



**FOUNDATION INVESTIGATION AND DESIGN REPORT**

**for**

**STEPHENSON ROAD NO. 2 UNDERPASS  
WP 5040-00-01, SITE 42-327  
HIGHWAY 11  
TOWNSHIP OF STEPHENSON  
DISTRICT 52, HUNTSVILLE**

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PML Ref.: 04TF006-SR2  
Index No.: 075FIR and 076FDR  
Geocres No.: 31E-238  
May 18, 2005



**FOUNDATION INVESTIGATION REPORT**

**for**

**STEPHENSON ROAD NO. 2 UNDERPASS**

**WP 5040-00-01, SITE 42-327**

**HIGHWAY 11**

**TOWNSHIP OF STEPHENSON**

**DISTRICT 52, HUNTSVILLE**

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**FOUNDATION INVESTIGATION REPORT**

for  
Stephenson Road No. 2 Underpass  
WP 5040-00-01, Site 42-327  
Highway 11  
Township of Stephenson  
District 52, Huntsville

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**1. INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed construction of an underpass at Stephenson Road No. 2 and Highway 11 some 16 km south of Huntsville, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

The realigned Stephenson Road No. 2 crosses over Highway 11 at Station 11+711.887, Highway 11 chainage, in the Township of Stephenson (ref. Drawing 1 "Stephenson Road No. 2 Underpass Highway 11 - General Arrangement" prepared by MRC in July 2004). Relevant data from the preliminary foundation investigation carried out by Golder Associates Limited (GAL) reference No. 011-1104 dated April 2001 is provided in this report (Appendix A).

The report provides subsurface information pertaining to the proposed underpass structure and approaches within about 20 m of the abutments.

**2. SITE DESCRIPTION AND GEOLOGY**

The site is located about 500 m south of the existing Stephenson Road No. 2 at-grade crossing of Highway 11 about 4 km south of the Highway 141 intersection. Numerous rock exposures are visible along Highway 11 near the site. The vegetation cover is generally dense with mature trees and brush. A few residences exist near the existing Stephenson Road No. 2 and Highway 11 intersection. Photographs of the site are shown in Appendix B (Plates 1 to 4).

Highway 11 is presently a four-lane divided south-north highway. Therefore, the alignment of the underpass extends west-east.



The project site physiography comprises mainly sands and silts within a narrow band that extends from Gravenhurst to North Bay ("The Physiography of Southern Ontario", Chapman and Putnam, 1984). The topography is irregular but typically undulating and dotted with areas of wet ground separated by steep rock ridges.

The site is located within the Central Gneiss Belt (Geologic Map 2544, Ministry of Northern Development and Mines) that comprises Precambrian rock formations. The typical rock types in the project area are migmatites, gneisses and felsic igneous rocks, such as granite. The soil/bedrock interface is at variable depths ranging from the surface to over 35 m along the alignment.

### **3. INVESTIGATION PROCEDURES**

The field work for the centre pier and east abutment was carried out during the period of October 25 to November 1, 2004. To provide additional assurance of the bedrock profiles, Peto MacCallum Ltd. commissioned a survey of the rock surface profile under the center pier and east abutment using seismic refraction soundings (SRS). Geophysics GPR International Inc. (GII) carried out the survey on December 3, 2004.

The field work for the west abutment was started on November 4, 2004 with a visual inspection by David F. Wood Consulting Ltd. (DWC) of the exposed rock face and evaluation of the rock coring program. Subsequent field drilling was carried out on December 15 and 16, 2004 and January 15, 2005.

The scope of the field work drilling comprised 25 boreholes designated by the 2-100 series of numbers. The boreholes were drilled to depths of 0.0 to 11.6 m at the locations shown on Drawing 1, appended. Further details are summarized in the following table.



LOCATION	BOREHOLE NO.	DEPTH (m)		
		AUGER	ROCK CORE <sup>(1)</sup>	TOTAL
West Approach	2-107	0.7	-	0.7
West Abutment	2-101	1.2	-	1.2
	2-102	1.2	-	1.2
	2-103	0.5	3.3 <sup>(2)</sup>	3.8
	2-104	1.0	3.3 <sup>(2)</sup>	4.3
	2-105	0.0	4.1 <sup>(1)</sup>	4.1
	2-106	0.0	-	0.0
	2-108	0.4	-	0.4
	2-109	0.7	3.0 <sup>(2)</sup>	3.7
	2-110	0.4	-	0.4
Centre Pier	2-111	11.6	-	11.6
	2-112	7.9	3.1 <sup>(1)</sup>	11.0
East Abutment	2-113	6.7	-	6.7
	2-114	1.2	3.1 <sup>(1)</sup>	4.3
	2-115	5.8	-	5.8
	2-116	2.1	-	2.1
	2-117	5.8	3.0 <sup>(1)</sup>	8.8
	2-118	3.0	-	3.0
	2-119	3.7	-	3.7
	2-120	4.7	3.5 <sup>(1)</sup>	8.2
	2-121	3.3	-	3.3
	2-123	1.4	-	1.4
	2-124	2.4	-	2.4
	2-125	2.2	-	2.2
East Approach	2-122	6.6	-	6.6

(1) NQ diamond rock coring

(2) BQ diamond rock coring

We also refer to Appendix A for the logs of previous boreholes drilled by GAL including the relevant results of their laboratory testing. The boreholes drilled by GAL are identified as boreholes 2-2, 2-3 and 2-5. These boreholes extended to depths from 5.2 to 12.2 m, including the core depth.



Tulloch Engineering Ltd. (TEL) staked the alignment of Stephenson Road No. 2 at the structure location. Peto MacCallum Ltd. (PML) selected the positions of the boreholes along the staked alignment and determined the ground surface elevations at the borehole locations. TEL provided the following temporary benchmarks (TBM) established on existing ground level at the working points (WP) for each of the foundation units:

TBM	DESCRIPTION	ELEVATION (*)
TBM1	Existing ground at west abutment WP	319.5
TBM2	Existing ground at centreline pier WP	313.8
TBM3	Existing ground at east abutment WP	313.8

(\*) Geodetic, metric

The boreholes were advanced by various methods, as required by accessibility and prevalent weather limitations. The ten boreholes located west of Highway 11 were advanced manually and/or with solid stem augers powered by portable equipment. The remaining fifteen boreholes were advanced using continuous flight hollow stem augers powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff.

Representative samples of the soils were recovered at frequent depth intervals. In the boreholes advanced with conventional drill rigs, the samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Where possible the relative density of the soils was also estimated from the rate of advance of the augers in unsampled boreholes and by manual examination in the boreholes sampled by hand.

Three boreholes at the east abutment, one borehole at the centre pier and four boreholes at the west abutment were extended 3.0 to 4.1 m into the bedrock using NQ and BQ diamond rock coring equipment supplemented by NW casing. The core length of borehole 2-105 drilled about 1.5 m behind the exposed rock face of the west abutment was extended from the standard 3.0 m length to 4.1 m to further evaluate the rock condition for potential previous blasting damage. Photographs of the site and selected rock cores are shown in Appendix B. The boreholes were backfilled in accordance with the MTO guidelines for borehole abandonment procedures using a bentonite/cement mixture grout.



The exposed rock at the west abutment and the four rock cores obtained at the same location were examined and logged by DWC. The report prepared by DWC contains a detailed description and photographs of the rock cores from the west abutment. A copy of the report is attached in Appendix C.

The rock surface profile under the east abutment and centre pier alignments was checked with Seismic Refraction Soundings (SRS) using the services provided by GII. A detailed description of the method, limitations and the results are described on the copy of the report that is enclosed as Appendix D. The buried soil/rock interface line determined with SRS was calibrated to a known rock depth proven previously with a borehole core at each of the two foundation units surveyed.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes. The water levels in the piezometers installed previously by GAL were measured during the PML investigation and are summarized on the attached Table 1.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The laboratory test program comprised the following tests:

- Natural water content determinations (30)
- Sieve and hydrometer analyses (12)

The results of the laboratory natural water content determinations and grain size determinations are shown on the Record of Borehole sheets. Grain size distribution charts are presented on Figures 2-1 to 2-3. Atterberg limits were not carried out since the soils are considered non-plastic.

#### **4. SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations and groundwater observations. Refer also to Appendices C and D for the rock assessment report for the west abutment and the SRS report for the center pier and east abutment, respectively.





The borehole locations and stratigraphic cross-sections prepared from the borehole data are presented on Drawings 1, 2 and 3.

The soil cover revealed in the boreholes varies from 7.9 to 11.6 m deep deposits at the centre pier, 1.2 to 6.7 m deep deposits at the east abutment and shallow soil cover less than 1.2 m thick at the west abutment. The soil cover at the east and centre foundation units generally comprises localized fill or peat/topsoil deposits covering cohesionless sand/silt/silty sand layers mantling bedrock. The soil cover at the west abutment comprises peat, topsoil and sand units over bedrock. At the west abutment, bedrock is exposed at two borehole locations and was exposed on a 5 m high rock cut facing Highway 11.

#### **4.1 Fill**

A surficial layer of fill occurs in borehole 2-2, drilled on the Highway 11 centreline median. The fill comprises loose to compact brown silty sand to sand trace silt and was 3.1 m thick, extending to elevation 310.8.

#### **4.2 Peat and Topsoil**

Surficial deposits of peat and topsoil are present in all the east and west abutment boreholes (except where rock was encountered surficially, boreholes 2-105 and 2-106) and in the approach embankment boreholes 2-107 and 2-122 and 2-5. The peat and topsoil layers are 100 to 700 mm thick and extend from elevations 314.1 to 314.4 (east approach); from elevations 313.3 to 314.0 (east abutment); from elevations 318.5 to 320.5 (west abutment); and elevation 321.1 at the west approach borehole.

#### **4.3 Sand/ Silty Sand**

A deposit of cohesionless sand with varying silt content occurs at the surface in boreholes 2-111 and 2-112 and below the fill, peat or topsoil units in the remaining boreholes, except boreholes 2-105 and 2-106 where bedrock is present at the surface and boreholes 2-103, 2-107, 2-108 and 2-110 where the peat/topsoil layers overlay directly the bedrock. The sandy soils extend to depths varying from 0.7 to 7.0 m, elevations 306.6 to 319.9, with the thicker deposits at the east abutment and centre pier. The relative density of the sandy soils is typically loose to compact (N=8 to 17) with localized very loose (N=3) and dense (N=33, 42) zones.



The particle size distribution of typical samples of these soils is shown of Figure 2-1. Similar grain size distribution charts from GAL boreholes 2-2 and 2-3 are shown in Appendix A. The water content ranges widely from 4 to 43% and is typically in the 8 to 10% range at the east abutment; 15 to 22% at the centre pier; and about 43% at the west abutment.

Cobbles and boulders are also encountered within the sand and gravel stratum above the bedrock in boreholes 2-101, 2-102, 2-104 and 2-109 (west abutment) and 2-121 near the ground surface (east abutment). Possible boulders could be present at other locations between the boreholes.

#### **4.4 Silt/ Sandy Silt**

Deposits of cohesionless silt trace to some sand and sandy silt are present below the sand unit. The silty materials extend to the 3.0 to 6.7 m termination depth of boreholes 2-113, 2-115, 2-117 to 2-119, 2-121 and 2-122 drilled at the east abutment and approach embankment. In boreholes 2-111, 2-112 and 2-2 (centre pier) the materials extend to depths from 7.2 to 10.2 m, elevations 303.4 to 307.1. At the boreholes 2-120 and 2-5 (east abutment and approach embankment) the silty soil units extend to 3.7 and 5.2 m depths, elevations 310.1 and 309.8, respectively. The silt soils are typically compact (N=11 to 28) with localized dense zones (N=33).

The particle size distribution charts of the silt are presented on Figure 2-2. Sand and silt samples from GAL boreholes 2-5 and 2-2, respectively are shown in Appendix A. The silty soils are non-plastic based on manual examination. The water content determinations in the deposits ranged from 19 to 24%, indicating wet soil conditions.

#### **4.5 Silty Sand/ Gravelly Sand**

Lower level deposits of cohesionless sand with silt and gravelly sand occur below the silt/sandy silt soils at the east abutment borehole 2-120 to 4.7 m depth, elevation 309.1 and in the centre pier boreholes 2-111, 2-112 and 2-2 extending to 7.9 to 11.6 m depths, elevations 302.0 to 306.4. The soil was classified as glacial till in the previous GAL borehole 2-2. The soils exhibit a variable very loose (N=3) to very dense (N=60) relative density.



The particle size distribution charts of a gravelly sand sample and sand with gravel are shown on Figure 2-3. The water content varies from 11 to 17%, indicating moist to wet conditions.

#### **4.6 Bedrock**

The bedrock comprises a dark grey to black biotite gneiss in the east abutment, centre pier boreholes and two of the four boreholes of the west abutment (2-104 and 2-109). Boreholes 2-103 and 2-105 revealed dark greyish green amphibolite bedrock. The rock is typically fresh/unweathered and exhibits medium to high strength in boreholes 2-114, 2-117 and 2-120 and high to very high strength in the remaining cored boreholes. A detailed description of the rock cores retrieved from boreholes 2-103, 2-104, 2-105, 2-109, 2-112, 2-114, 2-117 and 2-120 is provided in Table A and summarized on the record of borehole logs. The rock in boreholes 2-2 and 2-3 is described as biotite granite gneiss, which is considered consistent with the descriptions on the 2-100 series boreholes.

At the east abutment, the bedrock surface was confirmed by rock coring or inferred by refusal at depths of 1.2 to 6.7 m, elevations 306.7 to 312.7, indicating a maximum surface level difference of 6.0 m between borehole locations (1.6 m along the west face, 1.5 m across the center, 1.2 m across the east face, 3.9 m between the east face and center of the pier and 3.6 m between the center of and west face of the footing). The slope of the bedrock surface between boreholes is typically less than 20°, locally 46° between boreholes at the northwest corner (borehole 2-114). The outline of the buried rock line surveyed with RSS by GII is shown in Appendix D. The survey shows a continuous rock line along the proposed abutment and no sharp level changes or discontinuity. Photographs of the rock core taken in east abutment borehole 2-117 are shown on Plates 6 and 7, Appendix B.

In the centre pier boreholes, the soil/bedrock interface is confirmed by a minimum of 3 m of rock coring at depths varying from 7.9 to 11.6 m elevations 302.0 to 306.4, for an elevation difference of 4.4 m. The slope of the bedrock surface between boreholes is about 16°. The SRS survey of the rock surface along the centerline of the center pier shows a continuous rock line and no discontinuities. (Refer to the GII report in Appendix D for details).



Bedrock is exposed at the west abutment at boreholes 2-105 and 2-106 and was confirmed by rock coring or inferred by refusal at typically shallow depth from 0.4 to 1.2 m in the remaining west abutment boreholes. The exposed rock and the inferred soil/rock interface are found at levels ranging from elevations 318.1 to 320.3. The elevation difference is about 2.2 m between boreholes. A photograph of the rock core taken in the west abutment borehole 2-105 is shown on Plate 5, Appendix B. Additional photographs of the west abutment cores are included in Appendix C.

In the 2-100 series boreholes, the measured core recovery varies typically between 90 and 100%, with three isolated values of 89, 81 and 88% in boreholes 2-103, 2-104 and 2-109, respectively. The RQD determined from the rock cores is typically greater than 90% (range of 43 to 98%) at the east abutment boreholes 2-114, 2-117, 2-120 and 2-3 indicating excellent quality rock with local zones of poor to fair quality rock. The range of RQD for the centre pier rock in boreholes 2-112 and 2-2 is between 60 and 98%, indicating a fair to excellent quality rock. The west abutment rock RQD measured in boreholes 2-103, 2-104, 2-105 and 2-109 varies from 56 to 100%, indicating fair to excellent quality.

The condition of the west abutment rock was evaluated in detail by DWL (refer to Appendix C report). In summary, the rock is typically strong to very strong with estimated uniaxial compressive strength in the 100 MPa range. The assessment indicated that the rock exhibits some localized and surficial blasting damage; otherwise it is considered massive to blocky.

#### **4.7 Groundwater**

Groundwater strikes were observed in the boreholes during or upon completion of drilling and the groundwater was also measured in the piezometers installed in 2001 by GAL. The summary of the piezometer readings is shown on the attached Table 1. The water strikes during the drilling varied from 1.2 to 4.8 m in the 2-100 series boreholes and indicated the presence of surface water and pervious zones in the subsoil. The piezometer readings indicate that the depth to the groundwater is currently at 3.7 m depth, elevation 310.2 in borehole 2-2, drilled at the centre pier. The borehole 2-3 piezometer installed in the east abutment was dry. The variations of the water level were noted since installation in 2001 are considered to be caused by seasonal fluctuations and precipitation patterns.


## 5. CLOSURE

The field work was carried out under the supervision of Messrs. F. Portela, Senior Technician and M. Rapsey, Senior Technician and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Foundation Engineer. Marathon Drilling Co. Ltd. and Landcore Drilling Limited supplied the soil and rock drilling equipment.

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., and reviewed by Mr. D. W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Yours very truly,

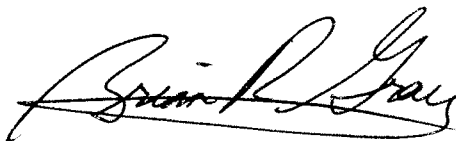
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MTO Designated Contact



CN/DWK:lr-mi



TABLE A  
ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
2-103	1	0.5 - 1.7	90	56	0.5 - 3.8	AMPHIBOLITE: Slightly weathered, massive to blocky, dark greyish green, coarse grained, strong, amphibolite with no dominant mineralogical trend. Fractures are iron stained with some minor infilling, planar and rough, at 20 to 60° to the core axis (tca) and with random orientations.
	2	1.7 - 2.5	100	78		
	3	2.5 - 3.5	89	89		
	4	3.5 - 3.8	100	100		
2-104 (*)	2	1.0 - 1.9	93	93	1.1 - 1.1	Very coarse-grained pegmatite vein with iron stained margins at 60 and 65° tca. Zone of more closely spaced fractures. Iron stained, random orientations.
	3	1.9 - 2.8	100		1.4 - 1.8	
	4	2.8 - 3.5	100		1.0 - 4.3	
	5	3.5 - 4.3	81			
					1.0 - 1.6	
2-105	1	0.0 - 0.4	100	90	0.0 - 4.1	Coarser grained with sporadic development of pegmatite, dark greenish grey. No significant structural defects.
	2	0.4 - 0.9	100	78		
	3	0.9 - 2.6	100	95		
	4	2.6 - 4.1	94	84		
					3.3 - 3.5	Fractured zone, angular, rust stained rock fragments from 10 mm to 50 mm across. Possible fault zone, but no indication of relative movement (slickensides).

Originated: FP  
Compiled: D.F. Wood  
Checked: CN



**TABLE A**  
**ROCK CORE DESCRIPTION**

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
2-109 (*)	2	0.7 - 1.5	100	92	0.7 - 3.7	BIOTITE GNEISS: Fresh to slightly weathered, massive, speckled black and grey, medium grained, strong, biotite gneiss, with variable development of fresh, massive, pinkish grey, coarse-grained, very strong pegmatite; to 15 mm crystals throughout hole. Inclined gneissic banding at 35-48° tca, curved, irregular, very rough. Pegmatite is more brittle and broken, but stronger than the gneiss. Veins cross foliation.
	3	1.5 - 2.1	100			
	4	2.1 - 2.7	88			
	5	2.7 - 2.9	100			
	6	2.9 - 3.7	95			
					1.0 - 1.6	Coarser grained with sporadic development of pegmatite, dark greenish grey. No significant structural defects.
					1.6 - 3.1	Pegmatite veining predominates

NOTES: RQD: Rock Quality Designation  
 (\*) Modified RQD for BQ size core  
 Refer to Appendix C for additional description of rock core for west abutment of structure.

Originated: FP  
 Compiled: D.F. Wood  
 Checked: CN



TABLE A  
 ROCK CORE DESCRIPTION

CORE RECOVERY				CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)
2-112	7	7.9 - 9.4	100	92	7.9 - 11.0
	8	9.4 - 11.0	100	98	
2-114	2	1.2 - 2.7	100	97	1.2 - 4.3
	3	2.7 - 4.3	100	98	
2-117	5	5.8 - 7.3	100	95	5.8 - 8.8
	6	7.3 - 8.8	100	73	
2-120	5	4.7 - 6.1	98	92	4.7 - 8.2
	6	6.1 - 7.6	91	43	
	7	7.6 - 8.2	100	96	

NOTES: RQD: Rock Quality Designation  
 (\*) Modified RQD for BQ size core  
 Refer to Appendix C for additional description of rock core for west abutment of structure.

Originated: MR/FP  
 Compiled: JFW  
 Checked: CN



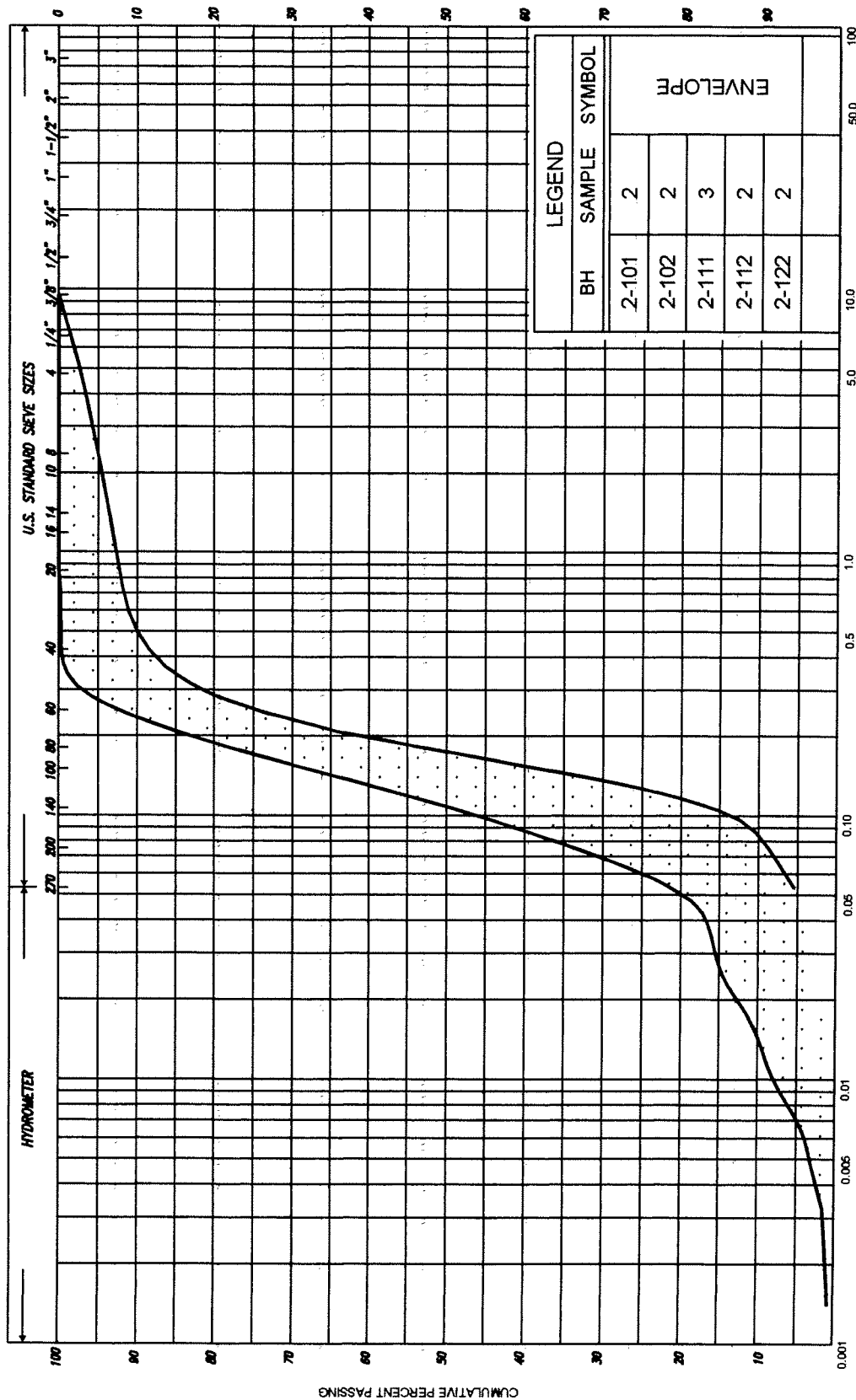


**TABLE 1**  
**PIEZOMETER WATER LEVEL READINGS**

Date	Borehole No. 2 - 2		Borehole No. 2- 3	
	Ground Surface Elevation 313.9		Ground Surface Elevation 314.19	
	Depth (m)	Elevation	Depth (m)	Elevation
* 2001-02-27	Frozen	-	2.5	311.6
* 2001-04-21	2.0	311.9	1.2	312.9
** 2004-10-25	4.2	309.7	Dry	<310.9
** 2004-11-12	3.7	310.2	Dry	<310.9

\* From Golder Associates Limited Report

\*\* Measured by Peto MacCallum Ltd.



CLAY		SILT & CLAY		FINE SAND		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		COR. BLS.		U.S. BUREAU	
CLAY		SILT		FINE		MEDIUM		COARSE		GRAVEL		CORBLES		COR. BLS.		U.S. BUREAU	
CLAY		SILT		FINE		MEDIUM		COARSE		GRAVEL		CORBLES		COR. BLS.		U.S. BUREAU	
CLAY		SILT		FINE		MEDIUM		COARSE		GRAVEL		CORBLES		COR. BLS.		U.S. BUREAU	
CLAY		SILT		FINE		MEDIUM		COARSE		GRAVEL		CORBLES		COR. BLS.		U.S. BUREAU	

GRAIN SIZE DISTRIBUTION

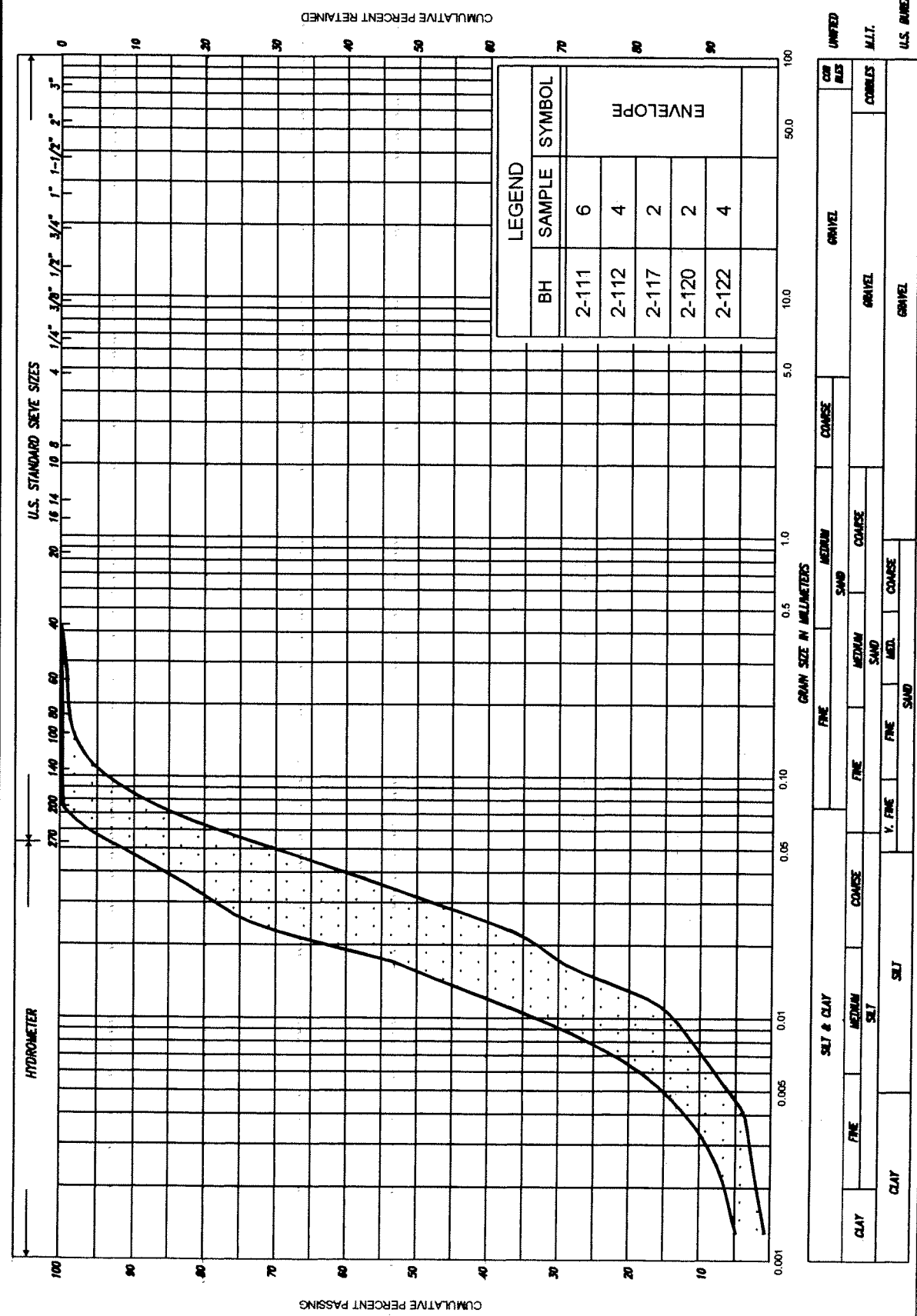
SILTY SAND, trace clay to

SAND trace silt trace gravel

FIG No. 2-1

HWY 11

W.P. No. 5040-00-01



**GRAIN SIZE DISTRIBUTION**  
 SILT, trace to some sand trace clay

FIG No. 2-2

HWY 11

W.P. No. 5040-00-01



## EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 31mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

DYNAMIC CONE PENETRATION TEST. CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (15mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3M ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_v$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN.  
THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

ROD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50 mm	50 - 300 mm	0.3 m - 1 m	1 m - 3 m	> 3 m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

## FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

## STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$\sigma'_v$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$E$	kPa	MODULUS OF LINEAR DEFORMATION
$G$	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

## MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_d$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	$m^2/s$	COEFFICIENT OF CONSOLIDATION
$H$	$m$	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{VO}$	$kPa$	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_{p'}$	$kPa$	PRECONSOLIDATION PRESSURE
$C_t$	$kPa$	SHEAR STRENGTH
$c'$	$kPa$	EFFECTIVE COHESION INTERCEPT
$\phi'$	$^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
$C_u$	$kPa$	APPARENT COHESION INTERCEPT
$\phi_u$	$^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	$kPa$	RESIDUAL SHEAR STRENGTH
$\tau_f$	$kPa$	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $\frac{-U}{\tau_f}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e - e_{min}}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$w$	1, %	WATER CONTENT	$D$	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	$n$ PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$i$	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL				$j$	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE			

RECORD OF BOREHOLE No 2-101

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 563 N; 319 551 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler + Solid Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE December 15, 2004 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
319.3	Ground Surface							20	40	60	80	100		
0.0	Peat and topsoil		1	GS	-		319							
318.8	Dark brown													
0.5	Sand, some silt, trace gravel, cobbles and boulders		2	GS	-									3 84 13 0
318.1	Brown Wet													
1.2	End of borehole													
	Refusal on probable bedrock													

RECORD OF BOREHOLE No 2-102

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 578 N; 319 550 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler + Solid Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE December 15, 2004 CHECKED BY *LS*




SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
321.1	Ground Surface														
0.0	Peat and topsoil														
320.5	Dark brown		1	GS	-										
0.6	Sand, some silt, trace gravel, cobbles and boulders		2	GS	-									0 89 (11)	
319.9	Brown Moist														
1.2	End of borehole														
	Refusal on probable bedrock														
	* Borehole dry on completion of drilling														
	Borehole advanced with portable equipment														

RECORD OF BOREHOLE No 2-103

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 563 N; 319 553 E ORIGINATED BY E.P.  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers + NQ Rock Coring COMPILED BY E.P.  
DATUM Geodetic DATE December 15 & 16, 2004 CHECKED BY C4

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED      + FIELD VANE									
								● QUICK TRIAXIAL      x LAB VANE									
319.0	Ground Surface						20	40	60	80	100					GR SA SI CL	
0.0	Peat and topsoil																
318.5	Dark brown																
0.5	Amphibolite Bedrock																
	Strong		1	RC NQ	REC 90%		318									RQD 56%	
	Slightly weathered		2	RC NQ	REC 100%		317									RQD 78%	
	Fair becoming good to excellent quality		3	RC NQ	REC 89%		316									RQD 89%	
	Refer to Table A for detailed description.		4	RC NQ	REC 100%											RQD 100%	
315.2	End of borehole																
3.8																	
	* 2004 12 16																
	Borehole charged with drilling water																
	Borehole advanced with portable equipment																
	Rock Core logged by David F. Wood Consulting Ltd.																



RECORD OF BOREHOLE No 2-104

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 579 N; 319 553 E ORIGINATED BY E.P.  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers + BQ Rock Coring COMPILED BY E.P.  
DATUM Geodetic DATE January 15, 2005 CHECKED BY CL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
320.5	Ground Surface							20	40	60	80	100					
0.0	Peat and topsoil		1	GS	-		320										
320.0	Dark brown																
0.5	Sand, with silt cobbles and boulders																
319.5	Brown Moist																
1.0	Biotite Gneiss Bedrock		2	RC BQ	REC 93%		319										
	Strong to very strong																
	Fresh to slightly weathered		3	RC BQ	REC 100%		318										Combined
	Excellent quality																
	Refer to Table A for detailed description.		4	RC BQ	REC 100%		317										RQD 93%
			5	RC BQ	REC 81%												
316.2	End of borehole																
4.3																	
	* Borehole charged with drilling water																
	Borehole advanced with portable equipment																
	BQ Core size recovered due to portable equipment limitations																
	Rock Core logged by David F. Wood Consulting Ltd.																



1 of 1

**METRIC**[illegible]

RECORD OF BOREHOLE No 2-107										1 of 1		METRIC					
W.P. 5040-00-01			LOCATION Co-ords. 5 003 567 N; 319 533 E			ORIGINATED BY F.P.											
DIST 52 HWY 11			BOREHOLE TYPE Manual Hand Sampler + Solid Stem Augers			COMPILED BY F.P.											
DATUM Geodetic			DATE December 15, 2004			CHECKED BY C4											
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 x LAB VANE					W <sub>p</sub>	W	W <sub>L</sub>		
321.8	Ground Surface																
0.0	Peat and topsoil																
	Dark brown		1	GS	-												
321.1																	
0.7	End of borehole																
	Refusal on probable bedrock																
	* Borehole dry on completion of drilling																
	Borehole advanced with portable equipment																

## METRIC

20  
15 — 5 (% STRAIN AT FAILURE)  
10

**METRIC**

20  
15 — 5 (%) STRAIN AT FAILURE  
10

RECORD OF BOREHOLE No 2-110										1 of 1		METRIC	
W.P. 5040-00-01		LOCATION Co-ords. 5 003 571 N; 319 555 E				ORIGINATED BY F.P.							
DIST 52 HWY 11		BOREHOLE TYPE Manual Sampling				COMPILED BY F.P.							
DATUM Geodetic		DATE December 15, 2004				CHECKED BY C							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							GROUND WATER CONDITIONS	ELEVATION SCALE
319.2	Ground Surface												
0.0	Peat and topsoil		1	GS	-								
318.8	Dark brown												
0.4	End of borehole												
	Refusal on probable bedrock												
	* Borehole dry on completion of drilling												

RECORD OF BOREHOLE No 2-111

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 573 N; 319 591 E ORIGINATED BY M.R.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY M.R.  
DATUM Geodetic DATE November 01, 2004 CHECKED BY Cz

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED	● QUICK TRIAXIAL							+ FIELD VANE	x LAB VANE	
313.6	Ground Surface						20	40	60	80	100							
0.0	Sand with silt Compact Brown Damp to light brown		1	SS	11													
			2	SS	14													
	trace silt Loose Wet		3	SS	6													
308.7																		
4.9	Silty Sand, stratified Loose Brown Wet		4	SS	8													
	Dense		5	SS	42													
306.6																		
7.0	Silt, some sand, trace clay Compact Brown Wet																	



## 1 of 1

**METRIC**

SOIL PROFILE	SAMPLES			DYNAMIC CONE PENETRATION			
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ON\_MOT VER3 04TF006\_SR2.GPJ ON\_MOT.GDT 17/05/05 1:59:53 PM

+7, X<sup>5</sup>: Numbers refer to Sensitivity

20  
15—○—5 (% STRAIN AT FAILURE)  
10

RECORD OF BOREHOLE No 2-113										1 of 1		METRIC				
G.W.P. 5040-00-01			LOCATION Co-ords. 5 003 579 N; 319 627 E			ORIGINATED BY F.P.										
DIST 52 HWY 11			BOREHOLE TYPE Hollow Stem Augers			COMPILED BY F.P.										
DATUM Geodetic			DATE October 25, 2004			CHECKED BY CL										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE								
313.4	Ground Surface															
0.0	Topsoil															
0.1	Sand with silt Loose Brown Moist		1	SS	8											
311.9			2	AS	-											
1.5	Silty sand Loose Brown Moist															
310.3	Grey Wet															
3.1	Silt trace sand Compact Grey Wet		3	AS	-											
306.7	End of borehole Refusal on probable bedrock															
6.7																

## METRIC

20  
15 — 5 (%) STRAIN AT FAILURE  
10

## 1 of 1

### METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 580 N; 319 629 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 25, 2004 CHECKED BY CS

[illegible]

RECORD OF BOREHOLE No 2-116

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 595 N; 319 629 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 25, 2004 CHECKED BY C4

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
314.0	Ground Surface							20	40	60	80	100					
0.0	Topsoil																
0.2	Probable sand with silt Compact Brown Moist																
							313										
311.9							312										
2.1	End of borehole Refusal on probable bedrock																
	<p>* Borehole dry</p> <p>Relative density of soils was estimated from rate of advance of augers.</p> <p>Soil descriptions based on examination of auger cuttings.</p>																



## METRIC

**+7, X5:** Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2-119										1 of 1		METRIC					
G.W.P. 5040-00-01			LOCATION Co-ords. 5 003 587 N; 319 627 E			ORIGINATED BY F.P.											
DIST 52 HWY 11			BOREHOLE TYPE Hollow Stem Augers			COMPILED BY F.P.											
DATUM Geodetic			DATE October 25, 2004			CHECKED BY <i>CG</i>											
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 kN/m <sup>3</sup>	GR SA SI CL
								20 40 60 80 100					W <sub>p</sub> W W <sub>L</sub>				
313.6	Ground Surface																
0.1	Topsoil																
	Probable sand with silt																
	Loose Brown Moist to Compact																
311.8																	
1.8	Probable silty sand trace clay																
	Compact Grey Moist																
310.5																	
3.1	Probable silt trace sand																
309.9	Compact Grey Wet																
3.7	End of borehole																
	Refusal on probable bedrock																
	* 2004 10 25																
	▽ Water level observed during drilling																
	Relative density of soils was estimated from rate of advance of augers.																
	Soil descriptions based on examination of auger cuttings.																



RECORD OF BOREHOLE No 2-120										1 of 1		METRIC		
G.W.P. 5040-00-01			LOCATION Co-ords. 5 003 587 N; 319 629 E			ORIGINATED BY F.P.								
DIST 52 HWY 11			BOREHOLE TYPE Hollow Stem Augers + NQ Rock Coring			COMPILED BY F.P.								
DATUM Geodetic			DATE October 26, 2004			CHECKED BY CL								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	T <sub>N</sub> VALUES			SHEAR STRENGTH kPa						
313.8	Ground Surface													
0.1	Topsoil Sand, with silt		1	SS	8									
	Loose Brown Moist													
312.7	Silt, trace sand trace clay													
1.1	Dense Grey Moist to wet		2	SS	33									
310.1			3	SS	33									
3.7	Gravelly sand, trace silt													
	Dense Grey Moist to wet		4	SS	10/ 3cm									
309.1	Biotite Gneiss Bedrock													
4.7	Medium to high strength		5	RC NQ	REC 98%									
	Slightly weathered to unweathered													
	Excellent to poor quality													
	Refer to Table A for detailed description		6	RC NQ	REC 91%									
			7	RC NQ	REC 100%									
305.6	End of borehole													
8.2														
<p>* 2004 10 26</p> <p>▽ Water level observed during drilling</p> <p>▼ Water level measured after drilling</p> <p>Borehole charged with drilling water</p>														

RECORD OF BOREHOLE No 2-121										1 of 1		METRIC	
G.W.P. 5040-00-01			LOCATION Co-ords. 5 003 588 N; 319 632 E			ORIGINATED BY F.P.							
DIST 52 HWY 11			BOREHOLE TYPE Hollow Stem Augers			COMPILED BY F.P.							
DATUM Geodetic			DATE October 25, 2004			CHECKED BY <i>CL</i>							
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	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL	
313.9	Ground Surface												
0.0	Topsoil												
0.1	Sand with silt cobbles in upper 0.2 m												
	Compact Brown Moist		1	AS			313						
311.8	Silt trace sand						312						
2.1	Compact Grey Moist		2	AS			311						
310.6	End of borehole												
3.3	Refusal on probable bedrock												
	* 2004 10 25												
	▽ Water level observed during drilling												
	▼ Water level measured after drilling												
	Soil descriptions based on examination of auger cuttings.												

RECORD OF BOREHOLE No 2-122

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 591 N; 319 645 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 25, 2004 CHECKED BY *u*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
314.7	Ground Surface							20	40	60	80	100					
0.0	Topsoil		1	SS	9												
314.1	Silty sand, trace clay						314										
0.6	Dense Brown Moist		2	SS	33		313										0 66 32 2
312.3	Silt, trace clay trace sand						312										
2.4	Compact Grey Wet		3	SS	24		311										
							310										0 1 94 5
			4	SS	28		309										
308.1	trace gravel		5	SS	20/ 5cm												
6.6	End of borehole																
	Refusal on probable bedrock																
	* 2004 10 25																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

RECORD OF BOREHOLE No 2-123

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 591 N; 319 627 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 26, 2004 CHECKED BY CS


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT $\gamma$ kNm <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100						
313.9	Ground Surface																
0.0	Topsoil																
0.2	Probable silty sand Loose Brown Moist																
312.5																	
1.4	End of borehole Refusal on probable bedrock																
	* Borehole dry																
	Relative density of soils was estimated from rate of advance of augers.																
	Soil descriptions based on examination of auger cuttings.																

RECORD OF BOREHOLE No 2-124

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 595 N; 319 628 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 26, 2004 CHECKED BY CH

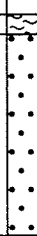
SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
313.8	Ground Surface																
0.0	Topsoil																
0.2	Probable silty sand																
	Loose Brown Moist to compact																
311.4	End of borehole																
2.4	Refusal on probable bedrock																
	* Borehole dry																
	Relative density of soils was estimated from rate of advance of augers.																
	Soil descriptions based on examination of auger cuttings.																

RECORD OF BOREHOLE No 2-125

1 of 1

METRIC

G.W.P. 5040-00-01 LOCATION Co-ords. 5 003 595 N; 319 630 E ORIGINATED BY F.P.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY F.P.  
DATUM Geodetic DATE October 26, 2004 CHECKED BY *cl*

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											
314.1	Ground Surface						20	40	60	80	100	20	40	60					
0.0	Topsoil						314												
0.2	Probable sand with silt						313												
	Loose Brown Moist to compact						312												
2.2	End of borehole Refusal on probable bedrock  * Borehole dry  Relative density of soils was estimated from rate of advance of augers.  Soil descriptions based on examination of auger cuttings.																		

(Legend Continued)

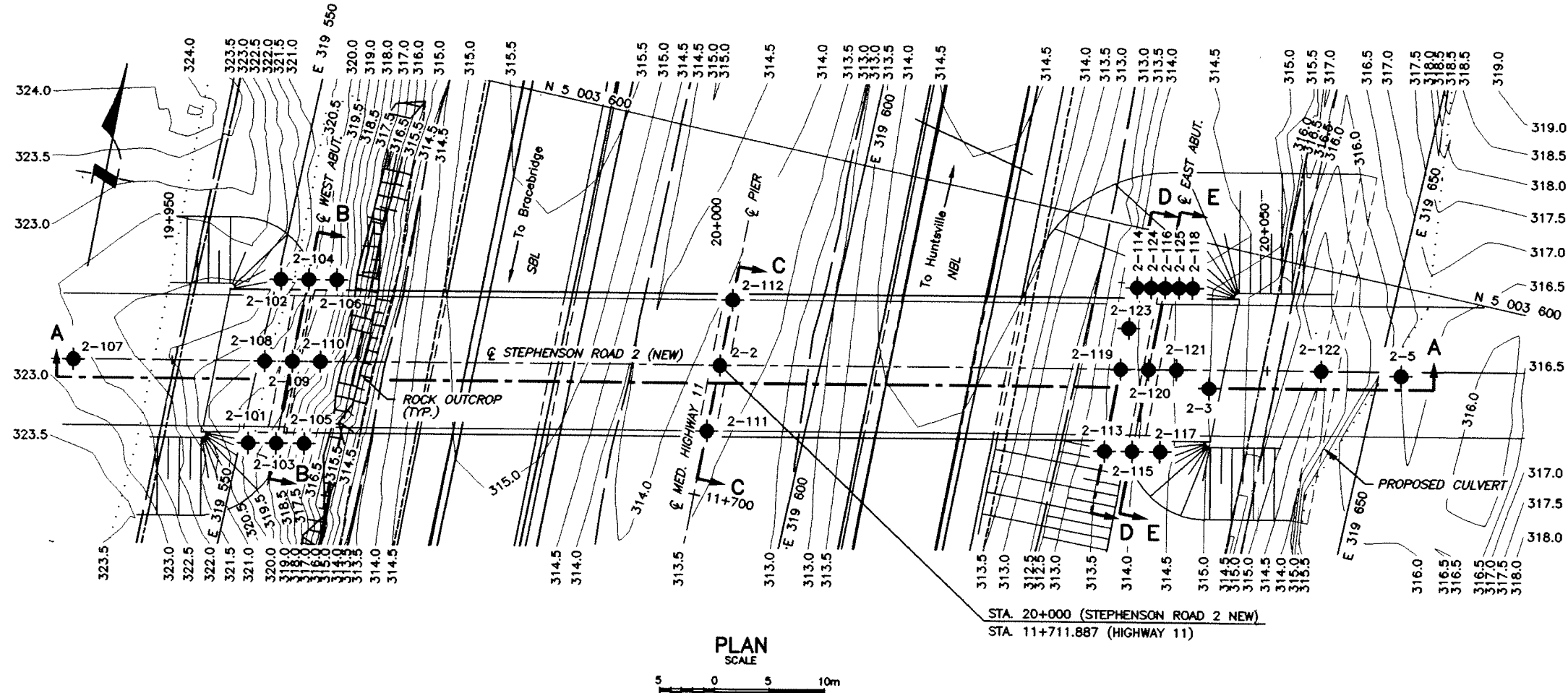
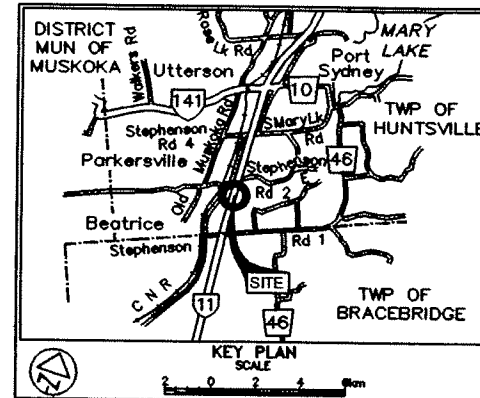
BH No	ELEVATION	CO-ORDINATES NORTH	EAST
2-108	319.9	5 003 570	319 551
2-109	319.5	5 003 571	319 553
2-110	319.2	5 003 573	319 555
2-111	313.6	5 003 573	319 591
2-112	314.3	5 003 585	319 591
2-113	313.4	5 003 579	319 627
2-114	313.9	5 003 595	319 627
2-115	313.5	5 003 580	319 629
2-116	314.0	5 003 595	319 629
2-117	313.7	5 003 580	319 632
2-118	314.1	5 003 596	319 631
2-119	313.6	5 003 587	319 627
2-120	313.8	5 003 587	319 629
2-121	313.9	5 003 588	319 632
2-122	314.7	5 003 591	319 645
2-123	313.9	5 003 591	319 627
2-124	313.8	5 003 595	319 628
2-125	314.1	5 003 595	319 630

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

CONT No  
WP No 5040-00-01  
HIGHWAY 11  
STEPHENSON ROAD No. 2 UNDERPASS  
BOREHOLE LOCATIONS



**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60° Cone, 475 J / blow)		
	W L at time of investigation Oct 2004		
	Head		
	ARTESIAN WATER Encountered		

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
2-2	313.9	5 003 579	319 591
2-3	314.1	5 003 587	319 635
2-5	315.0	5 003 592	319 652
2-101	319.3	5 003 563	319 551
2-102	321.1	5 003 578	319 550
2-103	319.0	5 003 563	319 553
2-104	320.5	5 003 579	319 553
2-105	318.6	5 003 564	319 556
2-106	320.3	5 003 579	319 555
2-107	321.8	5 003 567	319 533

(Legend Continues)

NOTES:

- BOREHOLES 2-2, 2-3, AND 2-5 WERE DRILLED BY GOLDER ASSOCIATES; REPORT REFERENCE NO. 011-1104 DATED APRIL 2001
- REFER TO DRAWING NO. 2 AND 3 FOR SECTIONS A-A, B-B C-C, D-D AND E-E.
- WL OCTOBER 2004 (2-100 SERIES BOREHOLES)  
WL FEBRUARY 2001 (2-2, 2-3 AND 2-5 BOREHOLES)

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

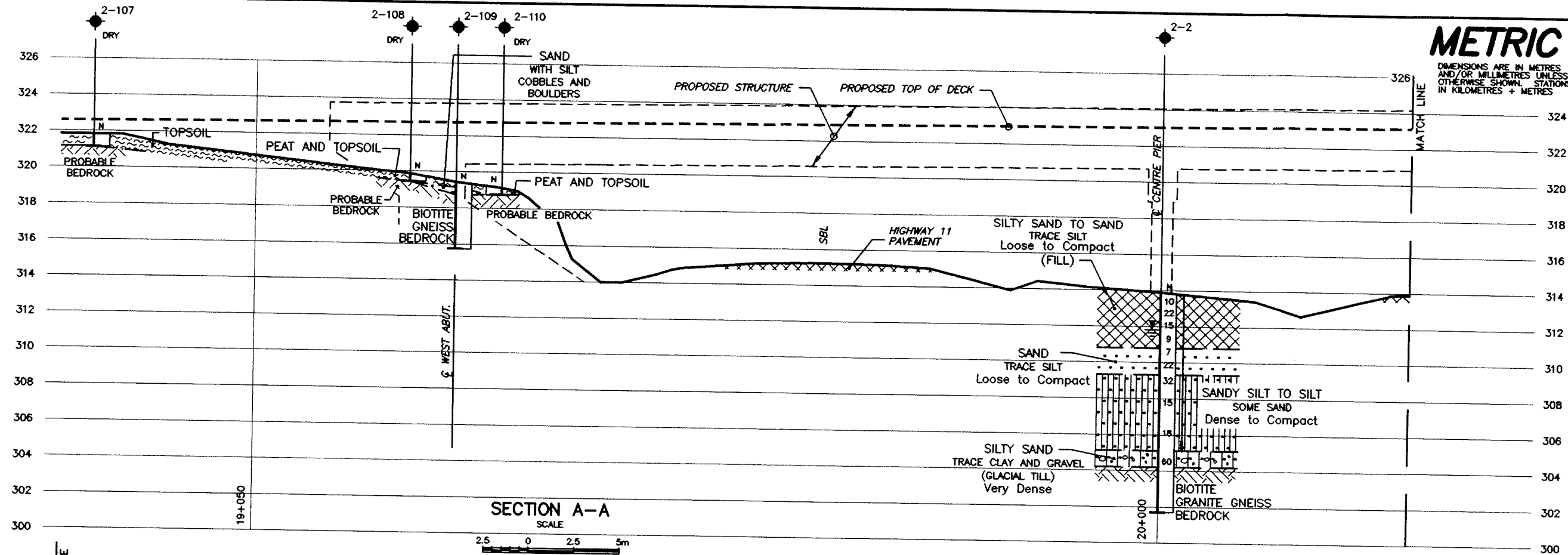
DATE	BY	DESCRIPTION

Geocore No. 31E-238

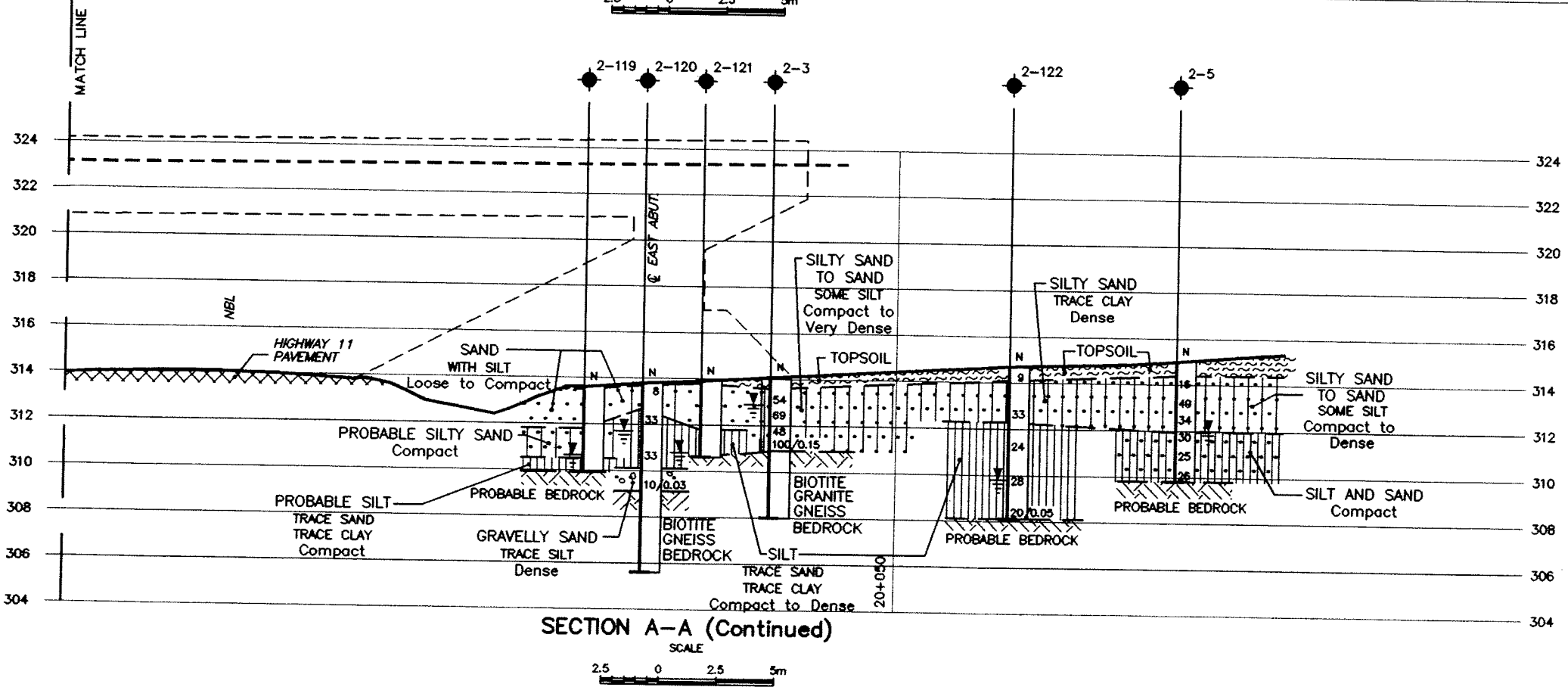
REVISED	NO	DATE	DESCRIPTION
	11		
	12		
	13		



REF No E-S5568-320GA001.dwg; h5568x81.dwg;  
h5568x82.dwg; October, 2004



**SECTION A-A**  
SCALE  
2.5 0 2.5 5m



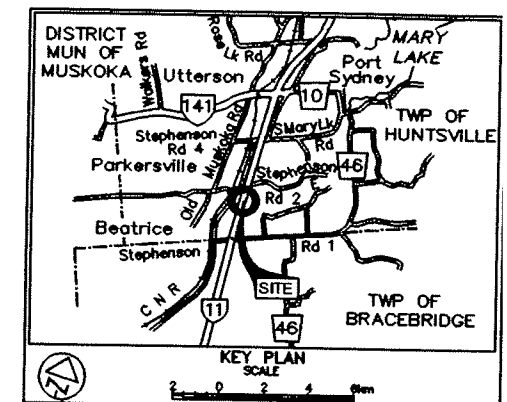
**SECTION A-A (Continued)**  
SCALE  
2.5 0 2.5 5m

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

CONT No  
WP No 5040-00-01  
HIGHWAY 11  
STEPHENSON ROAD No. 2 UNDERPASS  
SOIL STRATA

SHEET

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



**LEGEND**

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60° Cone, 475 J / blow)
- W L at time of investigation Nov 2004
- Head
- ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST

(Refer to drawing no. 1 for co-ordinates)

- NOTES:**
- BOREHOLES 2-2, 2-3, AND 2-5 WERE DRILLED BY GOLDER ASSOCIATES; REPORT REFERENCE NO. 011-1104 DATED APRIL 2001
  - REFER TO DRAWING NO. 1 FOR PLAN AND DRAWING NO. 3 FOR SECTIONS B-B, C-C, D-D AND E-E.
  - WL OCTOBER 2004 (2-100 SERIES BOREHOLES)  
WL FEBRUARY 2001 (2-2, 2-3 AND 2-5 BOREHOLES)
  - SECTIONS ARE PROVIDED SOLELY FOR ILLUSTRATIVE PURPOSES. REFER TO RECORD OF BOREHOLES FOR DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS, IN-SITU TEST DATA AND LABORATORY TEST RESULTS.

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



REF No E-S5568-320GA001.dwg; h5568xB1.dwg;  
h5568xB2.dwg; October, 2004

Georec No. 31E-238

DATE	BY	DESCRIPTION

DATE	BY	DESCRIPTION







## **APPENDIX A**

Previous Record of Boreholes and Laboratory Test Results

PROJECT 011-1104		<b>RECORD OF BOREHOLE No 2-2</b>		1 OF 1	<b>METRIC</b>
W.P. 62-96-00		LOCATION N 5003579; E 319591		ORIGINATED BY SB	
DIST 52 HWY 11		BOREHOLE TYPE 108mm Hollow Stem Augers		COMPILED BY DKB	
DATUM Geodetic		DATE Feb. 7/01		CHECKED BY ASP	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	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 80 100	W <sub>p</sub> W W <sub>L</sub>				
UNCONFINED + FIELD VANE QUICK TRIAXIAL x REMOULDED														
313.9	GROUND SURFACE													
0.0	Silty Sand to Sand, trace silt (Fill) Loose to compact Brown Moist		1	SS	10									
			2	SS	22									
			3	SS	15									
			4	SS	9									
310.8														
3.1	Sand, trace silt Loose to compact Brown Wet		5	SS	7									
			6	SS	22									
309.3														
4.6	Sandy Silt to Silt, some sand Dense to compact Grey Moist to wet		7	SS	32									
			8	SS	15									
			9	SS	18									
305.1														
8.8	Silty Sand, trace clay and gravel (Glacial Till) Very dense Grey Moist		10	SS	60									
304.2														
9.7	Biotite granite GNEISS Dark grey-black with white speckles and streaks, foliated Slightly weathered to fresh Medium jointed, coarse to very coarse grained, very to extremely strong Occasional pink, very coarse grained zones.  Bedrock cored from 9.7m to 12.2m depth.													
301.7	For bedrock coring details refer to Record of Drilling 2-2													
12.2	END OF HOLE													
	Note: 1. Water level measured in piezometer at 8.2m (El. 305.7m) upon completion of installation. 2. Water level frozen in piezometer at 0.1m above ground surface (El. 314.0m) on February 27, 2001. 3. Water level measured in piezometer at 2.0m depth (El. 311.9m) on April 21, 2001.													

ON MOT. OLD. 011-1104.GPJ ON MOT.GDT 5/304

+ 3, x 3. Numbers refer to      ○ 3% STRAIN AT FAILURE  
Sensitivity

PROJECT: 011-1104

## RECORD OF DRILLHOLE: 2-2

SHEET 2 OF 2

LOCATION: T15003570, E 010591

DRILLING DATE: February 7, 2001

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 BOMBARDIER

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No	PENETRATION RATE (mm/min)	COLLOID % RETURN	FRFX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED ST-STEPPED VN-VEIN S-SUCKENSIDED PL-PLANAR										SM-SMOOTH R-ROUGH ST-STEPPED C-CURVED			FL-FLEXURED UE-UNEVEN W-WAVY			BC-BROKEN CORE MB-MECH. BREAK B-BEDDING			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																						
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	CORRECTION CORRECTIONS	CORRECTIONS	CORRECTIONS	CORRECTIONS	CORRECTIONS	CORRECTIONS																																																																																																																																																																																																																																																																														
								TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION								TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION			TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE DESCRIPTION	TYPE AND SURFACE 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MISS ROCK 1104 ROCK GPJ GLDR CAN GOT 5304 MMZ

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: POG

PROJECT <u>011-1104</u>		RECORD OF BOREHOLE <b>No 2-3</b>		1 OF 1		<b>METRIC</b>	
W.P. <u>62-86-00</u>		LOCATION <u>N 5003587 E 319635</u>		ORIGINATED BY <u>SB</u>			
DIST <u>52</u> HWY <u>11</u>		BOREHOLE TYPE <u>108mm Hollow Stem Augers</u>		COMPILED BY <u>DKB</u>			
DATUM <u>Geodetic</u>		DATE <u>Feb. 1901</u>		CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
314.1	GROUND SURFACE															
0.0	Topsoil															
313.6																
0.5	Silty Sand to Sand, some silt Dense to very dense Brown, becoming gray below 2.3m depth Moist to wet		1	SS	54											
			2	SS	69											
			3	SS	48											
310.9			4	SS	100/15											
3.2	Biotite granite GNEISS Dark grey-black with white speckles and streaks, foliated Slightly weathered to fresh Medium jointed, coarse to very coarse grained, very to extremely strong Occasional pink, very coarse grained zones.  Bedrock cored from 3.4m to 6.1m depth.  For bedrock coring details refer to Record of Drillhole 2-3															
308.0																
6.1	END OF HOLE  Note: 1. Water level measured in piezometer at 2.5m depth (El. 311.6m) on February 27, 2001. 2. Water level measured in piezometer at 1.2m depth (El. 312.9m) on April 21, 2001.															

ON MOT. OLD 011-1104.GPJ ON MOT.GDT 5/3/04

PROJECT 011-1104

## RECORD OF DRILLHOLE: 2-3

SHEET 2 OF 2

LOCATION: N 5003587, E 319635

DRILLING DATE: February 19, 2001

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 BOMBARDIER

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No	PENETRATION RATE (mm/min)	COLLOR % RETURN	FR/FX-FRACTURE F-FAULT										3A-SMOOTH				FL-FLEXURED				9C-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
								CL-CLEAVAGE		J-JOINT		A-ROUGH		UE-UNEVEN		MB-MECH. BREAK		B-BEDDING		SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		C-CURVED				HYDRAULIC CONDUCTIVITY K <sub>f</sub> cm <sup>2</sup> /sec		DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
								TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 M	DP #11 CORE AHS	DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION		DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION		DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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MISS. ROCK 1104 ROCK GPJ GLDR. CAN GDT 5/304 MMZ

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: POG

PROJECT 011-1104		<b>RECORD OF BOREHOLE No 2-5</b>				1 OF 1		<b>METRIC</b>	
W.P. 62-86-00		LOCATION N 5003592; E 319652				ORIGINATED BY SB			
DIST 52 HWY 11		BOREHOLE TYPE 108mm Hollow Stem Augers				COMPILED BY DKB			
DATUM Geodetic		DATE Feb 19/01				CHECKED BY ASP			

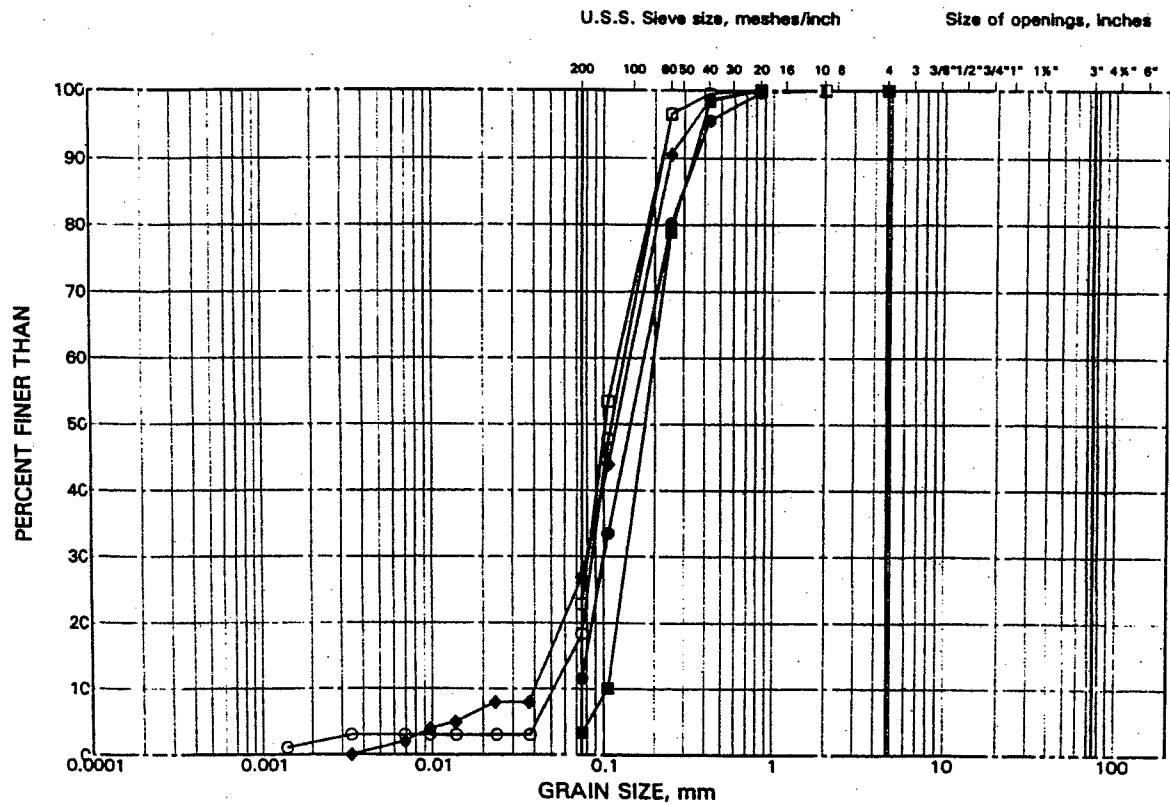
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
315.0	GROUND SURFACE													
0.0	Topsoil with occ. cobbles													
314.4														
0.6	Silty Sand to Sand, some silt Compact to Dense Brown, becoming grey below 1.5m depth Moist		1	SS	18									
			2	SS	49									
			3	SS	34									
311.9														
3.1	Silt and Sand Compact Grey Wet		4	SS	30									
			5	SS	25									
			6	SS	28									
309.8														
5.2	END OF BOREHOLE													
	Note: 1. Water level measured in open borehole at 3.0m depth (El. 312.0m) upon completion of drilling.													

ON MOT OLD 011-1104 GPJ ON MOT GOT 5/3/04

+ 3 . x 3. Numbers refer to      0 3% STRAIN AT FAILURE  
Sensitivity

# GRAIN SIZE DISTRIBUTION Sand to Silty Sand

FIGURE 1



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

## LEGEND

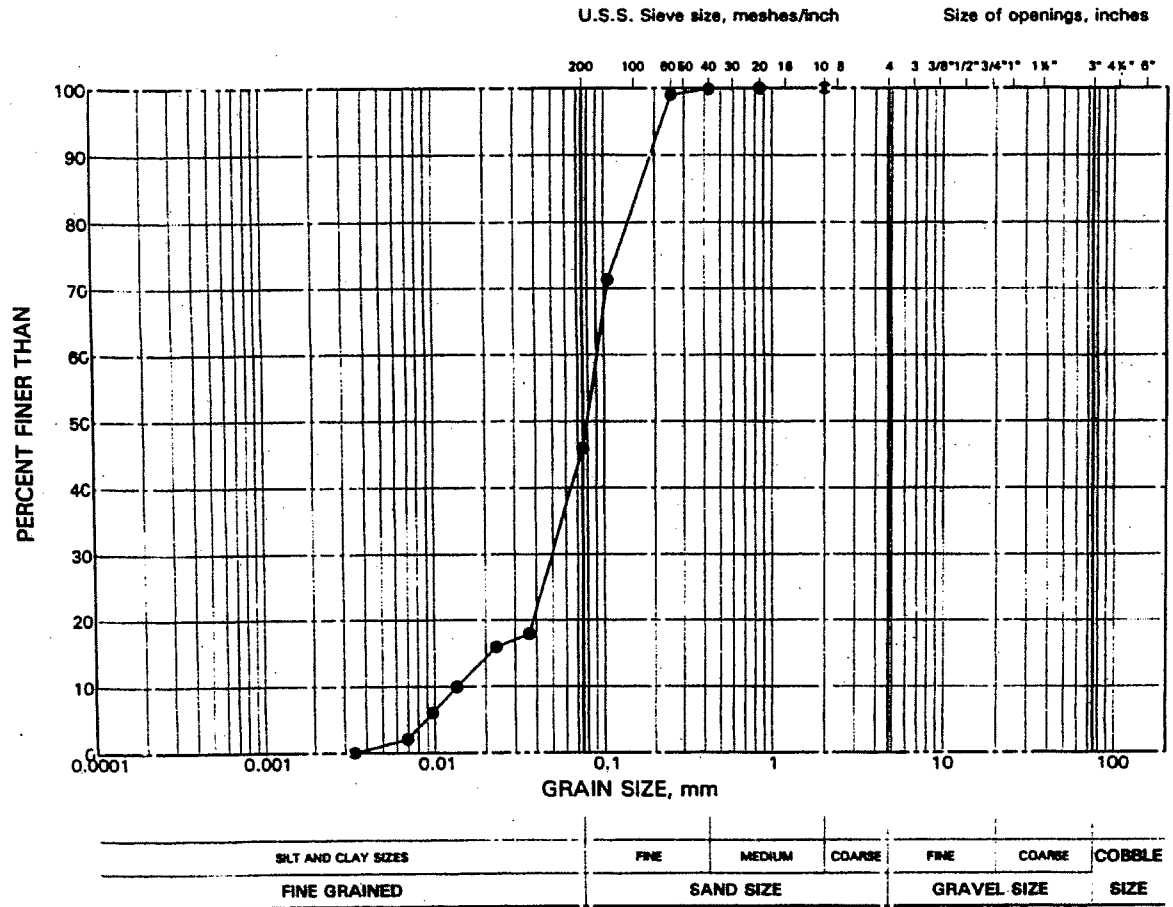
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1-8	3	318.6
■	2-2	5	310.2
◆	2-3	1	311.8
○	3-1	5	306.0
□	3-2	3	308.5



# GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE 2

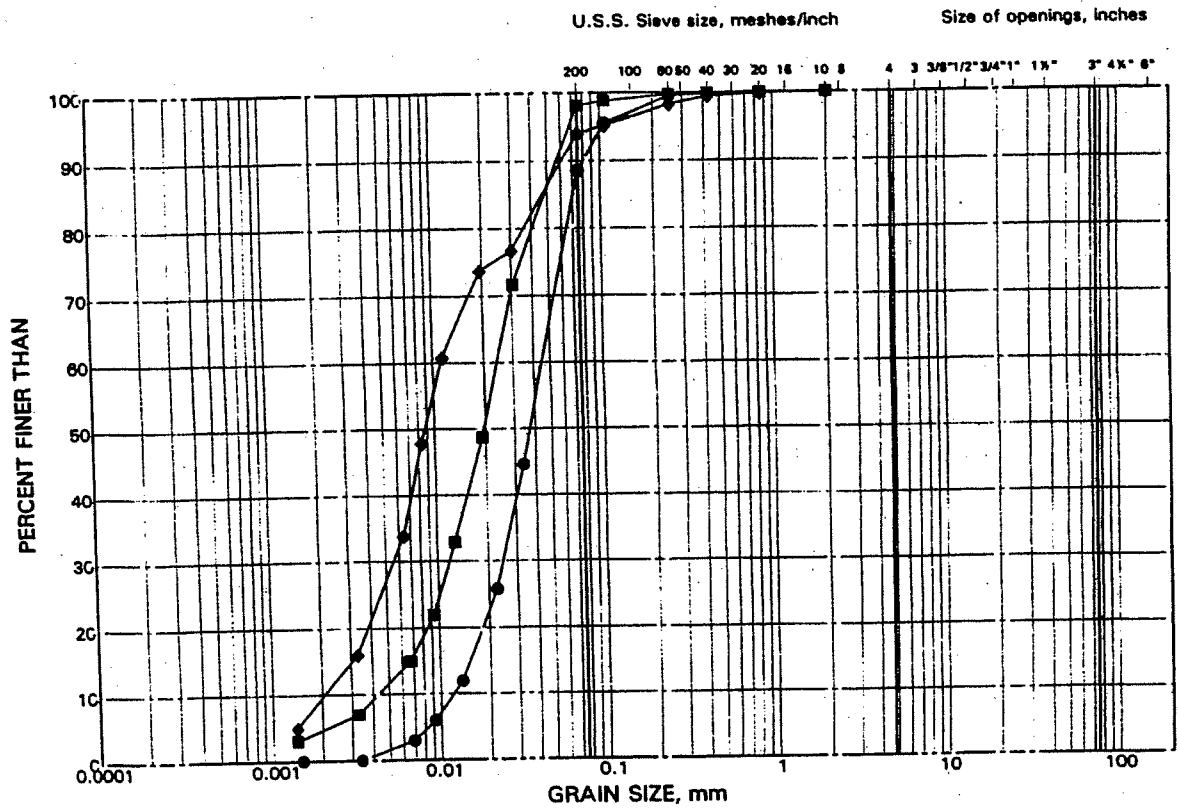


## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	2-5	5	310.6

# GRAIN SIZE DISTRIBUTION Silt

FIGURE 3



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

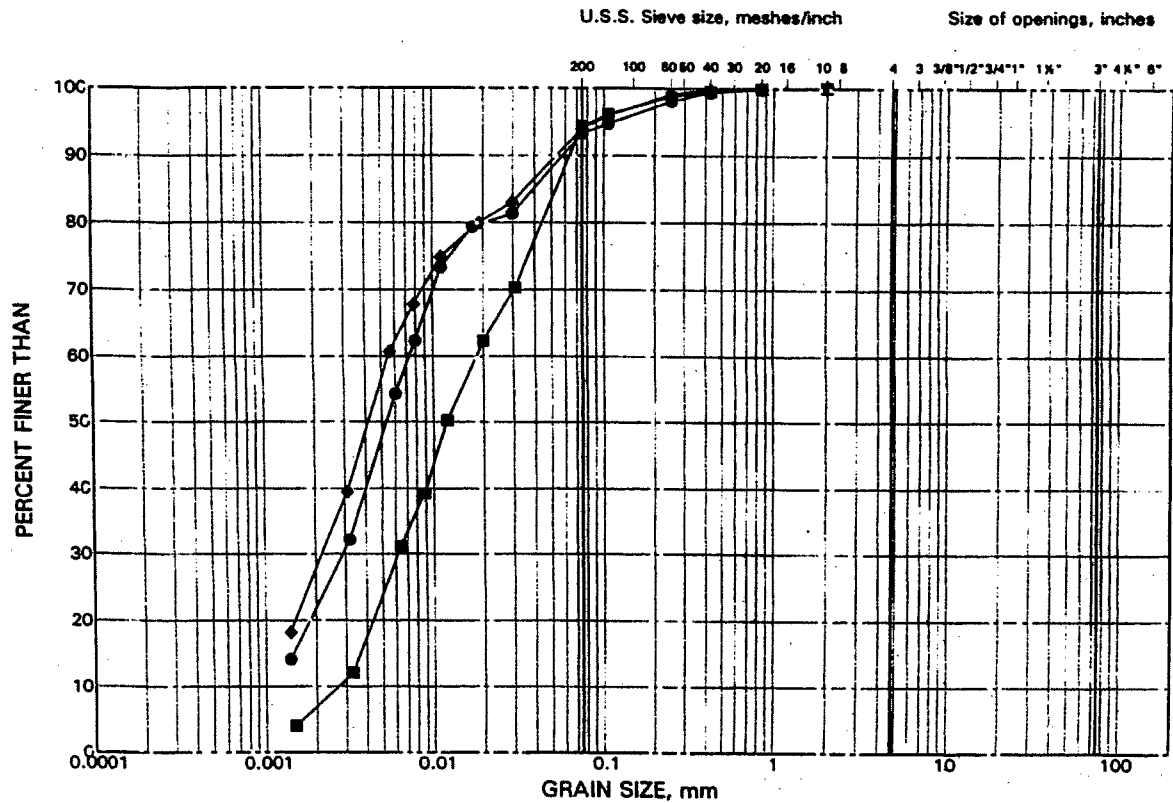
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	2-2	8	307.2
■	3-1	10	299.1
◆	3-6	6	305.0

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clayey Silt

FIGURE 6



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	3-1	8	302.2
■	3-2	7	304.7
◆	3-4	7	303.5

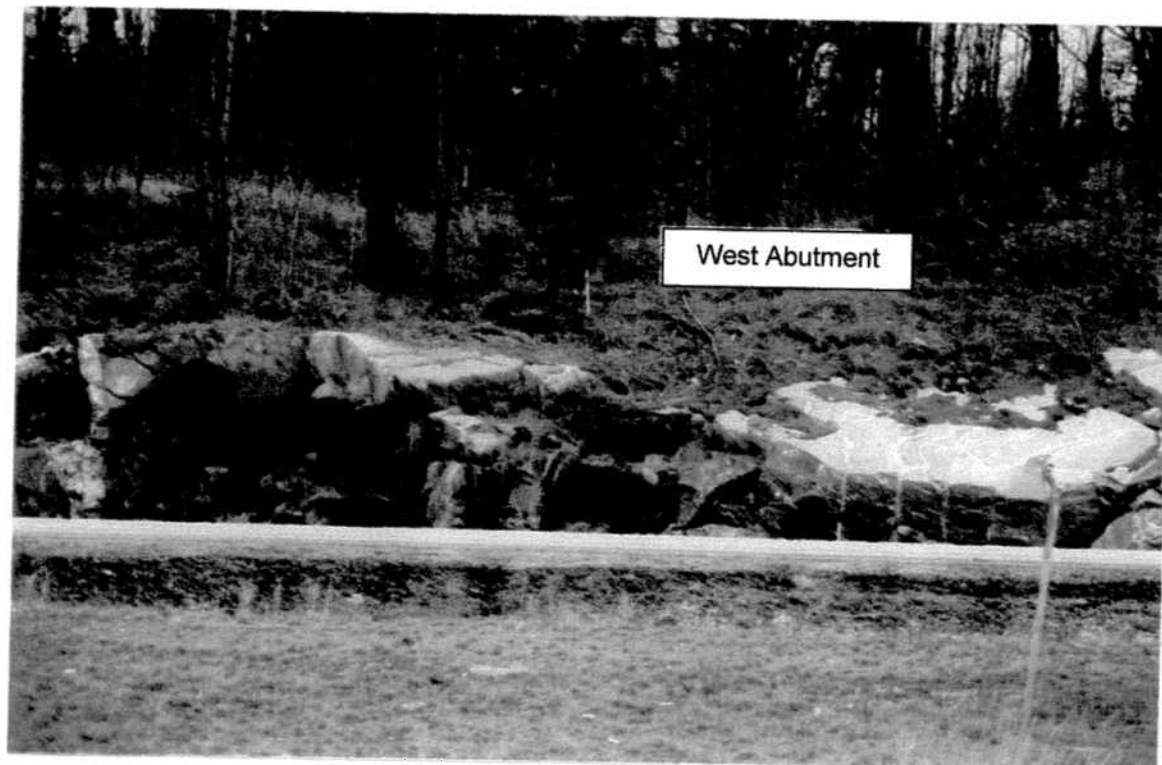


## **APPENDIX B**

Site and Rock Core Photographs



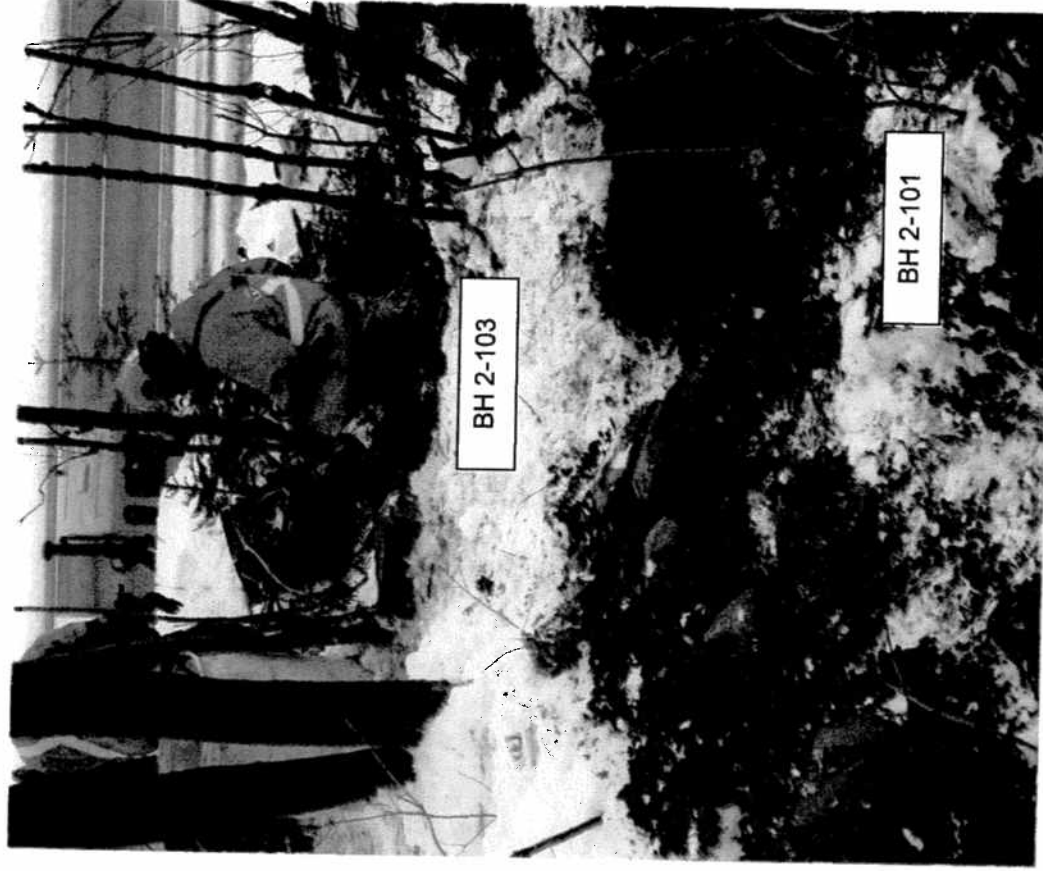
**Plate 1:** Stephenson Road # 2, East abutment WP stake – Centre pier and West abutment visible. (October 25/04)



**Plate 2:** Stephenson Road # 2 West abutment, facing west. (October 25/04)



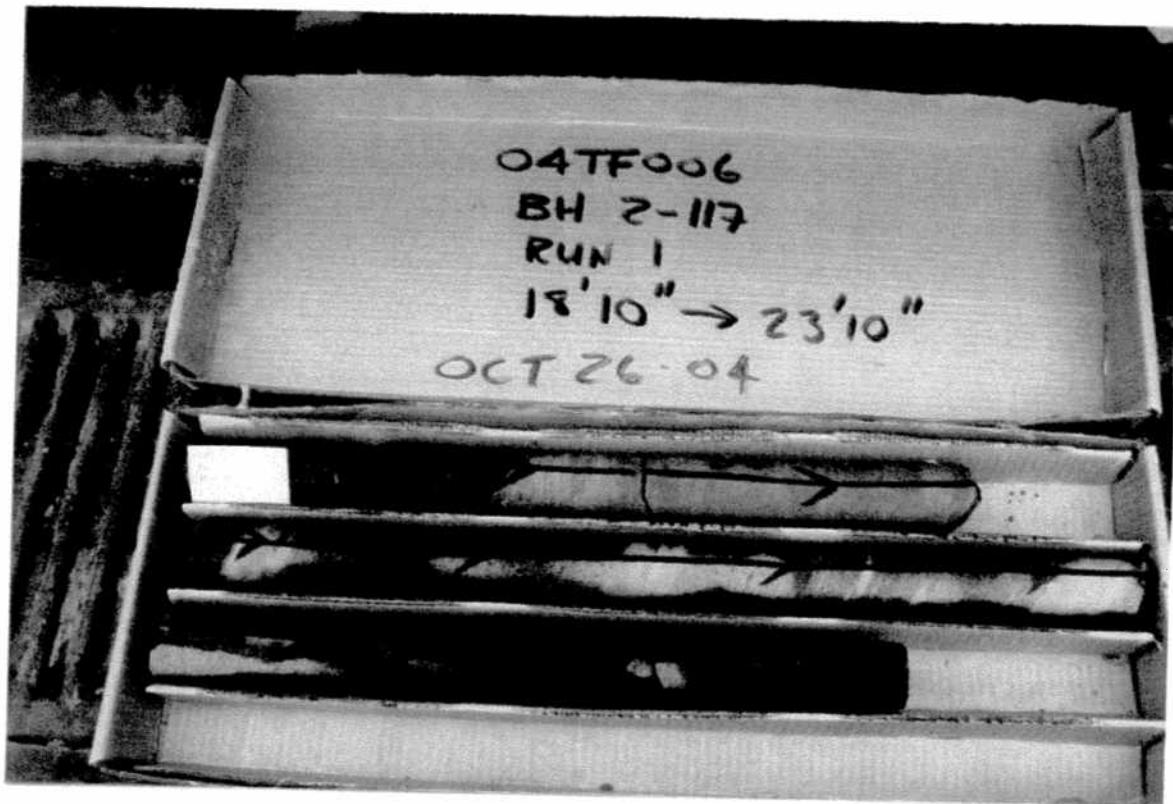
**Plate 3:** Looking east along boreholes 2-101, 2-103 and 2-105, drilled at the west abutment (December 15, 2004).



**Plate 4:** Looking east along same line as Plate 3.  
Note: cobbles and boulders removed by hand from boreholes 2-101 and 2-103.



**Plate 5:** Rock core from borehole 2-105. Refer to Appendix C for detailed description.



**Plate 6:** Stephenson Road # 2, East abutment core.



**Plate 7:** Stephenson Road # 2, East abutment core. (Continued from Plate 6)





## **APPENDIX C**

Rock Inspection Report  
(David F. Wood Consulting Ltd.)

# David F. Wood

DAVID F. WOOD CONSULTING LTD.  
55 GLOUCESTER COURT  
SUDBURY, ONTARIO  
CANADA P3E 5M2

consultant in engineering geology,  
rock engineering and  
shotcrete technology

Tel: +1 (705) 673-8080  
Fax: +1 (705) 673-0909  
e-mail: [info@dfwood.com](mailto:info@dfwood.com)

January 24, 2005

Peto MacCallum Limited  
165 Cartwright Avenue  
Toronto, Ontario  
M6A 1V5

Attention: Mr. Carlos M.P. Nascimento, P.Eng., Senior Consultant

**Re: Highway 11, Stephenson Road #2 Underpass,  
WP 5040-00-01, Site 42-327  
Foundation Investigation Report**

Dear Carlos;

## Introduction

This Foundation Investigation Report describes the work carried out for the evaluation of rock mass conditions for the west abutment of the proposed bridge carrying Stephenson Road #2 over Highway 11 south of Huntsville, Ontario. The project falls under MTO WP 5040-00-01 and is referred to as Site 42-327.

The first part of the project involved reviewing work carried out previously to investigate this site. Peto MacCallum (PetoMac) provided part of a report by Golder Associates Ltd. (GAL) who worked at the site and reported under Job Number 011-1104 in April 2001. Following this review, a site visit was made on 2<sup>nd</sup> November 2004 to gain first hand experience of the rock mass at the site. A drilling program was then conducted and the cores from four (4) holes were inspected on 18<sup>th</sup> January 2005.

## Prior Knowledge

The part of the GAL report relating to this site was reviewed. On the basis of drilling carried out in February 2001, GAL summarized subsoil conditions for the centre pier and the east abutment, but not the west abutment area. The depth of soil cover at the pier was 9.7 m while at the east abutment this depth had decreased to 3.2 m. The bedrock description given for the two cored holes in that investigation matches the descriptions determined during both the site investigation and core logging exercises carried out as part of the current assignment.

---

*David F. Wood Consulting Ltd.*

## Site Inspection

The site was visited on 2<sup>nd</sup> November 2004. The weather was cool and damp with a temperature range of 0-5°C. No construction work was carried out at the time of the site visit.

### Bridge structure foundations

The bridge at this location will span the two-lane divided Highway 11 and carry local traffic now using Stephenson Road #2 across the major highway. The area around the west abutment for the overpass structure was reviewed; it had not been drilled off at the time of the visit. The site had been surveyed prior to the inspection and the bridge alignment was staked in the field. The abutment is designed to be approximately 15 metres wide and 5 metres deep. The centreline stake observed in the field was marked "WP1 5003", and was set back from the face of a rock cut 5 metres high, itself set back 8 metres from the edge of the travelled portion of the highway, see Images 1 to 3.

### Rock Mass Conditions

The rock mass at the site can be described as: fresh to faintly weathered, massive, dark grey with white and pink speckles and bands, coarse to very coarse grained, very strong, biotite granitic gneiss. There is very limited natural jointing; the gneissic banding or foliation dips steeply to the northeast, but is not always represented as a discontinuity to the north of the proposed centreline. The metamorphic foliation is quite contorted at the site with folding and undulating observed, see Image 6. This site is considered to be founded on a good quality rock mass.

To the south of the site, the rock mass becomes weaker, more micaceous and blockier; the grain size increases and the presence of amphiboles changes the rock mass conditions. However, this part of the rock mass is not within the proposed abutment footprint.

### Excavated Conditions

The original rock mass conditions, prior to any work at the site, were not observed, but it might be assumed that they included an irregular surface configuration of rounded, glacially eroded knobs of rock. This would be due to the differential erosion of the various mineral components of the underlying rock. The rock face present shows some drill hole half barrels to the north of the abutment that are clean and do not exhibit signs of over-blasting. The folding mentioned above occurs to the south of the proposed centreline with some blast damage.

Limited rock removal by machine scaling (either using a hoe ram or a backhoe excavator) will be needed to ensure that the rock face is modified to a maintenance-free condition, Image 4. There is a "nose" of rock close to the south end of the proposed structure with a large feldspar porphyry dike that cannot be considered to have significant load bearing capacity without scaling, see Image 5. Further commentary regarding foundation conditions is provided in the Foundation Design Report.

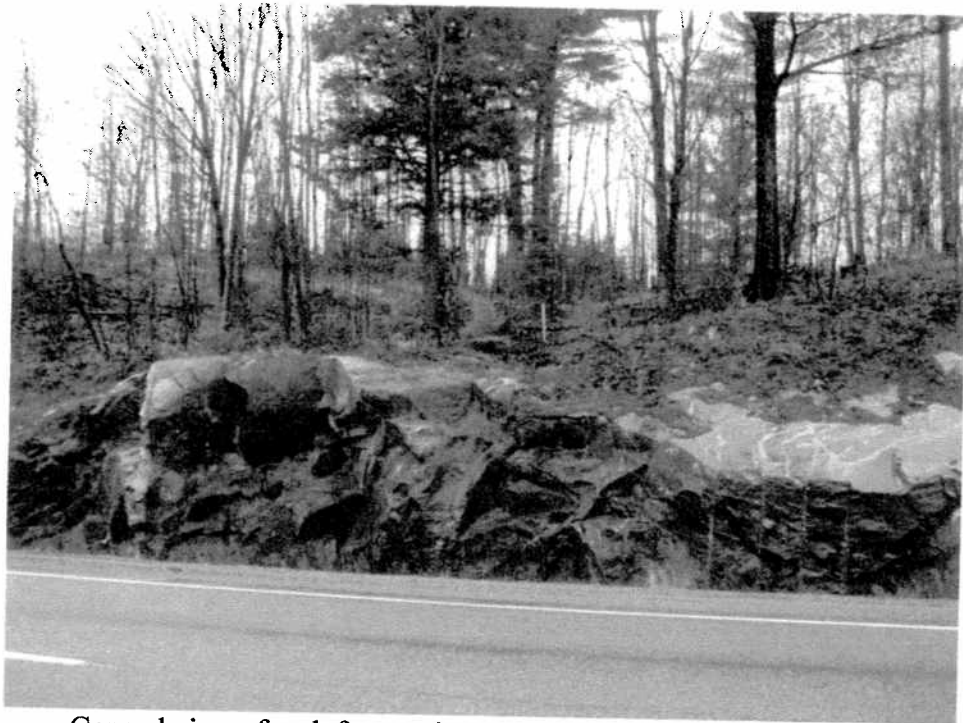


Image 1 General view of rock face at site. Note centreline stake, drill hole half barrels to north (right of image) and more broken rock mass to south.



Image 2 View along crest of slope looking south. Note set back of abutment centerline from crest of rock face, and vegetation/soil cover materials to remove prior to final inspection.

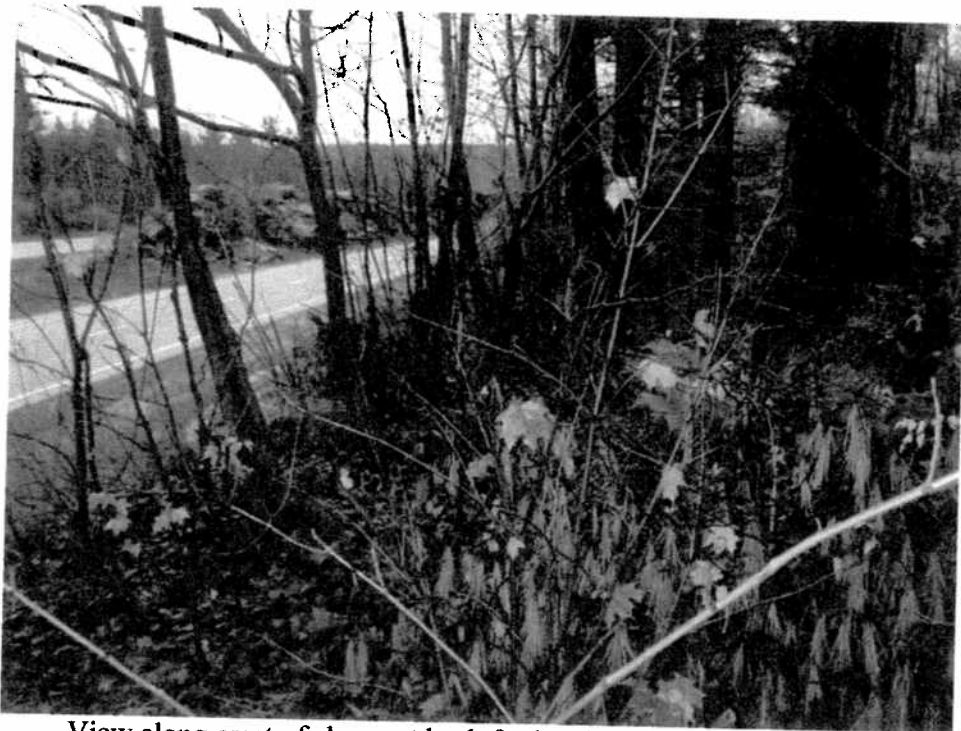


Image 3. View along crest of slope set back further from rock face. Centreline stake is visible in middle of image. Trees and soils will be removed prior to final inspection.



Image 4. Southern extremity of rock face under proposed abutment. Note fractured rock mass and curved, folded gneissic banding. Limited scaling will be needed to remove the loose rock from the face.



Image 5. Wide feldspar pegmatite vein to south of abutment site. The upper block shown in this image may need to be removed, depending on the exact configuration of the proposed abutment.



Image 6. View of contorted and folded gneissic banding at crest of rock face. Upper part of image is in the ditch adjacent to the shoulder of Highway 11.

## Rock coring program

In the investigation program for the west abutment, ten (10) boreholes were planned; nine covering the footprint of the proposed abutment (more or less) and one set back to the west on the approach. Four (4) of these holes were cored. Two holes, 2-105 and 2-103, were cored at nominal NQ size (core diameter approx. 43 mm) to the south of the proposed structure in coarse grained, dark coloured, possible amphibolite. Two holes, 2-109 and 2-104, were cored at nominal BQ size (core diameter approx. 36 mm) at the centre and to the north of the structure in medium grained, speckled, possible biotite gneiss with local development of very coarse grained pegmatite veins. All core exhibited intact uniaxial compressive strength in the range of approximately 100 MPa, i.e. "strong to very strong". RQD values vary but on close inspection, many of the core breaks are mechanically induced and interpreted RQD qualities are reasonably good. The rock mass penetrated by these holes appears to be a variable, high-grade metamorphic rock that is massive to blocky with minor weathering along natural discontinuity surfaces.

RQD values have been interpreted from the core reviewed as well as the original Peto MacCallum logs. The values presented here are considered to be accurate for the following reasons:

1. Only natural cores breaks have been considered; machine breaks or handling breaks have been ignored. Fresh breaks where mineral boundaries are clearly visible are usually unnatural fractures and should therefore be ignored. Natural fractures or joints are commonly seen to have smoother surfaces with some degree of iron staining.
2. Fractures at the end of each core run and associated with removing the core catcher were carefully evaluated, and ignored where appropriate.
3. Fractures that run parallel to the core axis are considered to be a 'sampling error' and are ignored in the calculation of RQD.
4. The original concept for RQD was based on NX core, with a diameter of 50 mm. The 'threshold piece length' of 100 mm is described by Deere (1964) was twice the core diameter. For BQ core the threshold piece length would be around 75 mm.

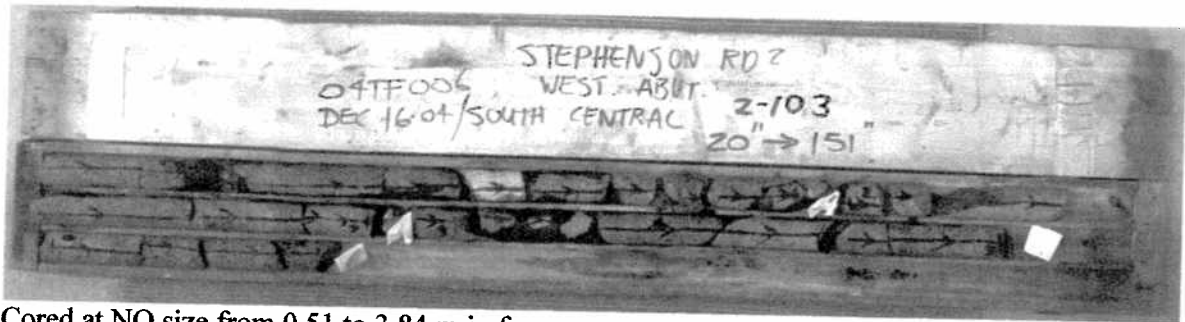
The rock mass quality as established by RQD would be described as "fair" for 1.2 metres of Hole 2-103, "good" for 1.7 metres of Hole 2-103 and 2.5 metres of Hole 2-105, and "excellent" for the remaining 8.4 metres of all four holes<sup>1</sup>.

Hole 2-105 is located to the south of the abutment and reasonably close to the crest of the rock mass excavation alongside Highway 11. It is noted that the cored hole was collared in bedrock. Hole 2-103 is also to the south of the abutment and set back from the excavated rock face; it was collared in soils. The rock core started at 0.5 m. Hole 2-109 is located at the intersection of the bridge and abutment centrelines, again set back from the rock mass excavation. The hole was collared in soils and the rock core was started at 0.7 m. The last core reviewed came from Hole 2-104 to the north of the proposed structure. It was collared in soils and the rock core started at 1.0 m. A review of the logs from all ten holes drilled at the site shows a general trend with the thickness of soil cover at the site increasing towards the west. Maximum soil cover depths are 1.2 metres across the western extremity of the proposed abutment. It appears that the bridge centreline has reduced soil cover depths to a maximum of only 0.7 metres.

---

<sup>1</sup> Measurements may not completely correspond with core. Logging was carried out in Imperial measurements and converted to Metric with one decimal place of precision.



**2-103**

Cored at NQ size from 0.51 to 3.84 m in four runs.

Core Run	Interval	Run length	Core recovery	RQD	Natural fract's
Run 1	0.51 – 1.72	1.22	1.09 – 90%	0.69 – 56%	8 – 9.82/m
Run 2	1.72 – 2.54	0.81	0.81 – 100%	0.69 – 78%	4 – 4.9/m
Run 3	2.54 – 3.45	0.91	0.81 – 89%	0.81 – 89%	5 – 5.5/m
Run 4	3.45 – 3.84	0.38	0.38 – 100%	0.38 – 100%	0 – 0/m

0.51 to 3.84 m (T.D.)

Slightly weathered, massive to blocky, dark greyish green, coarse grained, strong, amphibolite with no dominant mineralogical trend. Fractures are iron stained with some minor infilling, planar and rough, at 20 to 60° to the core axis (tca) and with random orientations.

1.10 to 1.14 m

Coarse-grained pegmatite vein with crosscutting, iron stained margins at 60 and 65° tca.

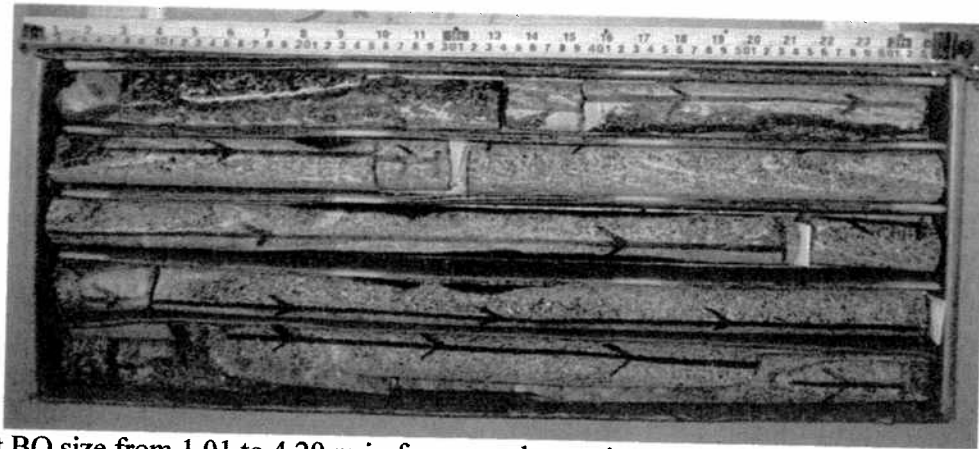
1.37 to 1.78 m

Zone of more closely spaced fractures. Iron stained, random orientations.



Core surface shows some grinding and diameter reduction. Core broken with single firm blow of a geological hammer along weakness associated with presence of micas – approx. uniaxial strength 75-100 MPa. Core stronger in pure uniaxial loading.



**2-104**

Cored at BQ size from 1.01 to 4.29 m in four runs, but no intervals given on "End of Run" slips. Core recovery data taken from Peto MacCallum logs.

Core Run	Interval	Run length	Core recovery	RQD	Natural fract
Run 1	1.01 – 1.9	0.79	93%	93%	1.3/m
Run 2	1.9 – 2.8	0.9	100%		
Run 3	2.8 – 3.5	0.7	100%		
Run 4	3.5 – 4.29	0.79	81%		

1.01 to 4.29 m (T.D.)

Fresh to slightly weathered, massive, speckled black and grey, medium grained, strong to very strong, biotite gneiss, with sub-vertical gneissic banding (sub-parallel tca) only locally developed as a natural discontinuity. Most fractures are irregular, rough to very rough,  $\approx 10^\circ$  tca.

1.01 to 1.63 m

Coarser grained with sporadic development of pegmatite, dark greenish grey. No significant structural defects.



The core does not show any inclined joints, and has very limited iron staining.

**2-105**

Cored at NQ size from 0 to 4.14 m in four runs.

Core Run	Interval	Run length	Core recovery	RQD	Natural frags
Run 1	0.00 – 0.38	0.38	0.38 – 100%	0.34 – 90%	3 – 7.9/m
Run 2	0.38 – 0.89	0.51	0.51 – 100%	0.39 – 78%	2 – 3.9/m
Run 3	0.89 – 2.57	1.68	1.68 – 100%	1.59 – 95%	6 – 3.6/m
Run 4	2.57 – 4.14	1.57	1.47 – 94%	1.32 – 84%	4 – 2.5/m + fz

0.00 to 4.14 m (T.D.)

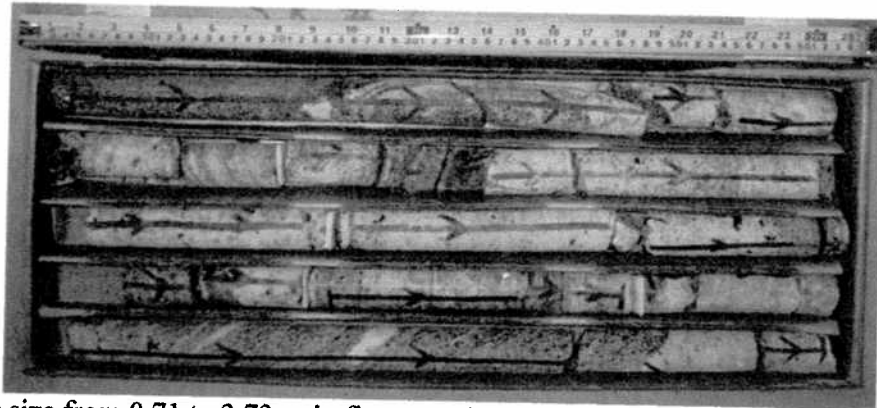
Slightly weathered, massive to blocky, dark greyish green, coarse grained, strong, amphibolite with no dominant mineralogical trend. Fractures are iron stained with some minor infilling, planar and rough, at 20 to 60° to the core axis (tca) and with random orientations.

3.28 to 3.48

Fractured zone, angular, rust stained rock fragments from 10 mm to 50 mm across. Possible fault zone, but no indication of relative movement (slickensides).



Core surface shows some grinding and diameter reduction.

**2-109**

Cored at BQ size from 0.71 to 3.73 m in five runs, but no intervals given on "End of Run" slips. Core recovery data taken from Peto MacCallum logs.

Core Run	Interval	Run length	Core recovery	RQD	Natural fract's
Run 1	0.71 – 1.5	0.79	100%	92%	1.66/m
Run 2	1.5 – 2.1	0.6	100%		
Run 3	2.1 – 2.7	0.6	88%		
Run 4	2.7 – 2.9	0.2	100%		
Run 5	2.9 – 3.73	0.83	95%		

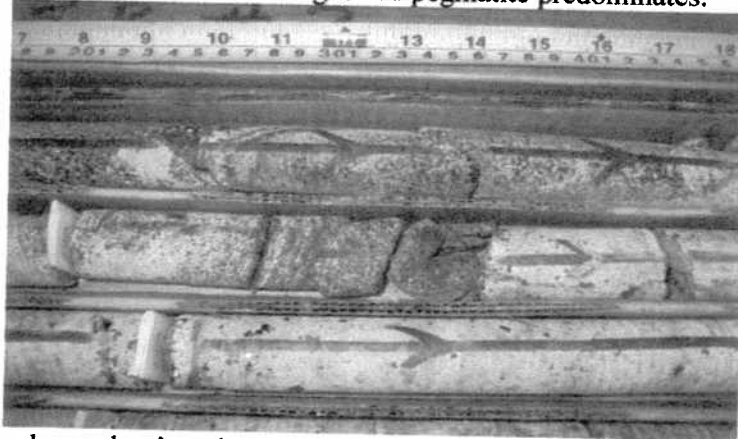
There are no obvious signs of any lost core – all core pieces fit together on mineral boundaries.

0.71 to 3.73 m (T.D.)

Fresh to slightly weathered, massive, speckled black and grey, medium grained, strong, biotite gneiss, with variable development of fresh, massive, pinkish grey, coarse-grained, very strong pegmatite; to 15 mm crystals throughout hole. Inclined gneissic banding at 35-48° tca, curved, irregular, very rough. Pegmatite is more brittle and broken, but stronger than the gneiss. Veins cross foliation.

1.02 to 1.63 m – Coarser grained with sporadic development of pegmatite, dark greenish grey. No significant structural defects.

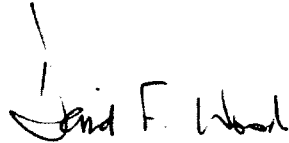
1.63 to 3.13 m – Coarse-grained pegmatite predominates.



Core breaks predominantly on mica rich horizons or in pegmatite veins. Excellent mineral boundary breaks throughout. Minor core grinding observed.

I trust that this letter provides the information you need at this time. If there is anything else I can do to assist, please do not hesitate to contact me.

Yours sincerely,  
David F. Wood Consulting Ltd.

A handwritten signature in black ink, appearing to read "David F. Wood". The signature is written in a cursive style with a large initial 'D'.

David F. Wood, P.Eng., President



## **APPENDIX D**

Seismic Refraction Soundings Report  
(Geophysics GRP International Inc.)



GEOPHYSICS GPR INTERNATIONAL INC.

6741 Columbus Road  
Unit 103  
Mississauga, Ontario  
Canada L5T 2G9

Tel.: (905) 696-0656  
Fax: (905) 696-0570  
gprtor@on.aibn.com  
www.geophysicsgpr.com

December 15, 2004

our file: T04670B

Carlos Nascimento, P.Eng  
**Peto MacCallum Limited**  
165 Cartwright Avenue,  
Toronto, Ontario,  
M6A 1V5

**RE: Stephenson Road #2 Seismic Refraction Soundings**

Geophysics GPR International Inc. was requested by Peto MacCallum Limited to perform seismic refraction soundings at Stephenson Road #2 east abutment and the center pier in the District of Muskoka, Ontario.

The surveys were performed December 3, 2004.

The primary aim of the investigation was to produce depth profiles to overburden layers and bedrock

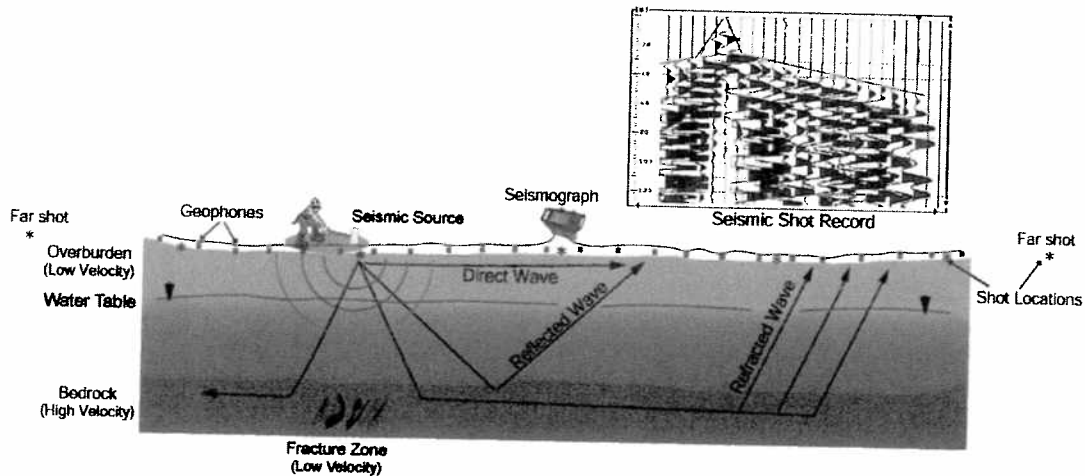
**Methodology:**

***Basic Theory***

The seismic refraction method relies on measuring the transit time of the wave that takes the shortest time to travel from the shot-point to each geophone. The fastest seismic waves are the compressional (P) or acoustic waves, where displaced particles oscillate in the direction of wave propagation. The energy that follows this first arrival, such as reflected waves and transverse (S) waves, is not considered under routine seismic refraction interpretation. Figure 1 illustrates the basic operating principle for refraction surveys.

***Survey Design***

A seismic spread typically consists of 24 vibration monitoring devices (geophones) connected in line (spread) to a seismograph (ABEM Terraloc Mark 6) by two 12 connector cables. Seismic pulses (shots) are then generated at various locations with respect to the spread. This seismic investigation used spacing between geophones of 3m. Typically seven shots were executed: five shots within the profile to obtain the lateral velocity variation in the overburden and two shots on either side of the spread to provide the true velocity of the bedrock surface.



**Figure 1: Seismic Method Operating Principle**

A "buffalo gun" (a muzzle loaded instrument used to detonate blank 12-gauge shells approximately 30 to 80cm below surface) was used as an energy source for this investigation. This energy source is typically sufficient for shallow to intermediate depths of investigation (0 to 20m).

#### ***Interpretation Method and Accuracy of Results***

Interpretation of the seismic data was done using Hawkins' method and double checked by the critical distance method. This method provides information on the thicknesses of the various overburden layers, depth to bedrock and rock quality. It is based on the closure times of the inner shots. It calculates the true velocities of the rock using the apparent velocities, with the information provided by the outer shots.

The seismic refraction method allows the determination of the bedrock profile with a precision of 10% or better for depths greater than 10 m and a precision of 1 m for depths less than 10 m. The precision in the determination of rock velocities is plus or minus 3%. The vertical contacts (lateral velocity change), usually associated with faults and deep valleys, are generally accurate to within 5 m in width; although, this is somewhat site specific.

**Results:**

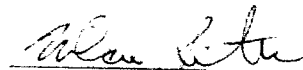
Two seismic profiles were collected and there is enclosed one drawing that identifies the centre of the abutment as 0.0. The profile is orientated from south to north.

It is possible to determine the depth to bedrock beneath each of the 24 geophones.

Boreholes were available along both profiles and were correlated against the initial interpretation. There did not appear to be any significant discrepancy. The measured velocities for the overburden layers were maintained and velocities for the overburden and bedrock are presented.

If you have any questions or comments, do not hesitate to contact me.

Sincerely,



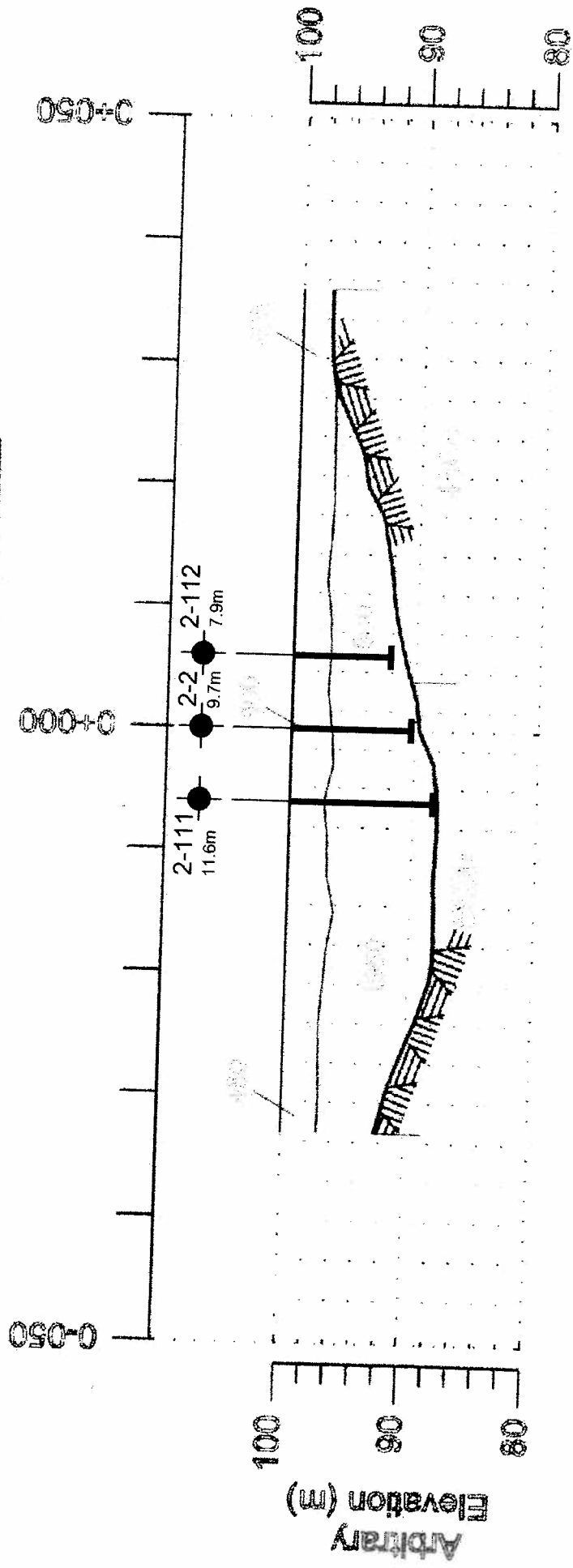
Milan Situm, P.Geo.  
Manager







# Stephenson Road #2 Median - Centre Pier

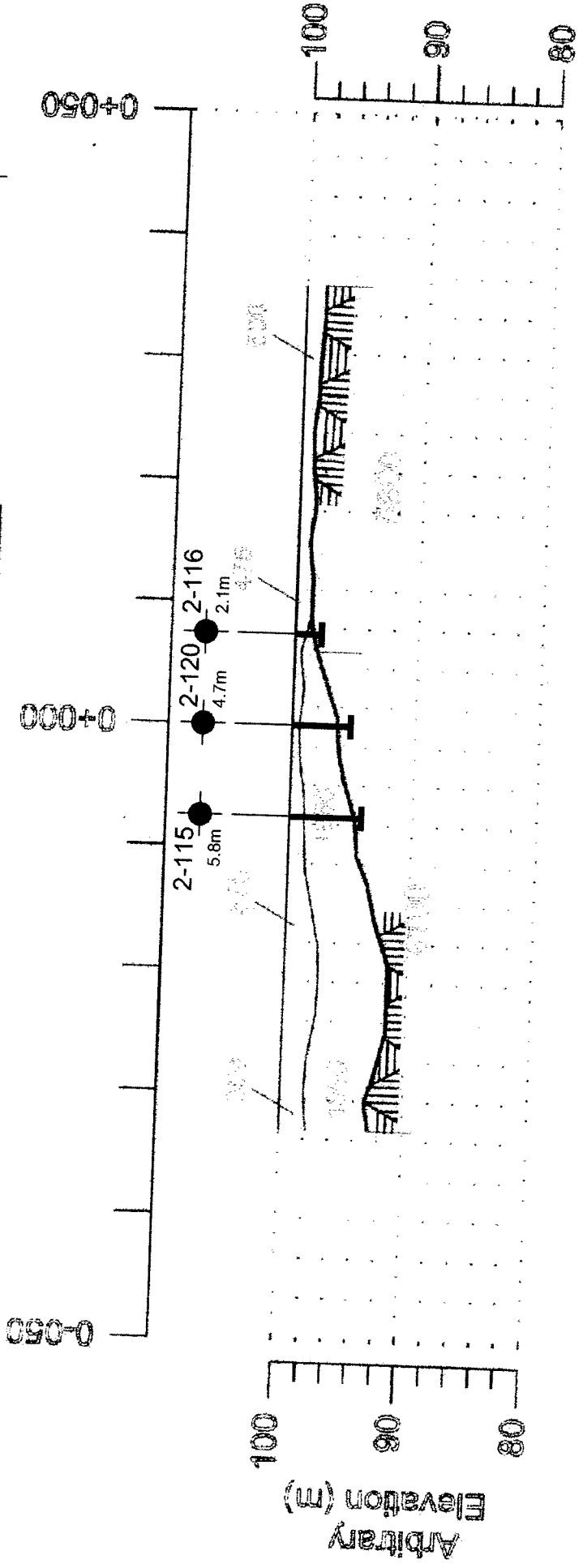


## NOTES:

1. BOREHOLE LOCATIONS ADDED.
2. DIMENSIONS REFER TO DEPTH TO BEDROCK IN THE BOREHOLE.



# Stephenson Road #2 East Abutment



## NOTES:

1. BOREHOLE LOCATIONS ADDED.
2. DIMENSIONS REFER TO DEPTH TO BEDROCK IN THE BOREHOLE.



**FOUNDATION DESIGN REPORT**

**for**

**STEPHENSON ROAD NO. 2 UNDERPASS  
WP 5040-00-01, SITE 42-327  
HIGHWAY 11  
TOWNSHIP OF STEPHENSON  
DISTRICT 52, HUNTSVILLE**

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Table 1 - List of Standard Specifications Referenced in Report

Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core

Figure 2 - Rockfill Drainage in Slope Flattened Areas

Appendix A - Rock Foundation Conditions (David F. Wood Consulting Ltd.)

**FOUNDATION DESIGN REPORT**  
for  
**Stephenson Road No. 2 Underpass**  
**WP 5040-00-01 , Site 42-327**  
**Highway 11**  
**Township of Stephenson**  
**District 52, Huntsville**

---

**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations, abutments and approach embankments for the proposed construction of an underpass at Stephenson Road No. 2 and Highway 11 located some 16 km south of Huntsville, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

The new Stephenson Road No.2 alignment passes over Highway 11 at Station 11+711.887, Highway 11 Chainage, in the Township of Stephenson (ref. Drawing 1 "Stephenson Road No. 2 Underpass Highway 11 – General Arrangement" prepared by MRC in July 2004). Data from the preliminary foundation investigation carried out by Golder Associates Limited (GAL) reference No. 011-1104 dated April 2001 was used during preparation of this report.

The new Stephenson Road No. 2 underpass comprises a two-span structure with two equal spans of 39 m for a total length of 78 m. The road grade at the west abutment of the Stephenson Road No. 2/Highway 11 underpass is planned at elevation 322.7 and elevation 323.3 at the east abutment. Based on the existing ground levels at the borehole locations, the embankment fill will be about 3.5 and 9.0 m high at the west and east abutments, respectively. No fill or cut is anticipated at the centre pier foundation that is planned in the existing Highway 11 median.

The depth of the soil cover at the location of the boreholes varies from 0.0 to 1.2 m (typically less than 0.6 m deep) at and 20 m west of the west abutment, 7.9 to 11.6 m at the centre pier, 1.2 to 6.7 m at the east abutment and 5.2 to 6.6 m within 20 m of the east approach embankment.



The subsurface stratigraphy at the west abutment and approach embankment boreholes consists a thin layer of peat and topsoil overlying cohesionless sandy soils typically less than 0.6 m thick, with bedrock outcrop exposures. An approximate 5 m high rock cut exists at this location along the southbound lanes of Highway 11. At the east approach embankment, east abutment and centre pier the soil cover generally comprises localized fill or peat and topsoil units covering typically loose to compact sand/ silty sand overlying compact to dense silt/sandy silt layers and discontinuous typically compact to very dense silty sand/gravelly sand mantling bedrock.

The groundwater table was at 3.7 m depth elevation 312.0 at the center pier on November 12, 2004. Perched groundwater fluctuated between 2.0 and 4.2 m depths at the centre pier and from dry conditions to 2.5 m depth at the east abutment on readings obtained in February/April 2001 and in October/November 2004, indicating seasonal variations.

The depth below grade and surface elevation of the bedrock identified in the boreholes drilled at this site are summarized in the following table:

LOCATION	BOREHOLE NO.	DEPTH TO ROCK (m)	BEDROCK ELEVATION
West Approach	2-107	0.7	321.1
West Abutment	2-101	1.2	318.1
	2-102	1.2	319.9
	2-103	0.5*	318.5
	2-104	1.0*	319.5
	2-105	0.0*	318.6
	2-106	0.0	320.3
	2-108	0.4	319.5
	2-109	0.7*	318.8
	2-110	0.4	318.8
	2-111	11.6	302.0
Centre Pier	2-112	7.9*	306.4
	2-2	9.7*	304.2



LOCATION	BOREHOLE NO.	DEPTH TO ROCK (m)	BEDROCK ELEVATION
East Abutment	2-113	6.7	306.7
	2-114	1.2*	312.7
	2-115	5.8	307.7
	2-116	2.1	311.9
	2-117	5.8*	307.9
	2-118	3.0	311.1
	2-119	3.7	309.9
	2-120	4.7*	309.1
	2-121	3.3	310.6
	2-123	1.4	312.5
	2-124	2.4	311.4
	2-125	2.2	311.9
	2-3	3.2*	310.9
East Approach	2-122	6.6	308.1
	2-5	5.2	<309.8

\* - Confirmed by rock core

The assessment of the soil/rock interface line by means of Seismic Refraction Sounding (SRS) surveyed along two lines parallel to Highway 11 at the east abutment and centre pier locations indicate continuous rock surface lines without discontinuities or sharp grade level differences in the rock surface at these locations.

## 2. FOUNDATIONS

### 2.1 General

#### 2.1.1 East Abutment and Centre Pier

The depth of the soil/bedrock interface is 7.9 to 11.6 m in the boreholes drilled for the centre pier and 1.2 to 6.7 m at the east abutment of the new bridge structure. The relative density of the soils to depths of 4 to 6 m (elev. 308 to 310) at the center pier is loose to compact. At the east abutment, the bedrock surface in the north quarter to central part is near elevation 311.1 to 312.7 sloping down to elevation 306.7 to 307.9 in the south half to three quarters of the abutment. The



overlying soil to near elevation 312 is typically loose to compact. Approximately 9 m of fill is anticipated at the east approach embankment.

It is considered that spread footings constructed on the native soils are not suitable to support the loads of these structure foundations. It is considered feasible to support the east abutment and centre pier on piles driven to bedrock or caissons installed on the bedrock. Spread footings founded on engineered fill placed on the compact soil strata are also feasible for the east abutment foundation, however not considered feasible for the centre pier due to existing fill and loose soil extending below the groundwater table. Drilled caissons founded and socketted into bedrock could be considered but difficult construction conditions are anticipated at the centre pier due to the high groundwater level.

#### 2.1.2 West Abutment

The soil/bedrock interface at the boreholes drilled for the west abutment of the new bridge structure is 0.0 to 1.2 m below existing grade. The GA drawing calls for the west abutment to be supported by a spread footing and the top of footing at elevation 318.2. Since the bedrock surface is between elevations 318.1 and 320.3, this abutment may be founded on spread footings bearing directly on the bedrock. The adequacy of the rock condition to support the abutment was validated from the geological perspective by close inspection of the exposed rock face and rock cores by David F. Wood Consulting Ltd. (DWL), a rock engineering consultant firm. DWL recommendations are enclosed in Appendix A.

#### 2.1.3 Comparison of Foundation Alternatives

Footings bearing on engineered fill or bedrock, piles driven to bedrock and caissons installed on the bedrock were considered feasible alternatives for foundations at this site. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives is presented on the following table.





Foundation Type	Advantages	Disadvantages	Applicable Foundation Element
Spread Footings on Rock	<ul style="list-style-type: none"> <li>• Ease of construction</li> <li>• Lower cost than deep foundations</li> <li>• High bearing resistance</li> </ul>	<ul style="list-style-type: none"> <li>• Need to remove rock to foundation elevation</li> <li>• Needs mass concrete to achieve a level bearing surface</li> </ul>	West Abutment (*)
Spread Footings on Structural Fill	<ul style="list-style-type: none"> <li>• Ease of construction</li> <li>• Lower cost than pile foundation</li> <li>• Level bearing surface</li> </ul>	<ul style="list-style-type: none"> <li>• Requires the construction of a structural fill pad</li> <li>• Difficult condition for preparation of structural fill pad foundation</li> <li>• Low bearing resistances</li> <li>• Needs erosion protection</li> </ul>	East Abutment
Caissons on Rock	<ul style="list-style-type: none"> <li>• Provides positive socket into bedrock</li> </ul>	<ul style="list-style-type: none"> <li>• Requires specialized construction equipment for drilling into bedrock</li> <li>• Difficult installation due to sloping soil/rock interface and high groundwater</li> <li>• Unwatering of caisson holes required</li> <li>• Requires construction of a fill pad ahead of the approach embankment construction</li> <li>• High installation cost</li> </ul>	East Abutment Centre Pier
Piles on Rock	<ul style="list-style-type: none"> <li>• Does not require specialized construction equipment to drill into bedrock</li> <li>• High bearing resistance</li> <li>• Technique and construction equipment to drive piles is available in the industry</li> </ul>	<ul style="list-style-type: none"> <li>• Requires rock points due to locally inclined rock surface</li> <li>• Requires construction of a fill pad ahead of the approach embankment construction</li> <li>• High installation cost</li> </ul>	East Abutment (*) Centre Pier (*)

Note: (\*) Preferred foundation type for structure foundation element.



Conventional or semi-integral abutments are considered feasible at this site based on the foregoing considerations. Integral abutments are considered not feasible due to the west abutment high rock elevation. The system employed to found the proposed structure will be dictated by structural design and economic considerations and construction constraints. From a foundations engineering perspective, use of piles driven to bedrock is the preferred means of founding the east abutment and centre pier; spread footings constructed on bedrock is the preferred foundation system for the west abutment.

#### 2.1.4 Common Recommendations

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 [soil profile Type I, Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00, clause 4.4.6]. Based on the grain size and relative density/consistency of the soil cover at the site, it is considered that liquefaction of the soil is unlikely to occur (refer to clause 4.6.2 of the CHBDC).

All footings and/or pile caps subject to frost action should be provided with 1.8 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover. Footings bearing directly on bedrock do not require protection from frost.

A list of the standard specifications referenced in this report is provided in Table 1.

## 2.2 Spread Footings

### 2.2.1 General

Supporting the west abutment of the underpass structure on conventional spread footings founded on the bedrock is recommended.

Construction of the footings should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface. In addition, a rock engineering specialist should be retained to examine the integrity and/or impact on bedrock below the footings should blasting be required near the structure foundations.



## 2.2.2 Footings Constructed on Bedrock

Footings bearing on the bedrock should be designed using a factored bearing resistance at ULS of 10,000 kPa. Considering the bedrock to be non-yielding, the design will not be governed by settlement criteria since the loading required to produce 25 mm deformation is much larger than the factored capacity at ULS. No reduction of the bearing resistance for inclined loads is required in reference to clause 6.7.4 of the CHBDC, in view of the rock conditions observed by DWL.

The anticipated depths/elevations to bedrock at the west abutment are indicated in the table provided in a previous section of this report. The bedrock surface ranges from elevation 318.1 to 320.3 at the west abutment, some 1.8 m above the proposed top of footing elevation 318.5 shown on the GA drawing.

Comments concerning excavation of bedrock to enable construction of the footings are provided in subsequent sections of the report and in the recommendations prepared by DWL (Appendix C).

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the bedrock. If the footings are cast directly on the surface of the bedrock (bedrock surface not roughened by excavation/construction activities), an unfactored friction factor of 0.6 should be employed since this bedrock surface is expected to be relatively smooth, as a result of weathering and/or glaciation. If excavation of the bedrock is required, an unfactored friction factor of 0.7 could be used.

The lateral resistance of footings founded on bedrock could be increased by means of a shear key, sockets and/or by installing dowels/anchors into the bedrock (SP 999S26). The increased lateral resistance will be provided by the shear strength of steel dowels if used, the horizontal resistance of the bedrock, the horizontal component of tensile forces developed in any inclined anchors and/or a greater frictional resistance between the footing and rock if the anchors are prestressed to increase the vertical pressure. The factored horizontal resistance at ULS of the bedrock is considered to be 5,000 kPa.

If dowels are used, a NSSP (Dowels Into Concrete) should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels.



Design, installation and testing of anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC. If anchors are installed, factored bond stresses at the grout/rod and grout/rock interfaces of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) and 800 kPa respectively are recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.

### 2.2.3 Footings Constructed on Structural Fill

A structural fill alternative is not appropriate at the west abutment since the bedrock surface is higher than the base of the abutment stem and consequently over excavation of rock would be required to accommodate the minimum 1.0 m granular fill pad thickness.

However, the east abutment foundation is considered feasible on structural fill placed on the compact soil layers underlying the east abutment location. Particular care is required to prepare the structural fill founding subgrade. All loose soil and boulders under the engineered fill pad should be removed. The compact sand was identified at variable levels from elevations 310.3 to 313.8 in the east abutment boreholes 2-113 to 2-125 drilled during the field investigations. Bedrock was encountered at levels as high as elevation 312.7. The structural fill pad should be founded at elevation 310.3 on compact sand and on the bedrock surface where the rock level is higher than the compact sand.

The structural fill should comprise OPSS Granular A material placed in maximum 200 mm thick lifts, compacted to 100% maximum dry density determined by the MTO test method LS-706 (standard Proctor) and extended laterally to a line inclined downwards at 45° to the horizontal originating at least 1 m from the top of the footing. This scheme is illustrated in Figure 1, appended. The limits of the required fill pad should be clearly marked and surveyed in the field accounting for the undulations on the bedrock surface.

Footings should not be constructed on rockfill. However, rockfill (if applicable) may be placed adjacent to the Granular 'A' core noted in Figure 1.



The recommended bearing resistance for minimum 2.5 m wide footings constructed on structural fill at least 2.5 m thick is as follows:

$$\begin{aligned}\text{Factored Bearing Resistance at ULS} &= 900 \text{ kPa} \\ \text{Bearing Resistance at SLS} &= 350 \text{ kPa}\end{aligned}$$

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.8 m and groundwater level below the founding depth was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.7 is recommended for footings on the structural fill.

## 2.3 Pile Foundations

It is recommended that the foundation loads of the centre pier and east abutment be supported by driven piles.

Piles for the center pier and east abutment should be driven to refusal on bedrock at the estimated range of reference founding levels that are in the table provided in Section 1.0.

The recommended factored axial resistance at ultimate limit states (ULS) for the four pile sections noted is considered to be appropriate:

<b><u>Factored Axial Resistance at ULS, kN</u></b>	
HP 310 x 110	2000
HP 310 x 152	2800
HP 360 x 108	2000
HP 360 x 152	2800



The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length required the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or bedrock are much larger than the ULS factored capacity.

The presence of cobbles/boulders was not identified immediately above the bedrock at the east abutment and center pier locations. These deposits may be present between the boreholes, based on the findings at other locations in the area (South Mary Lake Road interchange structure). The risk of damage during driving is considered to be low; nevertheless, a NSSP should be prepared to advise the contractor of the potential presence of boulders and sloping bedrock at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01.

The NSSP should provide specific direction for the contractor to provide experienced full time foundation engineering supervision to monitor the driving operations over the full length of the pile within 5.0 m of the top of the bedrock, elevation 308 (east abutment) and elevation 301 (centre pier). This should involve assessment of the performance of the hammer, recording of the number of blows required to advance the pile during each 25 mm of penetration, interpretation of the penetration data as the pile is driven for evidence of unusual conditions that could be indicative of damage, ensuring the piles have been driven to refusal on bedrock, directives not to overdrive the pile on sloping bedrock surfaces and the need to drive replacement piles if evidence of damage is detected.

The compacted granular fill pad used as a working platform for construction equipment during the installation of the east abutment piles should comprise OPSS Granular A material to allow installation of the piles without damage. Alternate granular fill materials could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through 4 to 6 m of compacted granular fill and the underlying native soils that typically comprise very loose to compact cohesionless silty/sandy strata. It is considered, based on our extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated



energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed. Since the piles will be driven to bedrock, a specific set is not provided.

The rock surface is relatively steep locally, in particular, the rock surface slopes at about 16° between boreholes at the centre pier and at angles of 4° to 23° at the east abutment, locally 46° at the northwest corner. The piles will be set on or into bedrock should be equipped with Oslo Points (OPSD 3304) or Titus H Bearing Pile Points, Rock Injector Model (SP 903S01; clause 3.1.2 and 3.3.1-6 of the Structural Manual (Division 1 - Exceptions to the Canadian Highway Bridge Design Code) dated June 2002) to minimize damage to the pile toe when driving through potential layers of cobbles/boulders overlying bedrock at the site.

### 2.3.1 Integral Abutments on Piles

Use of driven piles in conjunction with an integral abutment system was considered. However, an approximate 5 to 7 m deep cut into the bedrock adjacent to the existing 5 m high rock face would be needed to accommodate the minimum pile length required for integral abutments as indicated in the MTO Report Ref. No. SO-96-01, consequently there would be no lateral support for the piles. Therefore, construction of integral abutments would not be practical for this structure.

### 2.3.2 Lateral Resistance

The soil adjacent to the upper section of the piles is expected to comprise the compacted approach fill placed on typically cohesionless very loose/compact sands and silts.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The assessed lateral resistance for the pile sections noted previously is as follows:

SOIL TYPE	NATIVE SILT/SAND		GRANULAR BACKFILL	
Pile Section	HP310	HP360	HP310	HP360
Factored Lateral Resistance at ULS, kN	105	140	120	170
Lateral Resistance at SLS, kN	35	45	50	70



The assessed values of lateral resistance values assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended in the section titled "Approach Embankments". If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction,  $k_s$  ( $\text{MN/m}^3$ ) should be computed using the following equation (Canadian Foundation Engineering Manual, 3<sup>rd</sup> Edition):

$$k_s = n_h z/b$$

where  $n_h$  = coefficient related to soil density  
 =  $10.0 \text{ MN/m}^3$  for granular backfill  
 =  $2.0 \text{ MN/m}^3$  for native silty/sandy soils above groundwater table  
 =  $1.3 \text{ MN/m}^3$  for native silty/sandy soils below groundwater table

$z$  = depth, m

$b$  = pile width, m

For lateral resistance design purposes, the groundwater should be considered at 2.0 m depth to allow for seasonal fluctuations.

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

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





## **2.4 Caisson Foundations**

Use of caissons to support both the centre pier and east abutment could be considered. However, it must be understood that installation of the caissons will be difficult due to the requirement to control the influx of groundwater into the caisson holes at the center pier and the sloping bedrock surface elevation.

The rock surface is relatively steep locally, as indicated previously in this report. Due to the locally sloping bedrock surface at the centre pier and east abutment, it will be necessary to employ special equipment to install the caissons and limit the diameters of the caissons to those of the available equipment. In general, this would involve drilling a small diameter (about 50 mm) pilot hole to 'pin' the caisson location and a large diameter core barrel that, while maintained by the pin, will core into the high edge of the sloped rock surface. Due to equipment limitations, it will probably be necessary to limit the diameter of the socket to about 600 mm. The core barrel can be advanced into bedrock the requisite depth to satisfy structural design considerations (axial and lateral resistance) as noted in subsequent paragraphs.

The length of the caisson sockets into rock should be a minimum of 0.5 m in view of the fair to excellent rock quality. The need for a greater socket length to resist lateral loads should be assessed. Further comments in this regard are provided in subsequent paragraphs.

The caissons should be designed using a factored end bearing resistance at ULS of 8,000 kPa and a factored bond stress at ULS of 800 kPa. A reduction factor of 0.8 was applied due to the difficult construction conditions and the locally sloping bedrock surface.

The full value for shaft adhesion may be employed in caisson design based on shaft adhesion only (end-bearing ignored) or where the end bearing resistance exceeds the total shaft resistance. In cases where the total shaft resistance is greater than the end bearing resistance and the design is based on resistance being developed by both end-bearing and shaft adhesion, the mobilized resistance of the caisson should be limited to two times the end bearing resistance or the end bearing resistance plus 75% of the computed bond capacity, whichever is less.



Based on these values, the factored axial resistance at ULS for selected caisson diameters embedded into rock by 0.5 and 1.0 m is presented below.

CAISSON DIAMETER (m)	FACTORED AXIAL RESISTANCE AT ULS (kN)					
	0.5 m Long Socket			1.0 m Long Socket		
	End Bearing	Shaft Adhesion	Total	End Bearing	Shaft Adhesion*	Total
0.60	2,260	755	3,015	2,260	1,510	3,770
0.76	3,625	955	4,580	3,625	1,910	5,535
0.91	5,200	1,140	6,340	5,200	2,285	7,485

\* 75% of computed shaft adhesion

The resistance at SLS allows for 25 mm compression of the caisson and founding medium. Considering the bedrock to be non-yielding and the caisson length required, the design is not expected to be governed by settlement criteria since the loading required to induce the above deformation of the caisson and bedrock would be larger than the factored resistance at ULS.

The caissons should be installed and monitored in accordance with the requirements of SP 903S01.

Resistance to lateral loads will be provided by the 'horizontal bearing resistance' of the rock. The spacing between caissons in the direction transverse to the applied load should be at least three caisson diameters. The caisson spacing in the longitudinal direction should not be less than the socket length of the caisson to enable full mobilization of the lateral restraint. If the spacing is less than the socket length, the resistance should be reduced in direct linear proportion to the ratio of the two values.

The factored horizontal resistance at ULS of sound bedrock is considered to be 5,000 kPa. The resistance at SLS is greater and hence the factored resistance at ULS will govern design.

Since the bedrock cores retrieved from boreholes drilled at the abutments are typically fair to excellent quality, the bedrock is considered to be a 'frictional' material for the purpose of evaluating the point of contraflexure.



The coefficient of horizontal subgrade reaction,  $k_s$  ( $\text{MN}/\text{m}^3$ ) should be computed using the following equation (Canadian Foundation Engineering Manual, 3<sup>rd</sup> Edition) to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where:

$n_h$  = coefficient related to rock quality, =  $30 \text{ MN}/\text{m}^3$

$z$  = depth below bedrock surface, m

$b$  = caisson diameter, m

Due to the high groundwater table and cohesionless native materials at the center pier, it will be necessary to employ special methods to control water when installing the caissons. Sealing the bottom of the steel liner on the bedrock surface may be difficult in view of the possible undulations in the surface level of the bedrock. Placement of concrete by tremie will probably be necessary at the center pier caissons.

### 3. ABUTMENT WALLS

#### 3.1 General

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$  (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.

$$p = K(\gamma h + q) + C_p + C_s$$

where  $K$  = coefficient of lateral earth pressure (dimensionless)

$\gamma$  = unit weight of free-draining granular material,  $\text{kN}/\text{m}^3$

$h$  = depth below final grade, m

$q$  = surcharge load, kPa, if present.

$C_p$  = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

$C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where  $\emptyset$  = angle of internal friction of retained soil ( $35^\circ$  for Granular A or Granular B Type II)

$\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  for Granular A or Granular B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 2.1.



Free-draining granular material or rockfill should be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETERS	GRANULAR A	GRANULAR B TYPE II	ROCKFILL
Internal Friction Angle, $\phi$ (degrees)	35	35	42
Unit weight, $\gamma$ (kN/m <sup>3</sup> )	22.8	22.8	18.0
Coefficient of Active Earth Pressure, $K_a$	0.27	0.27	0.20
Coefficient of Earth Pressure At Rest, $K_o$	0.43	0.43	0.33
Coefficient of Passive Earth Pressure, $K_p$	3.69	3.69	5.04

### 3.2 Abutments

The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load ( $q$  in the preceding equation).

The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.9.1(a) of the CHBDC for this computation. The subsoil should be considered medium dense sand for the project.

A weeping tile system (SP 405F03) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standard specifications for granular or rock backfill at abutments (OPSD 3501 and 3505).



Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.

### 3.3 Retained Soil System Walls

A retained soil system (RSS) could also be employed at the abutments provided the predicted settlements noted in the section 4.3 are accommodated. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

The topsoil and peat encountered at both abutments are highly compressible and not considered adequate to support the RSS footings and should be removed from the RSS footprint

The RSS footings for the east abutment should be placed on a minimum 1.0 m thick structural fill pad in view of the loose upper zones of the subgrade. At the east abutment the recommended bearing resistance for a RSS wall footing (600 mm wide) constructed on structural fill placed at elevation 313.4 on the loose to compact native soil is as follows:

Factored Bearing Resistance at ULS	=	150 kPa
Bearing Resistance at SLS	=	100 kPa

The bearing resistance recommended previously for footings founded on bedrock at the west abutment is considered to be suitable for the RSS. The anticipated width of the RSS wall footing is 600 mm.

The earth pressure coefficients provided previously in this report are considered to be appropriate for the RSS wall.

The horizontal force at the base of the RSS will be resisted in part by the friction force developed through the granular backfill or along the interface between the granular backfill and the founding soil, subject to site specific design details. An unfactored friction factor of 0.7 and 0.55 is considered to be appropriate for the granular materials and native soil, respectively.



The RSS supplier should be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required and drainage requirements. The RSS wall designer should note that the MTO Northeastern Region requires that all fill to the structures comprises OPSS Granular B Type II for rockfill embankments. Transition treatments may be required between the RSS wall fill and the rockfill and/or OPSS Granular B Type II material. The RSS wall should be designed to withstand the estimated settlements of the native soils under the embankment loads indicated in the Approach Embankments section.

The supplier of the RSS should also be responsible for design of the structure (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.

#### **4. APPROACH EMBANKMENTS**

##### **4.1 General**

It is anticipated that the approach embankments will be constructed with earth borrow/granular materials or rockfill. The west and east approach fill embankments will be about 3 and 9 m high, respectively near the structures and taper down away from the structure. Construction of the fill on the bedrock and/or native sandy soils that underlie surficial deposits of peat or topsoil is considered feasible.

The peat and topsoil units identified in the abutment and approach embankment boreholes should be stripped prior to placement of the embankment fill.

The approach embankment constructed as recommended in this report will be stable, with minimum factors of safety against overall global stability greater than 1.5.



#### **4.2 Embankment Design and Construction Considerations**

The east and west approach embankment platforms should be widened by at least 1 m in accordance with the Northeastern Region Engineering Directive (NRE 98-200). The widening is required to allow for foundation and geotechnical considerations, such as settlement of the embankments and native soils and for future pavement overlays.

The embankments should be constructed in accordance with OPSD 201.010, 201.020, 202.010 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill. The 2 m wide mid-height berm called for in the earth fill embankments higher than 8 m and rockfill embankments higher than 10 m.

The earth fill slopes, if employed, should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened with earth fill to eliminate the need for a guide rail, a granular infilled drainage gap should be provided in accordance with Northeastern Region Pavement Design Practices and Guidelines as shown in Figure 2, appended. OPSS Granular B Type II should be used for the drainage gaps.

#### **4.3 Approach Embankment Settlements**

Settlement of the embankment platform is expected as a result of two mechanisms, consolidation of the native soil below the placed embankment fill and consolidation of the new fill.

The settlement under the west embankment fill due to consolidation of the underlying native relatively thin sandy soil is computed to be less than 10 mm and should occur within one month following placement of the fill.



, The typical computed settlement under the 9 m high east embankment fill due to the compression of the underlying native sandy/silty soil deposits is about 70 mm. It is estimated that the settlement will occur within one to two months of the placement of the fill.

If the embankments are constructed with rockfill, some settlement of the embankment fill surface, both during and following completion of construction, due to "consolidation" of the rockfill is likely to occur. The magnitude of total settlement is estimated to be 0.5% of the rockfill height (maximum of 15 and 45 mm at the west and east approaches, respectively). About 50% of these settlements should occur during the initial 12 month period following construction and 50% during the ten year period following completion of construction.

Some settlement of the road surface adjacent to the abutments due to "consolidation" of the granular backfill should also be expected. The granular backfill placed adjacent to the abutments will be about 3 and 9 m high at the west and east abutments, respectively. The magnitude of the "consolidation" of these fills depends on the workmanship employed by the contractor. If the fill is placed in 200 mm thick lifts compacted to 100% of standard Proctor maximum dry density near the optimum water content in accordance with the requirements of SP 206S03 and OPSS 501 (Method A), the estimated total settlements of the approach fill surface near the abutments should be less than 20 mm and be essentially complete within 2 to 4 months after placement of the fill.

The time schedule for completion of the approach embankment settlements may be accelerated with surcharge pre-loading of the fully constructed embankments.

It is estimated that preloading will not alter significantly the settlement completion time at the west abutment where a shallow soil cover was encountered. At the east abutment, placing a 2 m high surcharge of granular soils for a period of 1 month would accelerate the rate of consolidation so the predicted settlements noted previously are essentially complete at the end of the preload period. The long term consolidation of rockfill cannot be accelerated and should be incorporated in the design of the structure.





## 5. EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of the west abutment spread footing foundations founded on bedrock will extend through peat/topsoil, boulders and sandy soils to depths of up to 1.2 m. Excavation of the peat/topsoil and native soils is expected to be relatively straightforward at this location.

The sandy fill and native soils at the centre pier and east abutment sites are classified as Type 3 soils above the water table according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. The same materials below the water table are classified as Type 4 soils. The groundwater was found at about 2 to 4 m depths in November 2004 however the levels are subject to seasonal fluctuations and rainfall patterns. The temporary cut slopes for type 3 soils should be inclined at a 1H:1V slope over the full depth of the excavation. For type 4 soils, the side slopes should be cut at 3H:1V. Excavation of flatter side slopes may be required if excessively wet materials or concentrated seepage zones are encountered locally and/or higher groundwater levels are found during construction.

If space restrictions exist at the centre pier to establish the required 3H:1V excavation slopes, the groundwater level in the surficial sand deposits could be lowered locally using sump pumping or well pointing dependent on the geometry and depth of excavation below the water table. To minimize the extent and sophistication of groundwater control measures, consideration should be given to founding the pile caps at as high an elevation as possible with provision of Styrofoam-type insulation to provide adequate frost protection.

It is considered that the excavation of bedrock at the west abutment to establish the founding surface is possible with heavy equipment to jack-hammer the strong bedrock. Other conventional rock excavation techniques such as blasting (OPSS 120, General Specification for the Use of Explosives) may be required. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of cut and relative depth of excavation into the bedrock. Preshearing and presplitting to control the overbreak should be used to construct the west abutment rock face and founding surface, as required.



Mechanical means should be employed to remove loosened rock at the footing. Mass concrete could be employed to level minor variations in the bedrock surface as indicated previously. The footing founding area should be inspected and approved by a rock engineering specialist.

If blasting is required, a NSSP should be prepared to provide specific direction to the contractor to control the blasting/rock excavation activities to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded, require that a blasting specialist be retained to establish the charge to minimize overbreak, advise the contractor that any overblasting/overexcavation will be the sole responsibility of the contractor and require that loosened rock resulting from blasting operations be removed by mechanical means.

Near vertical sidewalls may be utilized for excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workmen.

Groundwater was not observed in the west abutment boreholes during or upon completion of drilling. It is anticipated that the groundwater is within 2 to 4 m depth at the center pier and east abutment and generally it is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations. It should also be noted that local seepage should be anticipated at the soil/bedrock interface and within depressions in the bedrock surface. Groundwater levels are subject to seasonal fluctuations and rainfall patterns.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.



## 6. CLOSURE

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer and reviewed by Mr. D. W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.



C. M. P. Nascimento, P.Eng.,  
Senior Foundation Engineer

Dennis W. Kerr, MEng, P.Eng  
Chief Foundation Engineer



Brian R. Gray, MEng, P.Eng.  
MTO Designated Contact



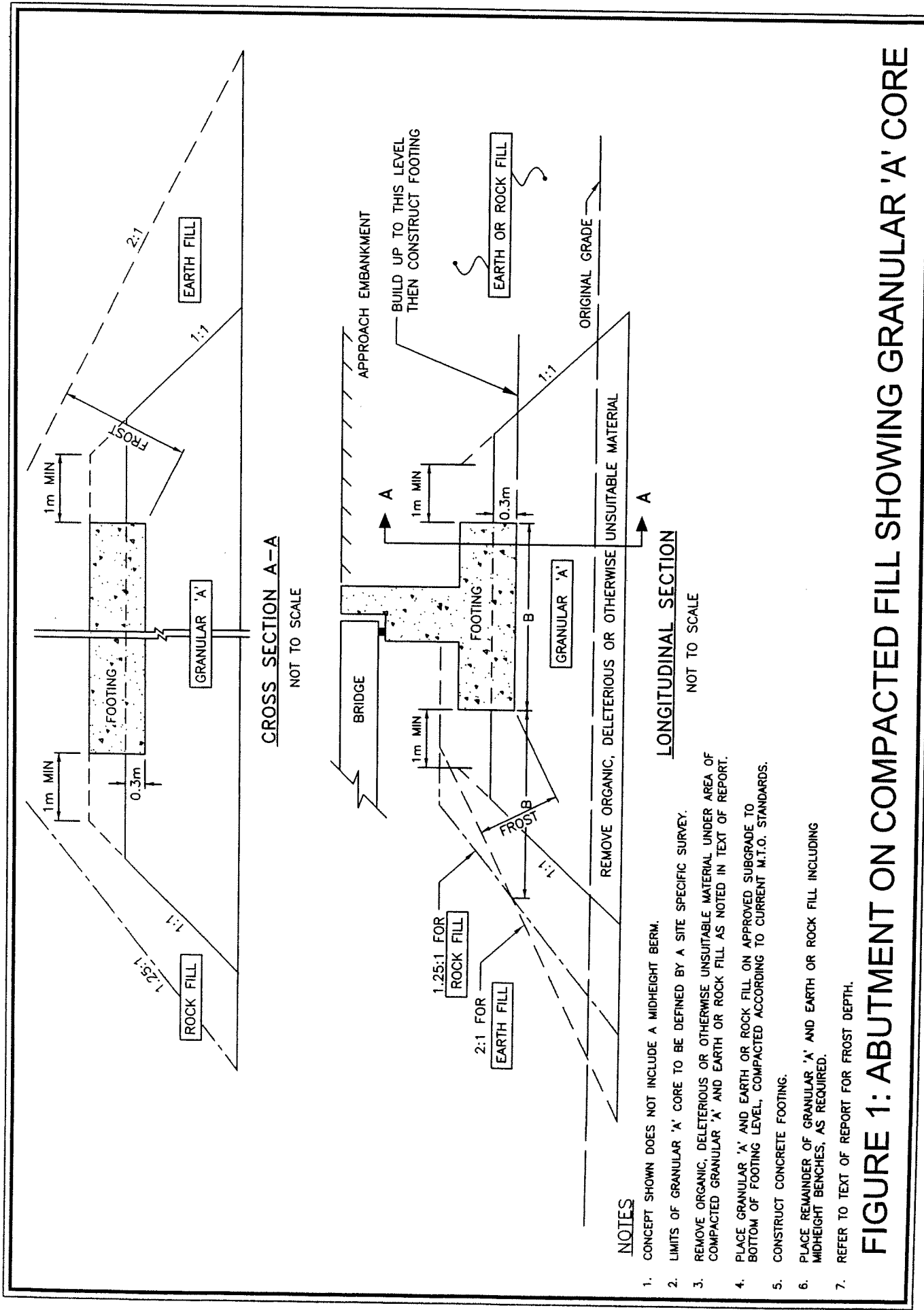
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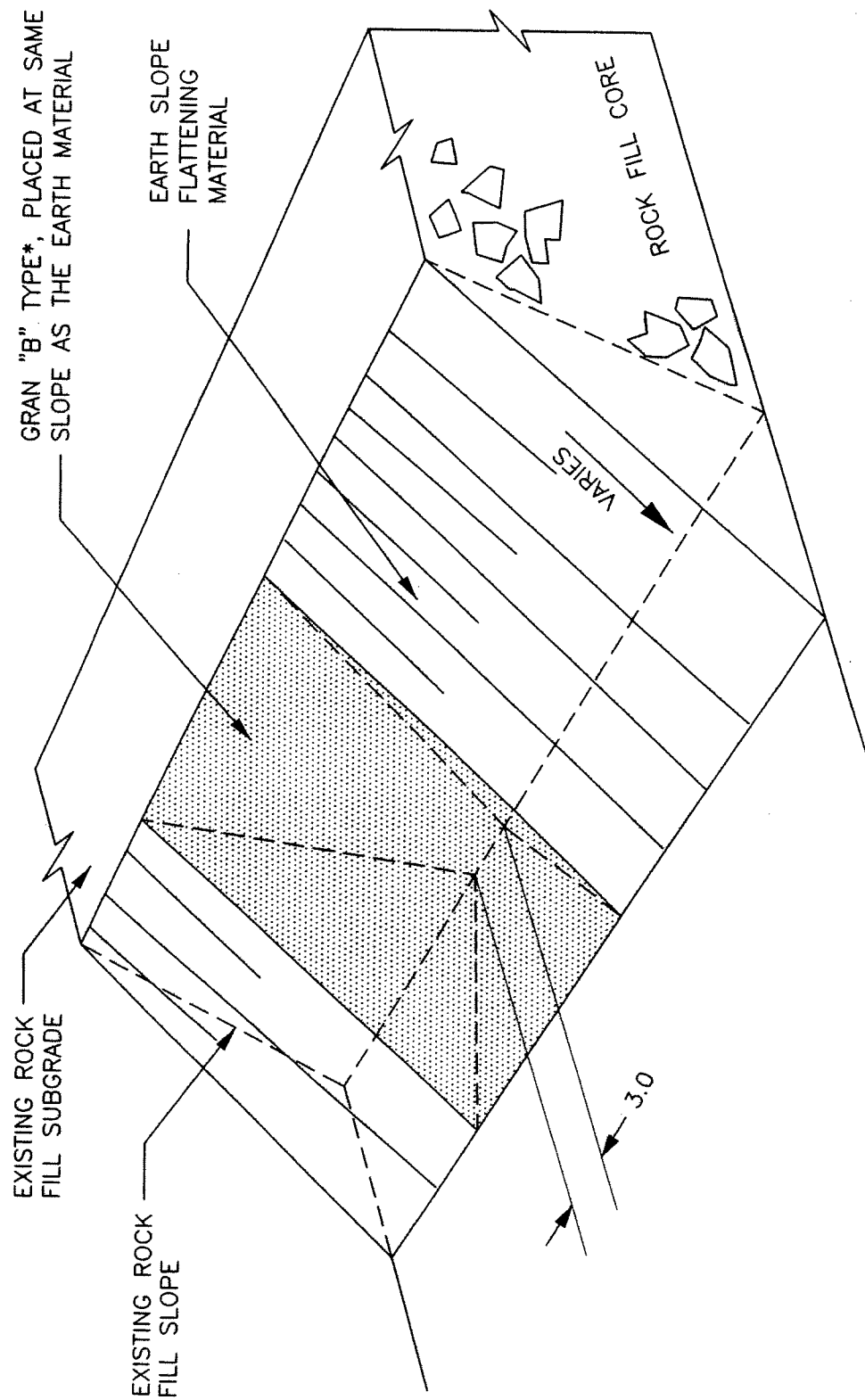


**TABLE 1**

**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

<b>TITLE</b>	<b>NO.</b>	<b>DATE</b>
General Specification for the Use of Explosives	OPSS 120	November 2003
Construction Specification for Compacting	OPSS 501	February 1996
Construction Specification for Sodding	OPSS 571	November 2001
Construction Specification for Seed and Cover	OPSS 572	November 2003
Rock Grading-Undivided Highway	OPSD-201.010	April 1999
Rock Grading-Divided Highway	OPSD-201.020	April 1999
Embankment Construction Using Excess Material Outside of Earthfill or Rockfill	OPSD-202.010	March 1, 1998
Drainage Gap for Slope Flattening on Rock or Granular Embankment	OPSD-202.020	March 1, 1998
Oslo Points for HP310 H-Piles	OPSD-3304.000	November 2001
Minimum Granular Backfill Requirements - Abutments	OPSD-3501.000	April 1999
Rock Backfill Requirements - Abutments	OPSD-3505.000	November 2001
Construction Specification for Compaction	SP 105S10	November 2004
Construction Specification for Grading	SP 206S03	January 2004
Construction Specification for Pipe Subdrains	SP 405F03	May 2004
Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)	SP 599S22	March 2001
Excavation and Backfilling of Structures	SP 902S01	September 2003
Construction Specification for Piling	SP 903S01	September 2004
Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock	SP 999S26	July 2004
Dowels Into Concrete	NSSP	December 2002
Northeastern Region Directive - Platform Widening	NRE 98-200	October 28, 1998





\* GRAN 'B' TYPE I OR TYPE II AS  
RECOMMENDED FOR PROJECT.

FIGURE 2: ROCK FILL DRAINAGE IN SLOPE FLATTENED AREAS

NOT TO SCALE



## **APPENDIX A**

Rock Foundation conditions  
(David F. Wood Consulting Ltd.)

# David F. Wood

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January 24, 2005

Peto MacCallum Limited  
165 Cartwright Avenue  
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M6A 1V5

Attention: Mr. Carlos M.P. Nascimento, P.Eng., Senior Consultant

**Re: Highway 11, Stephenson Road #2 Underpass,  
WP 5040-00-01, Site 42-327  
Foundation Design Report**

Dear Carlos;

## Introduction

This report describes the interpretation of rock mass conditions for the west abutment foundation design of the proposed bridge carrying Stephenson Road #2 over Highway 11 south of Huntsville, Ontario. The project falls under MTO WP 5040-00-01 and is referred to as Site 42-327.

The first part of the project involved reviewing previous studies to examine this site. Peto MacCallum provided copies of work by Golder Associates Ltd. who investigated the site and reported under Job Number 011-1104 in April 2001. Following this review, a site visit was made on 2<sup>nd</sup> November 2004 to gain first hand experience of the rock mass conditions at the site. A drilling program was then conducted and the cores from four (4) holes were inspected on 18<sup>th</sup> January 2005. The findings of the site inspection visit and core logging exercise are contained in the companion Foundation Investigation Report.

## Rock Mass Conditions

The rock mass at the site can generally be described as: fresh to faintly weathered, massive, dark grey with white and pink speckles and bands, coarse grained, strong to very strong, biotite granitic gneiss with amphibolite to the south. There is very limited natural jointing; the gneissic banding or foliation dips steeply to the northeast, but is not always represented as a discontinuity to the north of the proposed centreline. The foliation is also quite contorted at the site with folding and undulating observed. To the south of the site, the rock mass becomes weaker, more micaceous and blockier with the development of amphibolite. However, this part of the rock mass is not thought to be within the proposed abutment footprint.

---

*David F. Wood Consulting Ltd.*



The original rock mass conditions, prior to any work at the site, were not observed, but it may be assumed that they included an irregular surface configuration of rounded, glacially eroded knobs of rock. This would be due to the differential erosion of the various mineral components of the underlying metamorphic rock material. The excavated rock face along the west side of Highway 11 at this site shows some drill hole half barrels to the north of the proposed abutment that are clean and do not exhibit signs of over-blasting. Folding within the gneissic banding occurs to the south of the proposed centreline with some blast damage.

Limited rock removal by machine scaling (either using a hoe ram or a backhoe excavator) will be needed to ensure that the rock face is modified to a maintenance-free condition. There is a “nose” of rock close to the south end of the proposed structure with a large feldspar porphyry dike that cannot be considered to have significant load bearing capacity without scaling. Further commentary regarding foundation conditions is provided below.

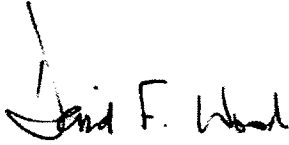
## Foundation conditions

The proposed bridge would be founded on a good quality rock mass. There is no justification for bulk blasting based purely on rock mass conditions. If blasting is required, then carefully executed wall control blasting per OPSS 206 is likely to have good results. However, no bulk rock mass excavation is required since geological processes have not adversely affected the rock mass just below the soil cover. The rock mass is only faintly weathered, with minor discolouration on discontinuity surfaces from the passage of water with iron-bearing minerals. Foundation preparation will require cleaning off all surficial materials (vegetation, topsoil and underlying loose material) to the bedrock surface, removing any loose surface rock fragments then, providing that the bedrock surface does not dip towards the highway at an angle steeper than about 10-15 degrees, installing rebar dowels in vertically drilled holes to provide shear resistance and vertical load transfer from the bridge abutment into the rock mass.

Rock bolting from the face does not appear to be warranted, but deep dowel foundations may be required to transfer load from the bridge abutment into the rock mass away from the excavated rock face. For design purposes, the length, diameter and spacing of dowels should be determined in consultation with the bridge design team. Dowels up to 3 metres in length may be required depending on the geometric relationship between the proposed structure and the existing landforms, however the dowel length will probably be reduced. A final decision regarding the precise location and depth of the dowels and the stability of the rock mass may have to be deferred until construction has started and the overlying materials have been completely stripped off.

I trust that this letter provides the information you need at this time. If there is anything else I can do to assist, please do not hesitate to contact me.

Yours sincerely,  
David F. Wood Consulting Ltd.

A handwritten signature in black ink, appearing to read "David F. Wood". The signature is written in a cursive style with a large initial 'D'.

David F. Wood, P.Eng., President