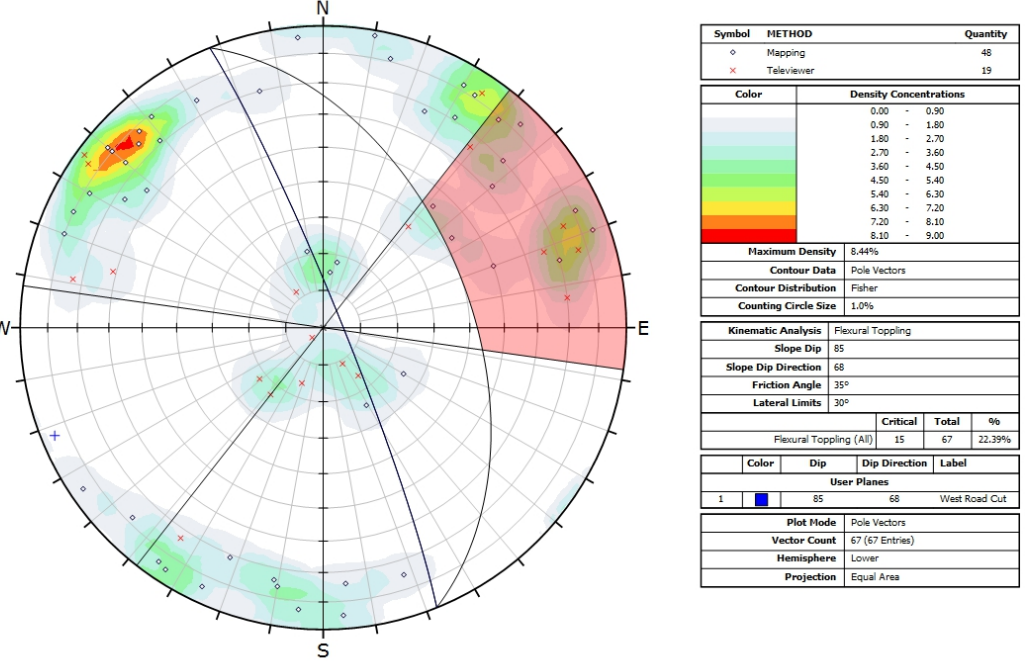
C) North Approach – Rock Cut on West Side – Direct Toppling Failure



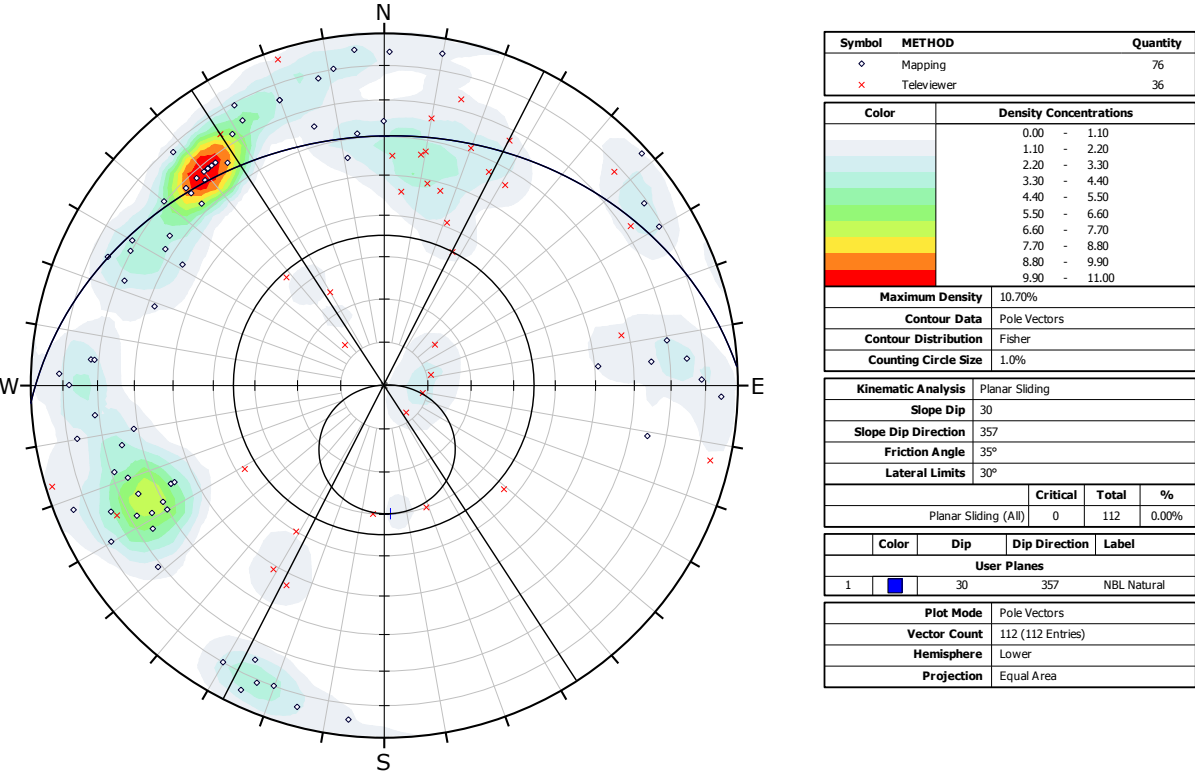
B) North Approach – Rock Cut on West Side – Wedge Failure



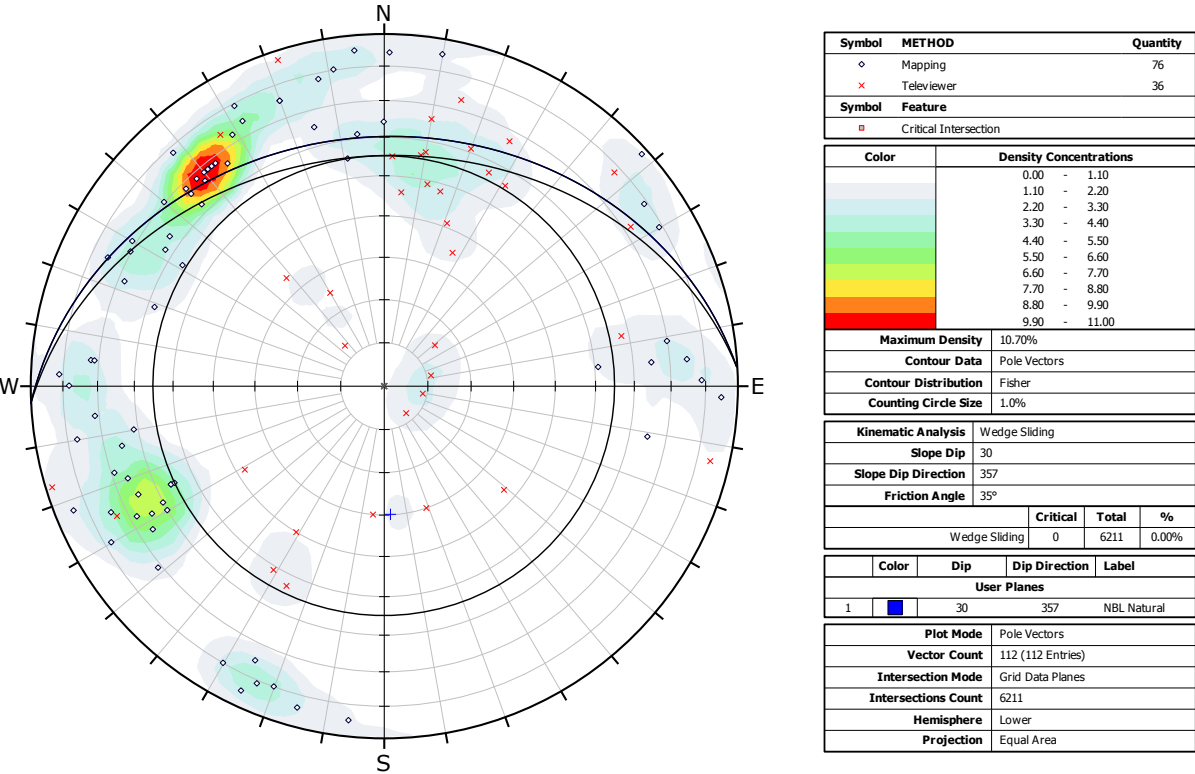
D) North Approach – Rock Cut on West Side – Flexural Toppling Failure



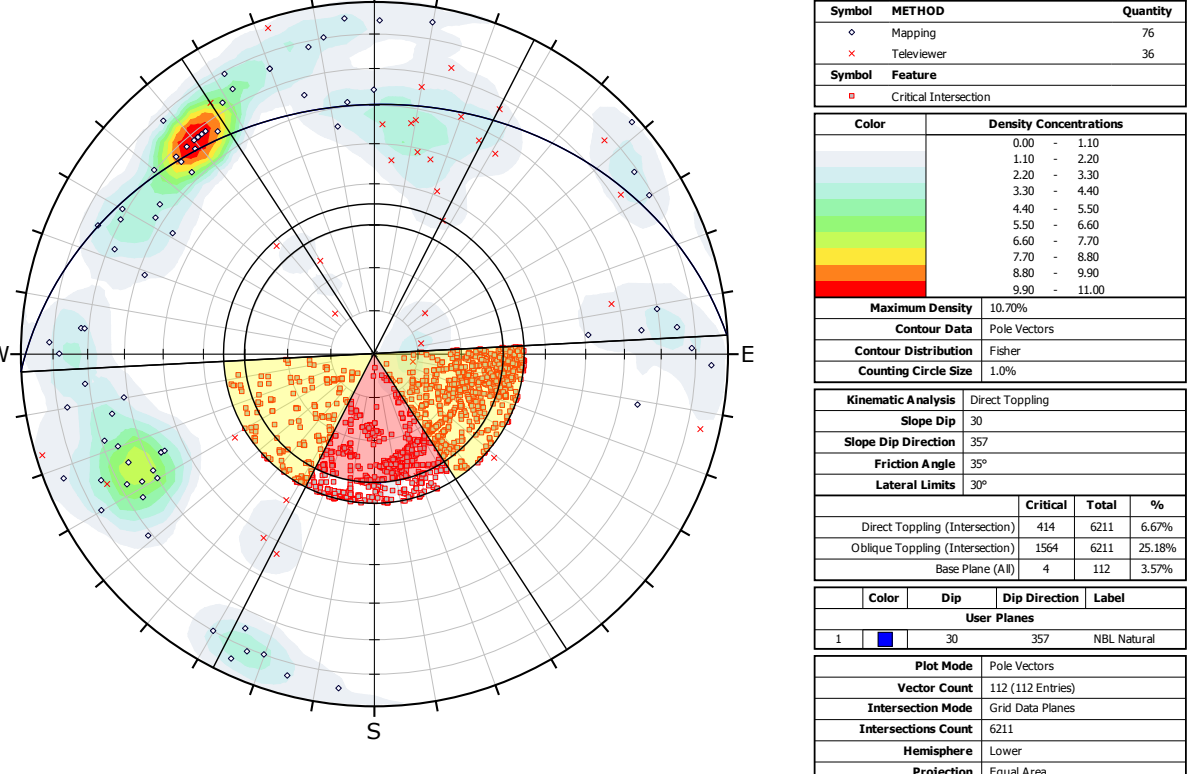
A) South Abutment - North Bound Lane - Natural Slope – Planar Failure



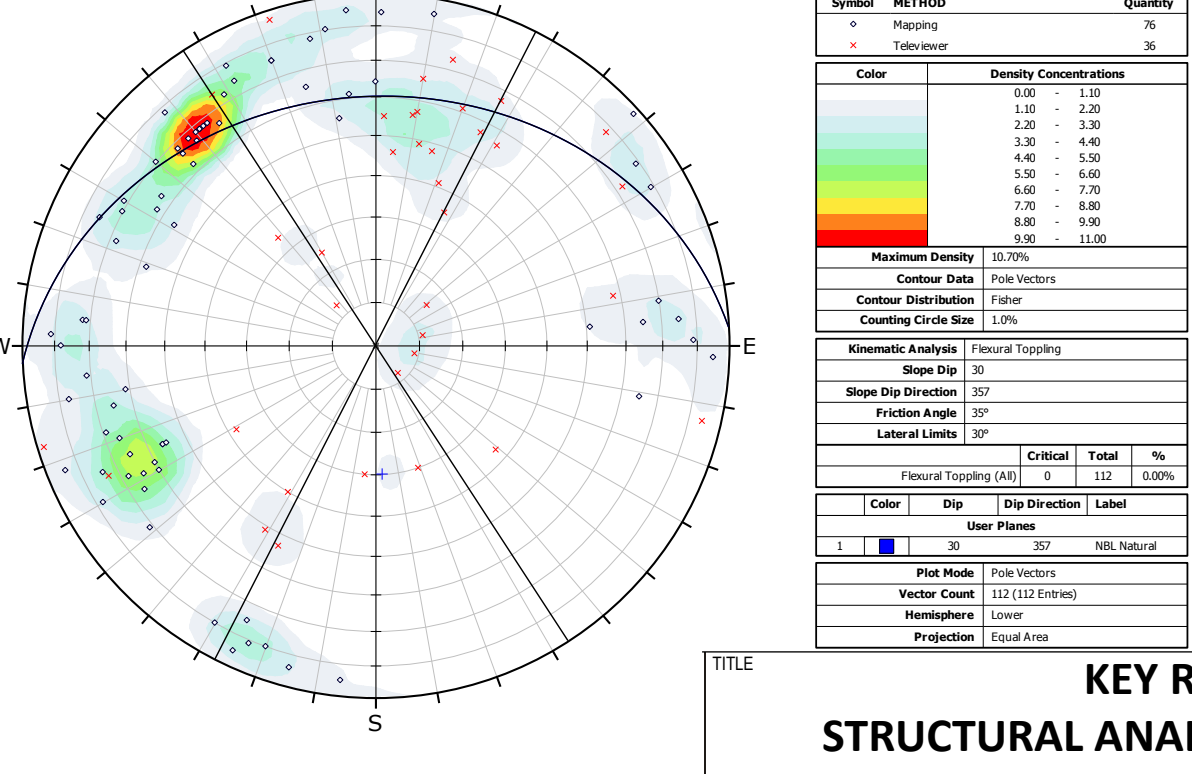
B) South Abutment – North Bound Lane – Natural Slope – Wedge Failure



C) South Abutment – North Bound Lane - Natural Slope – Direct Toppling Failure



D) South Abutment – North Bound Lane – Natural Slope – Flexural Toppling Failure



TITLE

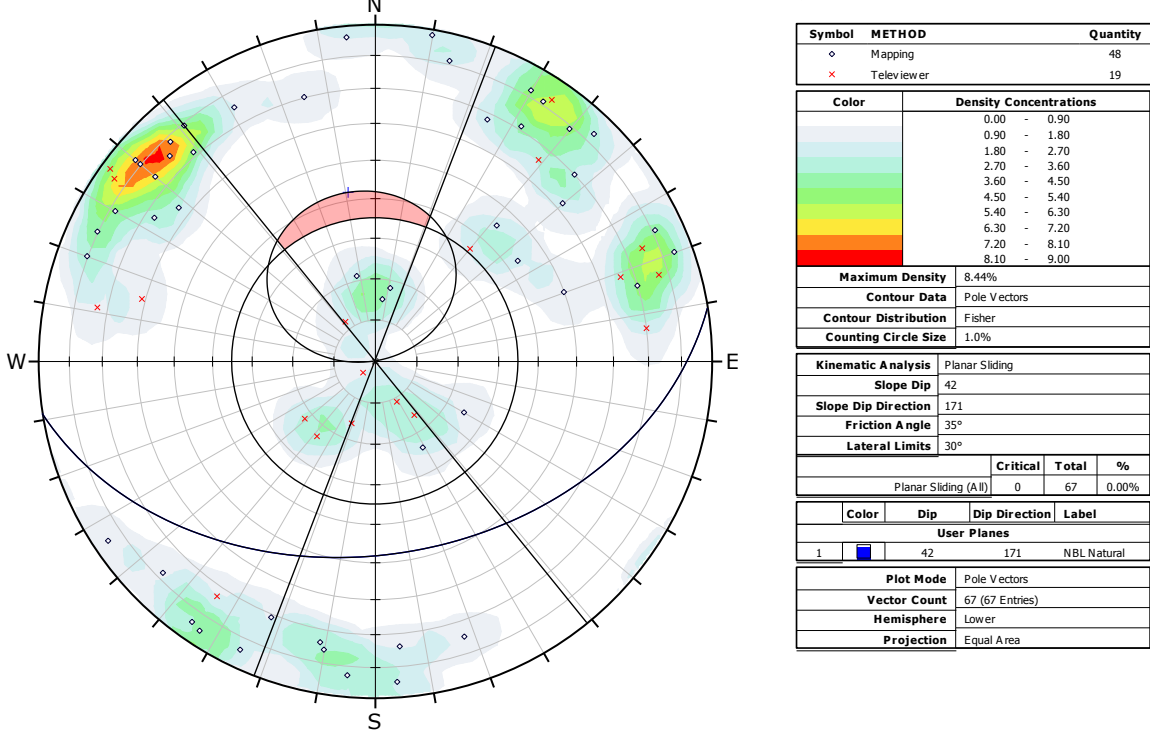
KEY RIVER STRUCTURAL ANALYSIS OF BEDROCK NBL South Abutment – Natural Front Slope



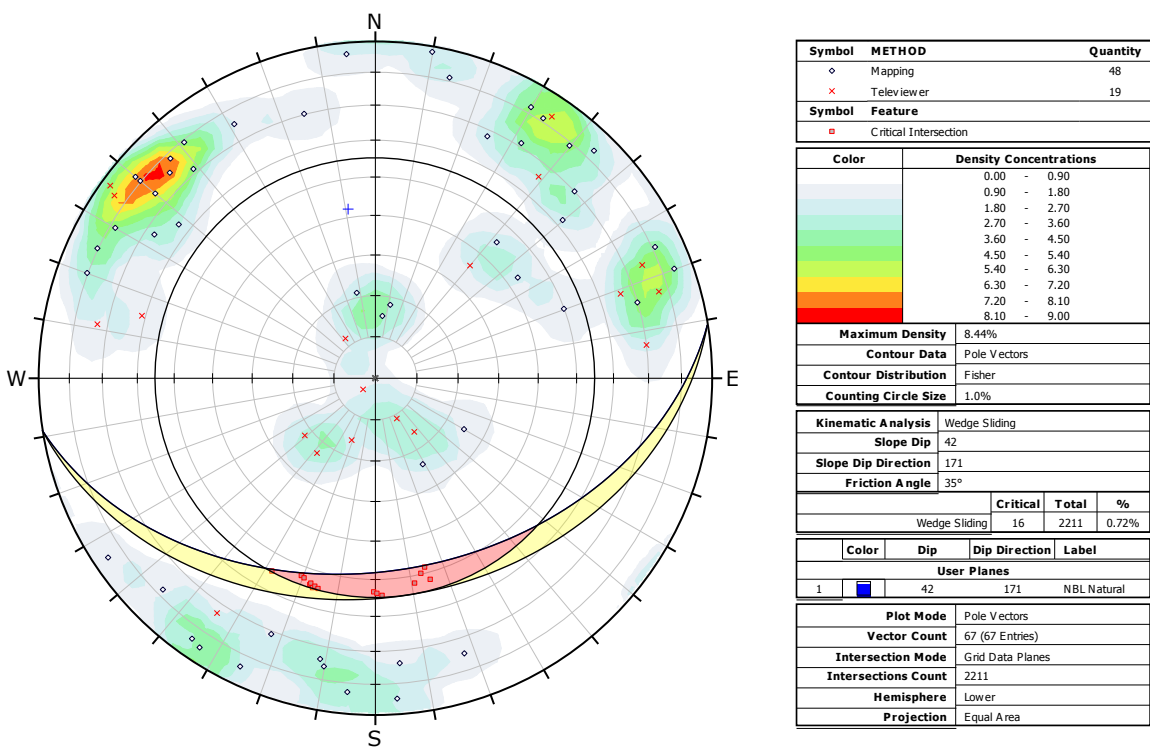
PROJECT No.	09-1111-6014	FILE No.	----
DESIGN	GK	June 2015	SCALE AS SHOWN
CADD	--		REV.
CHECK	GM	June 2015	
REVIEW			

FIGURE D14

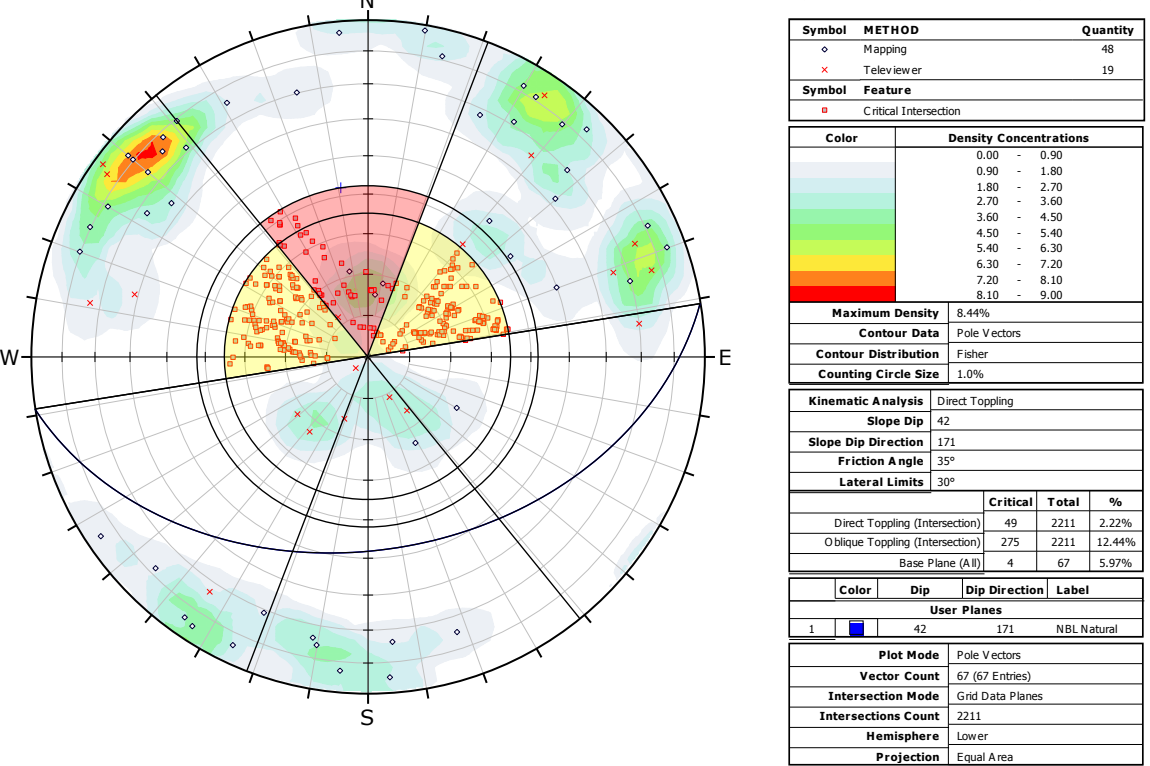
A) North Abutment - North Bound Lane - Natural Slope – Planar Failure



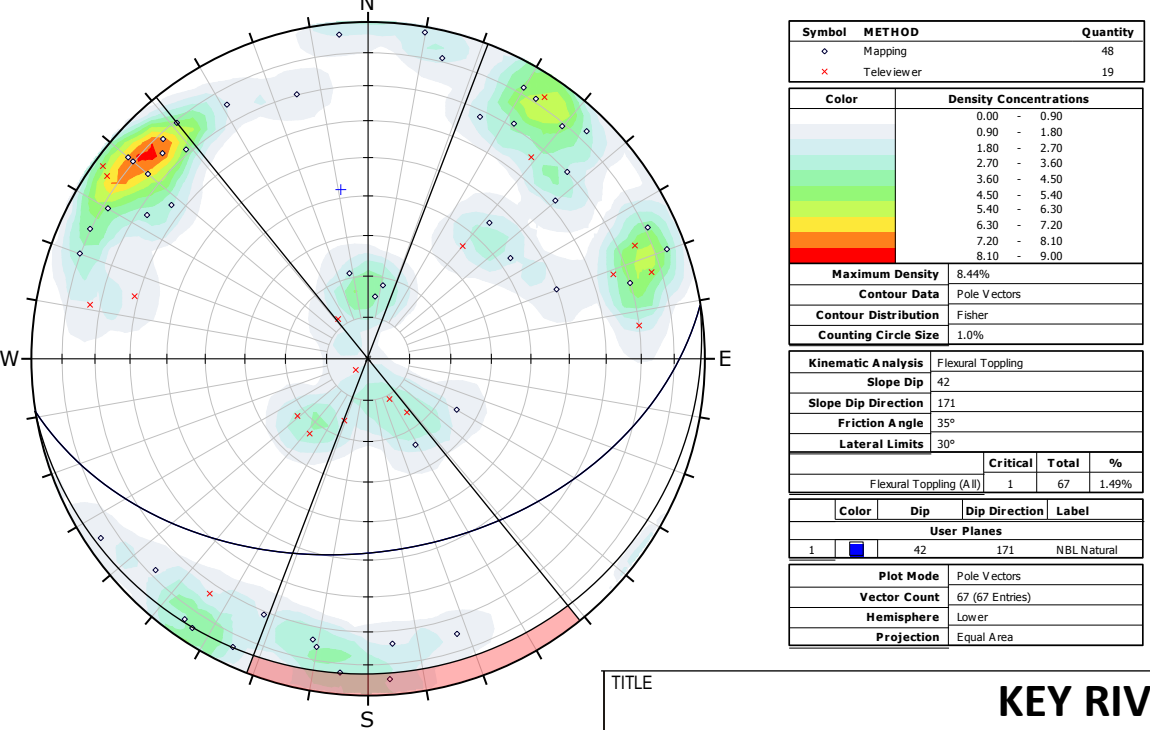
B) North Abutment – North Bound Lane – Natural Slope – Wedge Failure



C) North Abutment - North Bound Lane - Natural Slope – Direct Toppling Failure



D) North Abutment – North Bound Lane – Natural Slope – Flexural Toppling Failure



TITLE

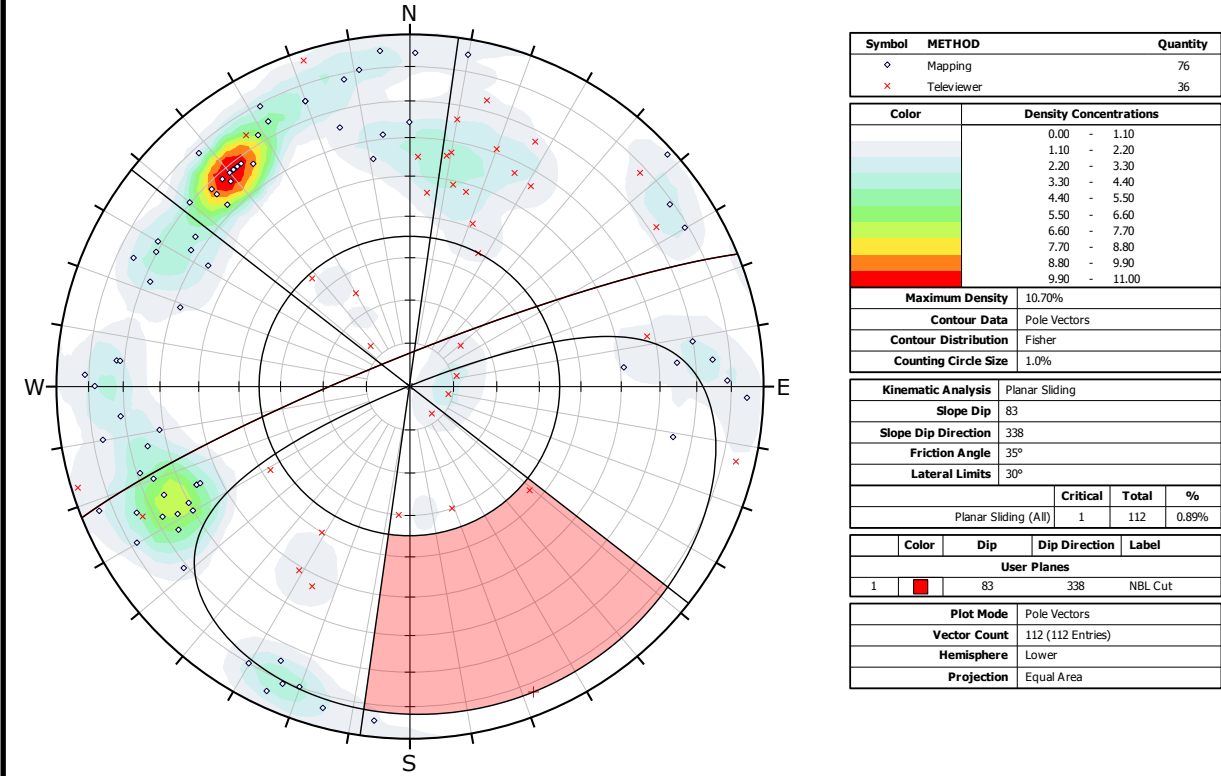
KEY RIVER
STRUCTURAL ANALYSIS OF BEDROCK
NBL North Abutment – Natural Front Slope



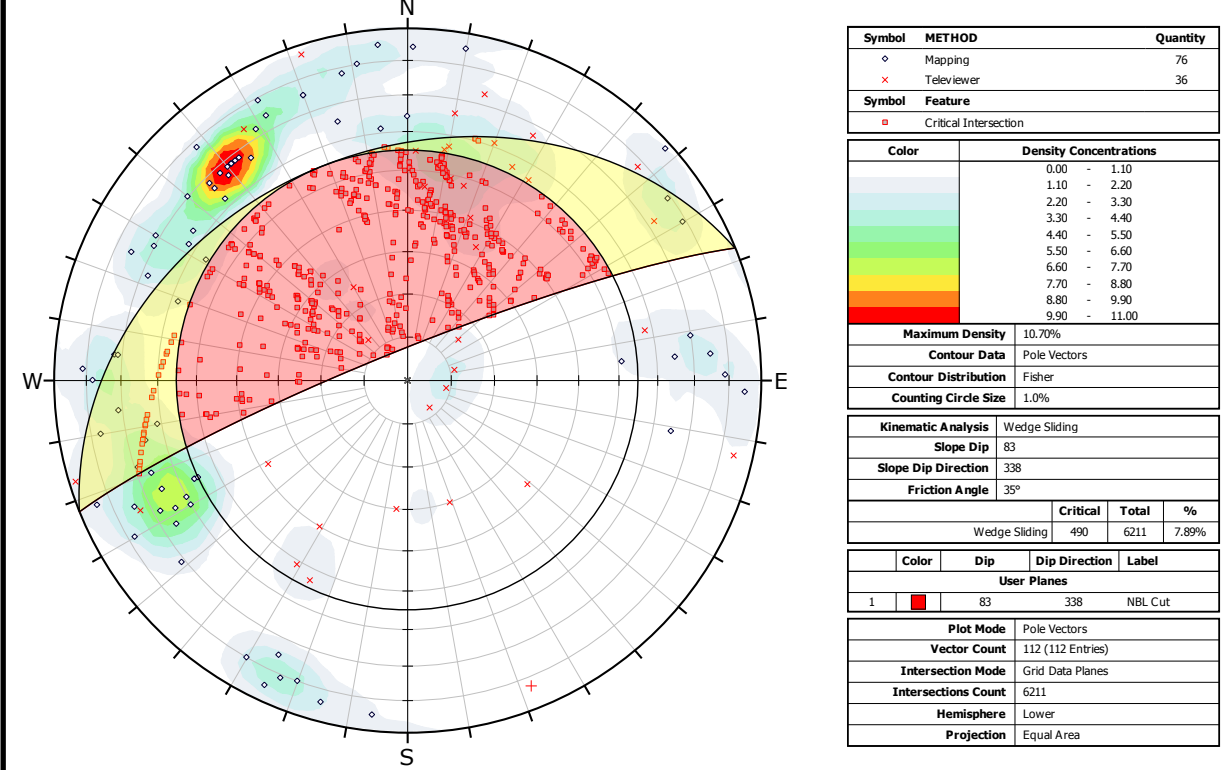
PROJECT No.	09-1111-6014	FILE No.	----
DESIGN	GK	June 2015	SCALE AS SHOWN
CADD	--		REV.
CHECK	GM	April 2015	
REVIEW			

FIGURE D15

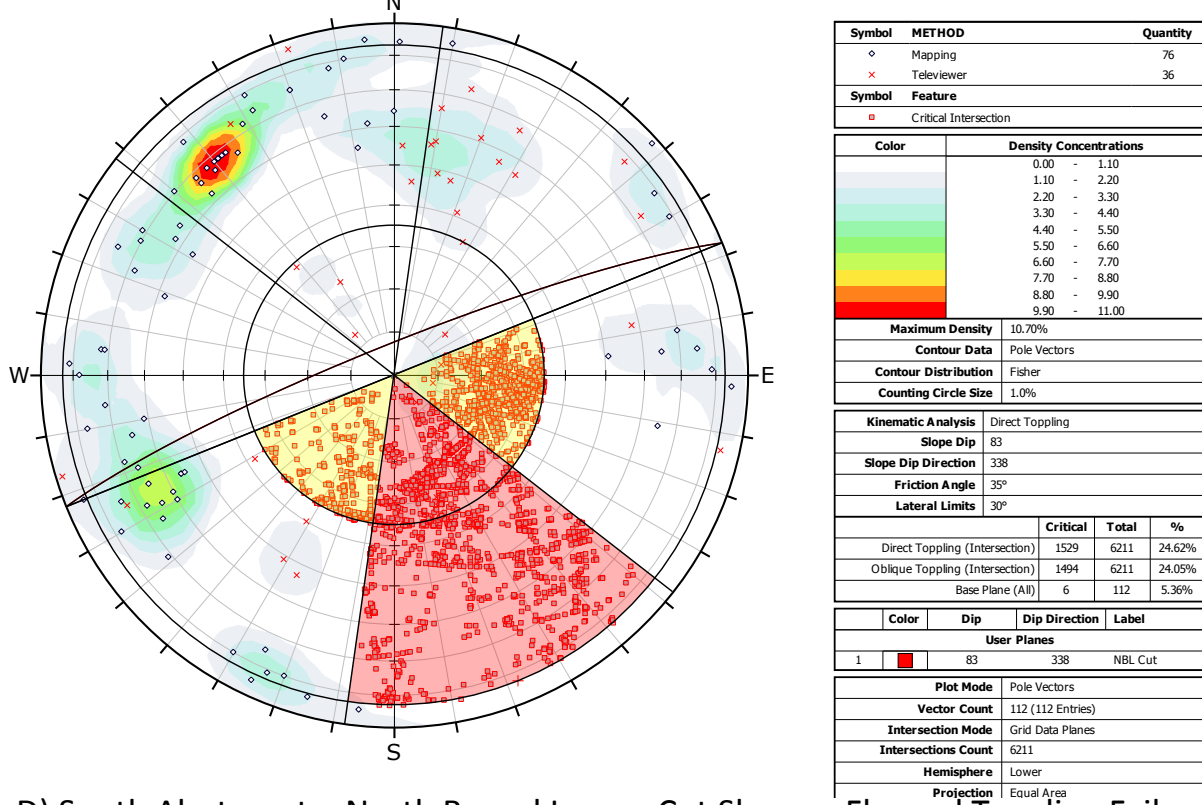
A) South Abutment - North Bound Lane – Cut Slope – Planar Failure



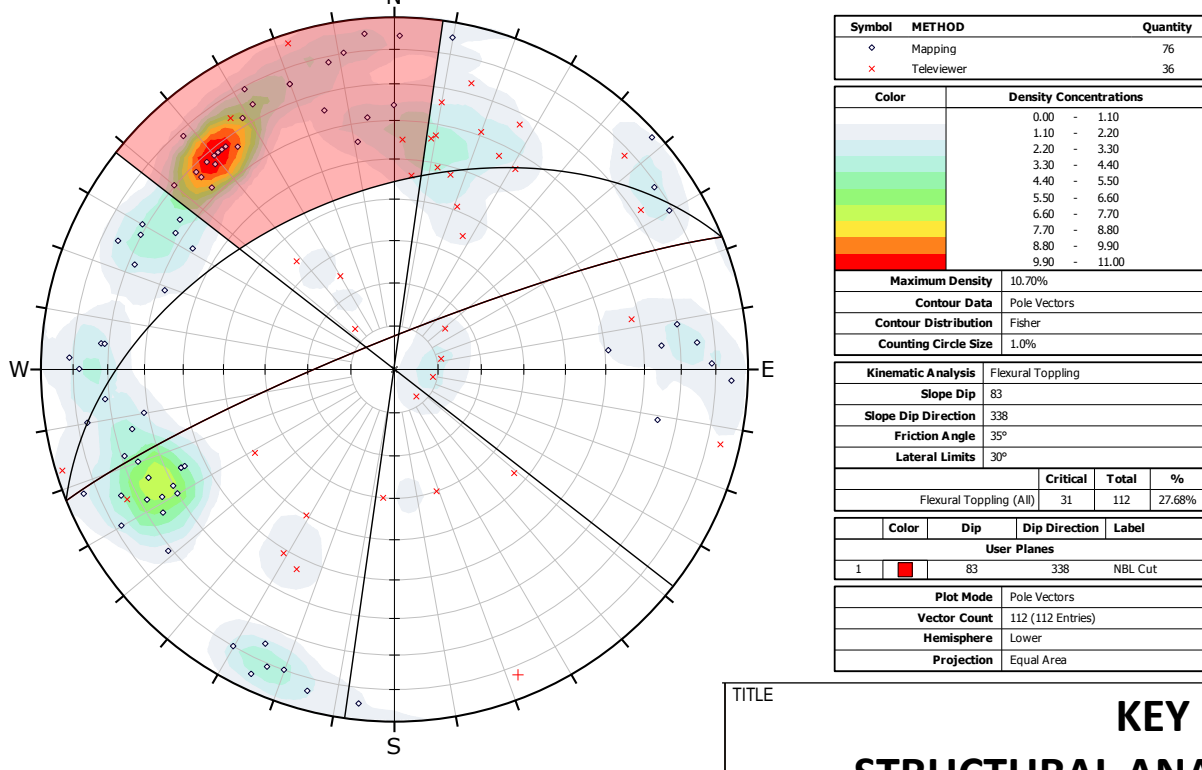
B) South Abutment – North Bound Lane – Cut Slope – Wedge Failure



C) South Abutment – North Bound Lane - Cut Slope – Direct Toppling Failure



D) South Abutment – North Bound Lane – Cut Slope – Flexural Toppling Failure



TITLE

KEY RIVER

STRUCTURAL ANALYSIS OF BEDROCK

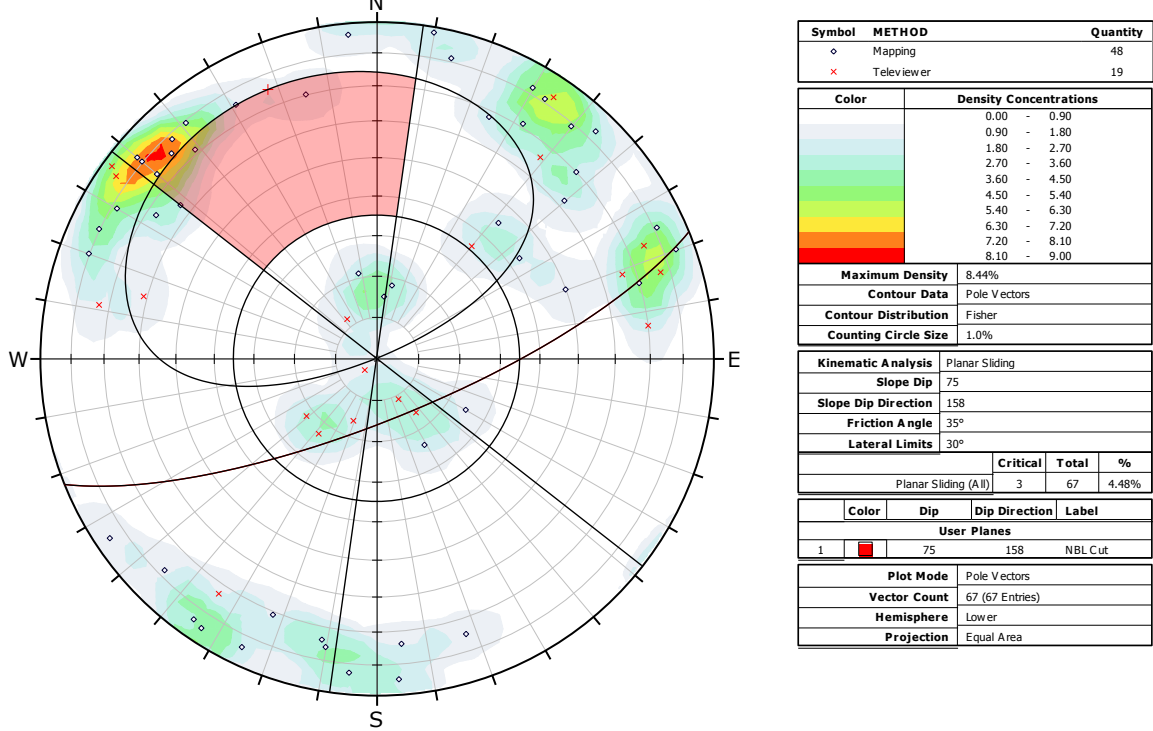
NBL South Abutment – Rock Cut at Abutment



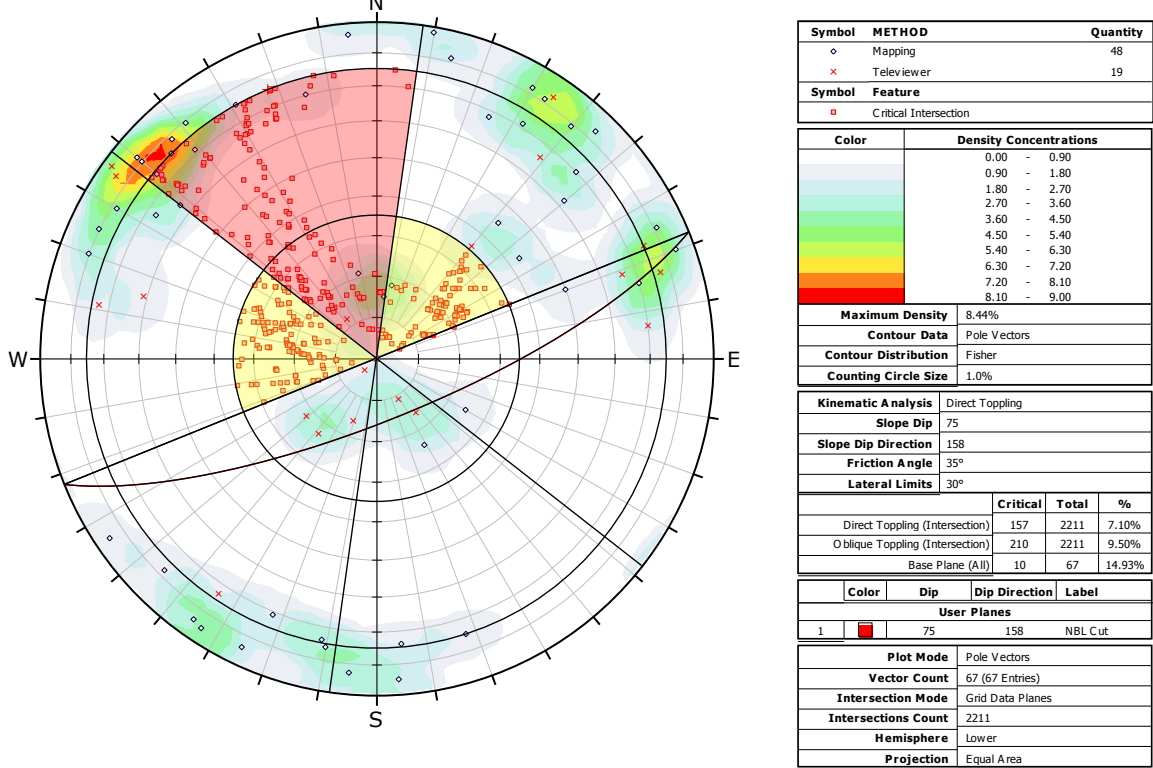
PROJECT No.	09-1111-6014	FILE No.	----
DESIGN	GK	June 2015	SCALE AS SHOWN
CADD	--		REV.
CHECK	GM	June 2015	
REVIEW			

FIGURE D16

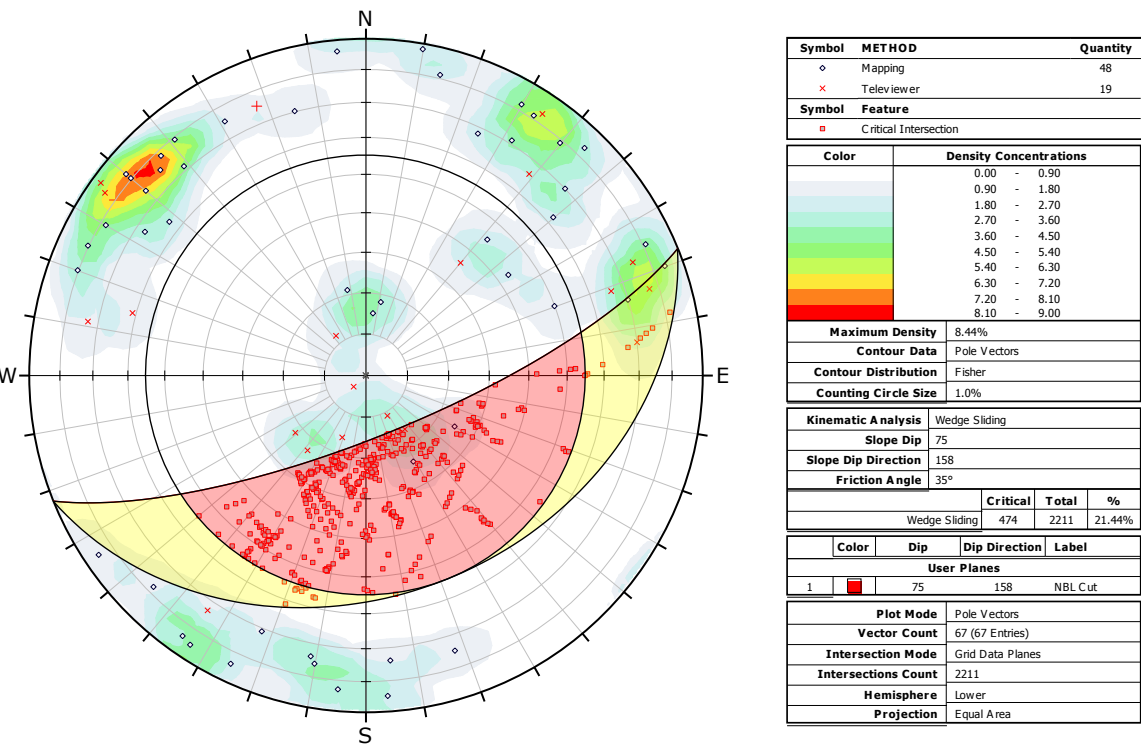
A) North Abutment - North Bound Lane - Cut Slope – Planar Failure



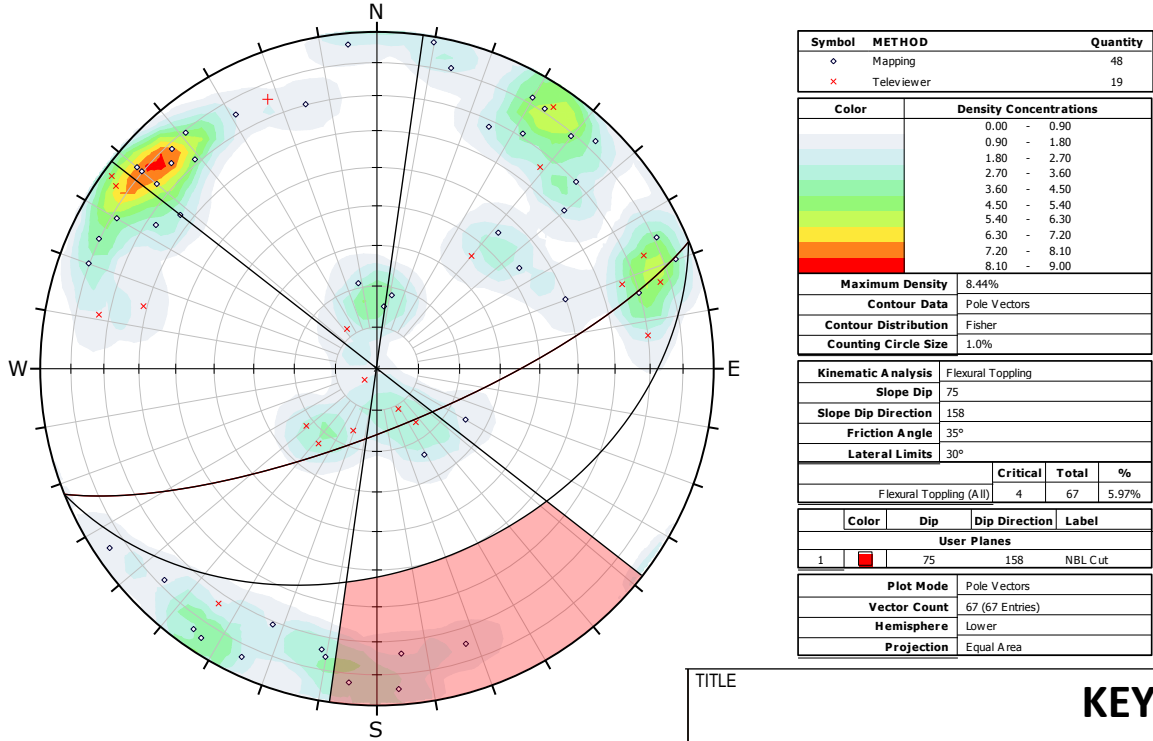
C) North Abutment - North Bound Lane - Cut Slope – Direct Toppling Failure



B) North Abutment – North Bound Lane – Cut Slope - Wedge Failure



D) North Abutment – North Bound Lane – Cut Slope – Flexural Toppling Failure



KEY RIVER
STRUCTURAL ANALYSIS OF BEDROCK
NBL North Abutment – Rock Cut at Abutment



PROJECT No.	09-1111-6014	FILE No.	----
DESIGN	GK	June 2015	SCALE AS SHOWN
CADD	--		REV.
CHECK	GM	June 2015	
REVIEW			

FIGURE D17



APPENDIX E

Non-Standard Special Provisions

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

Scope of Work

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.PROV 904 Concrete Structuresⁱ. All reinforcing steel supplied shall be in accordance with OPSS 1440 Steel Reinforcement for Concreteⁱⁱ (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 30 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 over Key River	North and South Abutments	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 pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 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 percent of the theoretical elastic elongation of the free stressing length and is less than the theoretical elongation of the free stressing length plus 50 percent 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.

END OF SECTION

ⁱ OPSS.PROV 904 Construction Specification for Concrete Structures

ⁱⁱ OPSS 1440 Material Specification for Steel Reinforcement for Concrete

Obstructions - Item No.

Non-Standard Special Provision

The Contactor is hereby notified that cobbles and boulders are present within/underlying the sand to gravelly sand deposit (overlying bedrock) below the river bed in Key River. Consideration of the presence of these obstructions must be made in selection of appropriate equipment for installation of the piles at the piers.

Basis of Payment

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

END OF SECTION

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

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