Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT

for

SOUTH MARY LAKE ROAD UNDERPASS
WP 62-86-02 , SITE 42-193
HIGHWAY 11
TOWNSHIP OF STEPHENSON
DISTRICT 52, HUNTSVILLE

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
3 cc: McCormick Rankin Corporation for distribution to MTO, North Bay + 1 digital copy
1 cc: Foundation Investigation Report to McCormick Rankin Corporation for distribution to MTO, North Bay + 1 digital copy
1 cc: McCormick Rankin Corporation for distribution to MTO, Downsview + 1 digital copy
2 cc: McCormick Rankin Corporation + 1 digital copy
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref.: 04TF006
Index No.: 072FIR and 073FDR
Geocres No.: 31E-228
May 17, 2005

FOUNDATION INVESTIGATION REPORT

for

SOUTH MARY LAKE ROAD UNDERPASS
WP 62-86-02 , SITE 42-193
HIGHWAY 11
TOWNSHIP OF STEPHENSON
DISTRICT 52, HUNTSVILLE

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
3 cc: McCormick Rankin Corporation for distribution to MTO, North Bay + 1 digital copy
1 cc: Foundation Investigation Report to McCormick Rankin Corporation for distribution to MTO, North Bay + 1 digital copy
1 cc: McCormick Rankin Corporation for distribution to MTO, Downsview + 1 digital copy
2 cc: McCormick Rankin Corporation + 1 digital copy
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref.: 04TF006
Index No.: 072FIR
Geocres No.: 31E-228
May 17, 2005

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
for
South Mary Lake Road Underpass
WP 62-86-02 , Site 42-193
Highway 11
Township of Stephenson
District 52, Huntsville

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed construction of an underpass at South Mary Lake Road and Highway 11 some 14 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 South Mary Lake Road alignment passes over Highway 11 at Station 14+079.36, Highway 11 chainage, in the Township of Stephenson (ref. Drawing 1 "South Mary Lake Road 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 is provided in this report.

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 200 m south of the existing South Mary Lake Road at-grade crossing of Highway 11 about 2 km south of the Highway 141 intersection. The structure to be erected will carry South Mary Lake Road traffic over Highway 11. The westerly structure alignment also crosses over the Lone Pine Drive, which is a local road with a low embankment constructed over a typically wet area. The vegetation cover in generally dense with mature trees and brush. A few commercial and industrial enterprises exist near the existing intersection.

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 strip of land that extends from Gravenhurst to North Bay ("The Physiography of Southern Ontario", Chapman and Putnam, 1984). Highway 11 roughly follows the alignment of this physiographic unit. 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, with the bedrock levels exhibiting locally sharp changes along the alignment.

3. INVESTIGATION PROCEDURES

The field work for this investigation was carried out during the period of October 18 to 22, 2004 and comprised 16 boreholes designated by the 3-100 series of numbers. A survey of the rock surface profile under at the west abutment using seismic refraction soundings (SRS) was carried out on November 4, 2004. The boreholes were drilled to depths of 0.0 to 17.3 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 (1)	TOTAL
West Approach	3-103	6.7	–	6.7
West Abutment	3-101	14.2	3.1	17.3
	3-102	11.7	3.2	14.9
	3-104	12.5	–	12.5
Centre Pier	3-105	10.9	3.3	14.2
	3-106	9.5	3.1	12.6
East Abutment	3-107	0.0	–	0.0
	3-108	0.9	–	0.9
	3-109	1.2	3.1	4.3
	3-110	1.7	3.5	5.2
	3-111	0.6	--	0.6
	3-112	0.8	–	0.8
	3-113	0.0	–	0.0
	3-114	2.5 (2)	4.2	6.7
	3-115	1.1	--	1.1
East Approach	3-116	0.0	–	0.0

(1) NQ diamond rock coring equipment
(2) Washboring required below boulder.

We also refer to Appendix A for the logs of previous boreholes drilled by GAL and the results of their laboratory testing, including plasticity charts and grain size distribution charts. The boreholes drilled by GAL are identified as boreholes 3-1 to 3-5. Borehole 3-3A was advanced at the same location as borehole 3-3 and therefore is not shown on the drawings.

Tulloch Engineering Ltd. (TEL) staked the alignment of South Mary Lake Road at the structure location. Peto MacCallum Ltd. (PML) selected the position 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 the existing ground surface at the working points (WP) for each of the foundation units:

TBM	DESCRIPTION	ELEVATION (*)
TBM1	Existing ground at west abutment WP	310.9
TBM2	Existing ground at centreline pier WP	311.5
TBM3	Existing ground at east abutment WP	314.4

(*) Geodetic, metric

The boreholes were advanced manually or using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. Two boreholes at each of the west abutment and centre pier and three boreholes at the east abutment were extended 3.1 to 4.2 m into the bedrock using NQ diamond rock coring equipment supplemented by NW casing and wash boring techniques. All boreholes were backfilled in accordance with the MTO guidelines for borehole abandonment procedures and using a bentonite/cement mixture grout.

Representative samples of the soils were recovered in the boreholes at frequent depth intervals. In the boreholes advanced with drill rigs, the samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Penetrometer tests were carried out on the cohesive samples; field vane tests were not conducted in view of the layered nature of the cohesive deposits and the amount of sand and silt contained in the soils. The results of the compressive strength measured by the penetrometer tests are reported on the attached Record of

Boreholes sheets. Photographs of the rock cores recovered in boreholes 3-101, 3-105 and 3-114 are enclosed in Appendix B.

The rock surface profile under the west abutment was checked with SRS using the services provided by Geophysics GPR International Inc. (GI). The report is included in Appendix C. A description of the SRS survey method is provided in the GI report. The depth to rock determined with SRS was calibrated to the known depth to rock in one of the cored boreholes drilled at the abutment.

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 (44)
- Sieve and hydrometer analyses (12)
- Atterberg limits tests (8)

The results of the laboratory natural water content determinations, grain size determinations and Atterberg limits are shown on the Record of Borehole sheets. Grain size distribution charts are presented on Figures 3-1 to 3-4. The Atterberg limits are listed on Table 2 and plotted on Plasticity Charts, Figures PC3-1 and PC3-2.

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.

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

The thickness of the soil cover revealed in the boreholes varies from deep deposits in excess of 9.5 m at the west abutment and centre pier to shallow soil cover less than 2.5 m thick at the east abutment. The soil cover at the west and centre foundation units generally comprises localized fill or peat deposits covering sandy silt/ silty sand interbedded with a clayey silt layer and mantling bedrock. The soil cover at the east abutment comprises topsoil and sand over bedrock. At the east abutment, bedrock is exposed at two borehole locations and was contacted or inferred at shallow depths in the remaining boreholes.

4.1 Fill

A surficial layer of fill occurs in borehole 3-105, drilled on the Highway 11 centreline median. The fill comprises loose brown sand trace gravel and was 600 mm thick, extending to elevation 311.1. Fill comprising 800 mm thick sand and gravel trace silt is also present in borehole 3-4 drilled west of Highway 11 off the shoulder of Lone Pine Drive. This fill unit extends to elevation 309.4.

4.2 Peat and Topsoil

Surficial deposits of peat and topsoil are present at the west abutment boreholes 3-101 and 3-102, approach borehole 3-103, centre pier boreholes 3-106 and 3-2 and east abutment boreholes 3-108 to 3-112 and 3-115. The peat and topsoil layers are 100 to 800 mm thick and extend to elevations 309.6 (west approach) to 314.5 (east abutment).

4.3 Sand/ Silty Sand/ Sandy Silt/ Sand with Silt

A deposit of cohesionless sand with varying silt content occurs at the surface in boreholes 3-104, 3-1 and 3-5 and below the fill, peat or topsoil units in the remaining boreholes, except boreholes 3-107, 3-113 and 3-116 where bedrock is present at the surface. The sandy soils extend to depths varying from 0.6 to 5.5 m, elevations 304.9 to 313.8, with the deeper deposits at the west abutment and centre pier. The relative density of the cohesionless soils typically is in the loose to compact (N = 5 to 29) range with very loose (N = 1 to 4) and dense zones (N = 31).

The particle size distribution of typical samples of these soils is shown of Figure 3-1. The water content ranges widely from 7 to 33% and is typically in the 18 to 22%, indicating typically wet conditions.

4.4 Cobbles and Boulders

A 900 mm thick boulder is present at the surface in east abutment borehole 3-114 and boulders are at the ground surface at borehole 3-107. Cobbles and boulders are also encountered within the silt and silty sand stratum above the bedrock in boreholes 3-104 (west abutment) and 3-105 (centre pier). Possible boulders could be present at shallow depth at other locations.

4.5 Clayey Silt

Discontinuous layers of cohesive clayey silt underlie the cohesionless sandy deposits at the west approach and abutment (boreholes 3-101 to 3-104, 3-1 and 3-4) and at the centre pier (borehole 3-2). (The clayey silt was identified as silty clay in the previous boreholes 3-1 and 3-4 however is described as clayey silt in this report for consistency with MTO classification system). The soil has a typically stiff consistency with very stiff zones (N = 8 to 20). Penetrometer tests range from 75 kPa to over 200 kPa.

The clayey silt extends to depths ranging from 4.6 to 9.7 m (elevation 300.7 to 306.8) where the layer was fully penetrated, and to the 6.7 and 8.2 m termination depths of boreholes 3-103 and 3-4. The thickness of the clayey silt layer ranges from 1.9 to 4.2 m.

The particle size distribution charts of the clayey silt samples obtained in the current boreholes are shown on Figure 3-2 and the plasticity chart is Figure PC3-1. The plasticity of the soil is low as indicated by Atterberg liquid limits ranging from 22 to 28, plastic limits from 16 to 21 and plasticity indexes from 6 to 10. The grain size distribution and plasticity charts of samples from the GAL boreholes are shown in Appendix A. The water content of the soils ranged from 24 to 32%.

4.6 Silt/ Sandy Silt

Deposits of cohesionless silt trace sand trace to some clay and sandy silt are present below the clayey silt /silty clay unit. In the west approach borehole 3-103, a 300 mm thick sandy silt layer occurs between 4.9 and 5.2 m depths above the clayey silt layer. The silty materials extend to the 12.5 m termination depth of borehole 3-104 at the west abutment and, in boreholes 3-101, 3-102 and 3-1 the materials extend to 11.2 to 13.1 m depths, elevations 297.6 to 299.3. At the centre pier, silt some clay trace sand and sandy silt units underlie the sand layer in boreholes 3-105 and 3-106 and extend to 8.2 and 9.3 m depths, elevations 302.4 and 303.2. The silt soils are typically compact (N = 12 to 29) with localized dense zones (N = 33).

The particle size distribution charts of the silt are presented on Figure 3-3 and the plasticity chart is Figure PC3-2. The silty soils are non-plastic or have a very low plasticity, indicated by liquid limits of 26 and 23, plastic limits of 19 and 21 and plasticity indexes of 4 and 5. The water content determinations in the deposits ranged from 12 to 28%, typically in the 22 to 28% range indicating wet soil conditions.

4.7 Silty Sand/ Gravelly Sand

Lower level deposits of cohesionless silty sand and gravelly sand occur below the silt/sandy silt soils at the west abutment boreholes 3-101, 3-102 and 3-1 extending to 11.7 to 14.2 m depths, elevations 296.2 to 298.8. The materials are present below the silt and clayey silt units in the centre pier boreholes 3-105, 3-106 and 3-2 extending to depths ranging from 9.5 to 10.9 m elevation 300.8 to 301.9. The soils exhibit a compact to very dense relative density (N = 19 to 63).

The particle size distribution chart of a gravelly sand sample is shown on Figure 3-4. The soil unit also contains boulders as described previously in this report. The water content varies from 10 to 20%, indicating wet conditions.

4.8 Bedrock

The bedrock comprises a dark grey biotite gneiss and light grey/grey to pink granitic gneiss. The rock is typically unweathered and exhibits medium to high strength in boreholes 3-101, 3-102 and 3-105 and high strength in the remaining boreholes. A detailed description of the rock cores retrieved from boreholes 3-101, 3-102, 3-105, 3-106, 3-109, 3-110 and 3-114 is provided in Table A and summarized on the record of borehole logs. The rock in boreholes 3-1, 3-2 and 3-3/3-3A is described as biotite granite gneiss, which is considered consistent with the descriptions on the 3-100 series boreholes.

At the west abutment, the bedrock surface was confirmed by rock coring or inferred by refusal at depths of 11.7 to 14.2 m depths, elevations 296.5 to 298.8, indicating a surface level difference of 2.3 m, between borehole locations. Photographs of the rock core taken in east abutment borehole 3-101 are shown on Plates 1 and 2, Appendix B.

The outline of the inferred surface of the bedrock surveyed with SRS is shown in Appendix C. The survey shows a continuous rock line along the proposed abutment and no sharp level variations or discontinuities.

In the centre pier boreholes, the soil/bedrock interface is confirmed by a minimum of 3 m of rock coring at depths varying from 9.5 to 10.9 m elevations 300.8 to 301.9, for an elevation difference of 1.1 m. Photographs of the rock core taken in the centre pier borehole 3-105 are shown on Plates 3 and 4, Appendix B.

Bedrock is exposed at the east abutment in boreholes 3-107, 3-113, and east approach borehole 3-116 and was confirmed by rock coring or inferred by refusal at typically shallow depth from 0.5 to 2.5 m in the remaining east abutment boreholes 3-108 to 3-112, 3-314 and 3-115, 3-3 and 3-5. The exposed rock and the inferred soil/rock interface are found at levels ranging from elevations 311.9 to 313.8. The elevation difference is about 1.9 m between boreholes. Photographs of the rock core taken in east abutment borehole 3-114 are shown on Plates 5 and 6, Appendix B.

In the 3-100 series boreholes, the measured core recovery varies typically between 91 and 100%, with two isolated values of 67 and 83% in boreholes 3-106 and 3-102, respectively. The RQD determined from the rock cores is in the range of 31 to 96% at the west abutment boreholes 3-101 and 3-102 indicating poor to excellent quality rock. The RQD for the centre pier boreholes 3-105 and 3-106 ranges between 55 and 100%, indicating a fair to excellent quality rock. At the east abutment boreholes 3-109, 3-110 and 3-114, the RQD varies from 80 to 100%, indicating good to excellent quality.

4.9 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. The summary of the piezometer readings is shown on the attached Table 1. The water strikes during the drilling varied from 0.6 to 11.7 m in the 3-100 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 varies from 0.8 m to 1.3 and 1.5 m in the boreholes 3-1 and 3-2 drilled at the west abutment and centre pier, respectively. The variations are considered likely caused by seasonal fluctuations and precipitation patterns.

5. CLOSURE

The field work was carried out under the supervision of Mr. F. Portela, Senior Technician, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Foundation Engineer. Marathon Drilling Co. Ltd. supplied the 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,

Peto MacCallum Ltd.

CNascimento

REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

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

Brian R. Gray

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
PROVINCE OF ONTARIO

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

CN/DWK:mi-lr

TABLE A

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
3-101	12	14.2 – 15.4	100	31	14.2 – 17.3	BIOTITE GNEISS: Dark grey, fine to medium crystalline, with slight banding, medium to high strength, unweathered, very close to close spaced flat partings, rough planar, tight, poor to fair becoming excellent quality.
	13	15.4 – 17.0	100	57		
	14	17.0 – 17.3	100	96		
3-102	9	11.7 – 12.8	98	56	11.7 – 14.9	BIOTITE GNEISS: Dark grey, fine to medium crystalline, slight banding, medium to high strength, unweathered, vertical parting from 11.9 to 12.2 m, open 1 mm, infilled with silt, very close to close spaced flat to dipping partings, rough planar, tight to oxidized, fair to good quality.
	10	12.8 – 14.3	91	64		
	11	14.3 – 14.9	83	83		
3-105	8	10.9 – 12.3	91	55	10.9 – 14.2	GRANITIC GNEISS: Grey to pink, medium crystalline, slight banding, occasional concentrations of biotite, medium to high strength, unweathered, very close to wide spaced flat to dipping partings, rough planar, tight, fair to excellent quality.
	9	12.3 – 13.5	100	100		
	10	13.5 – 14.2	97	73		
3-106	8	9.5 – 11.1	100	64	9.5 – 12.6	GRANITIC GNEISS: Light grey to pink, medium crystalline, slight banding, garnetiferous, high strength, unweathered, very close to moderate spaced flat partings, rough planar, tight, fair to excellent quality.
	9	11.1 – 12.4	100	100		
	10	12.4 – 12.6	67	67		

RQD:	Rock Quality Designation
Originated:	FP
Compiled:	JFW
Checked:	CN

TABLE A

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
3-109	1	1.2 – 2.7	100	96	1.2 – 4.3	GRANITIC GNEISS: Light grey to pink, medium crystalline, slight banding, high strength, unweathered, close to moderate becoming wide spaced flat partings, rough planar, tight, excellent quality.
	2	2.7 – 4.3	100	100		
3-110	1	1.7 – 3.2	92	92	1.7 – 5.2	GRANITIC GNEISS: Light grey to pink, medium crystalline, slight banding, high strength, unweathered, close to wide spaced flat to dipping partings, rough planar, tight, good to excellent quality.
	2	3.2 – 5.2	99	80		
3-114	1	2.5 – 4.1	100	100	2.5 – 6.7	GRANITIC GNEISS: Light grey to pink, medium crystalline, slight to moderate banding, high strength, unweathered, moderate to wide spaced flat to dipping partings, rough planar, excellent quality.
	2	4.1 – 4.8	97	97		
	3	4.8 – 6.7	100	100		

RQD:	Rock Quality Designation
Originated:	FP
Compiled:	JFW
Checked:	CN

TABLE 1

WATER LEVEL READINGS

DATE	BOREHOLE NO. 3-1		BOREHOLE NO. 3-2	
	Ground Surface Elevation 310.4		Ground Surface Elevation 311.4	
	Depth (m)	Elevation	Depth (m)	Elevation
* February 27, 2001	1.0	309.4	1.0	310.4
* April 21, 2001	0.8	309.6	0.8	310.6
** October 25, 2004	1.2	309.2	1.4	310.0
** November 12, 2004	1.3	309.1	1.5	309.9

* From Golder Associates Limited Report.

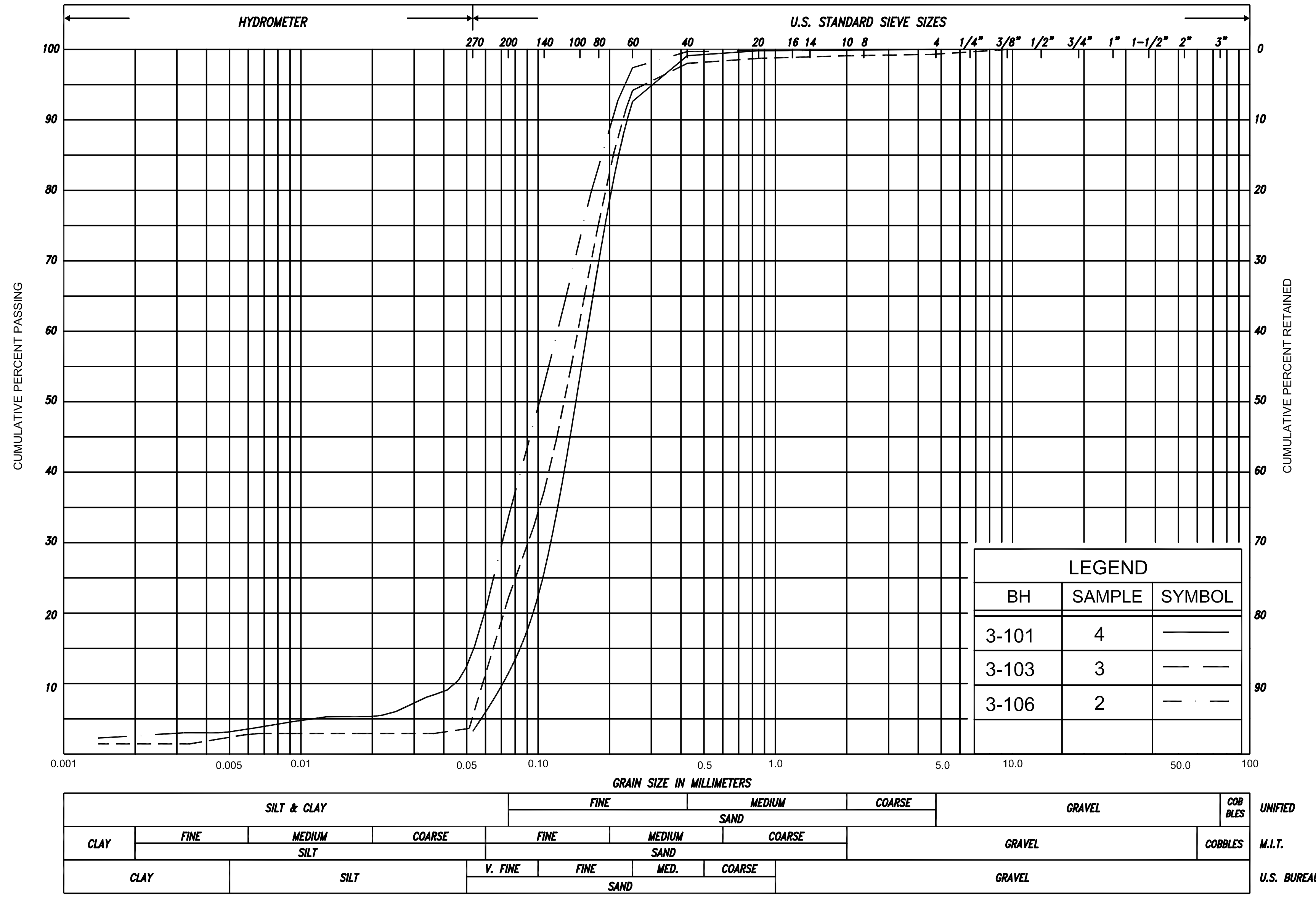
** Measured by Peto MacCallum Ltd.

TABLE 2

ATTERBERG LIMITS SUMMARY

SOIL TYPE	BOREHOLE NO.	SAMPLE NO.	DEPTH (m)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	NATURAL WATER CONTENT (%)
CLAYEY SILT (*) trace to some sand (CL to CL-ML)	3-101	6	6.4	25	18	7	22
	3-102	5	6.4	26	20	6	26
	3-103	6	6.4	28	21	7	27
	3-1	8	6.4	28	18	10	35
	3-2	7	6.4	22	16	6	25
	3-4	7	6.2	26	17	9	25
SILT trace sand trace to some clay (ML to CL-ML)	3-101	8	9.1	26	21	5	27
	3-101	10	12.3	Non-plastic		-	25
	3-102	6	8.0	Non-plastic		-	27
	3-106	4	5.0	23	19	4	27
	3-106	6	7.9	Non-plastic		-	22

NOTES: (*) Soil classified as silty clay in Golder Associates Limited (GAL) boreholes.
Values for GAL boreholes are estimated from record of boreholes.

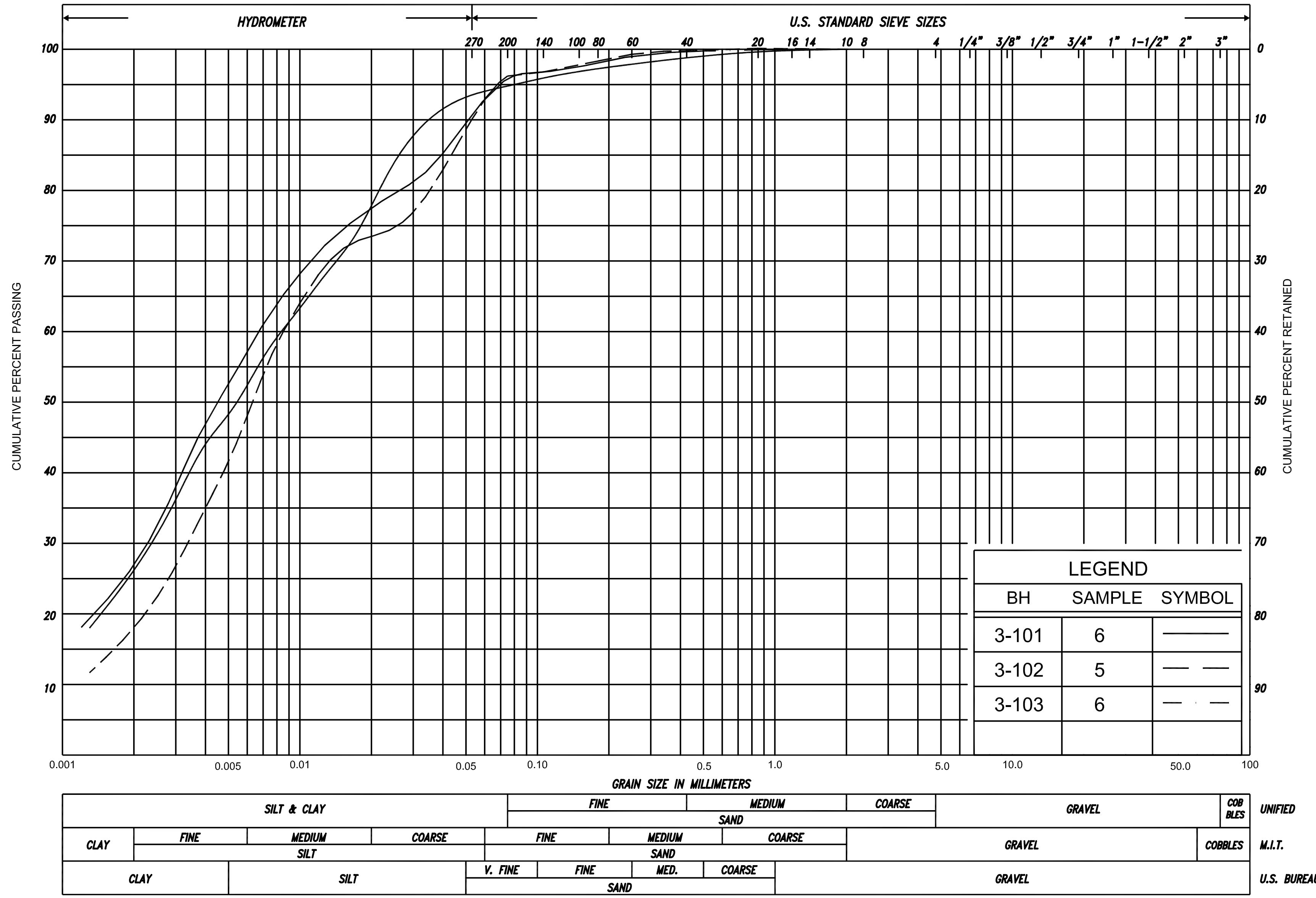


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

SAND, some to with silt trace clay trace gravel

FIG No.	3-1
HWY	11
W.P. No.	62-86-02

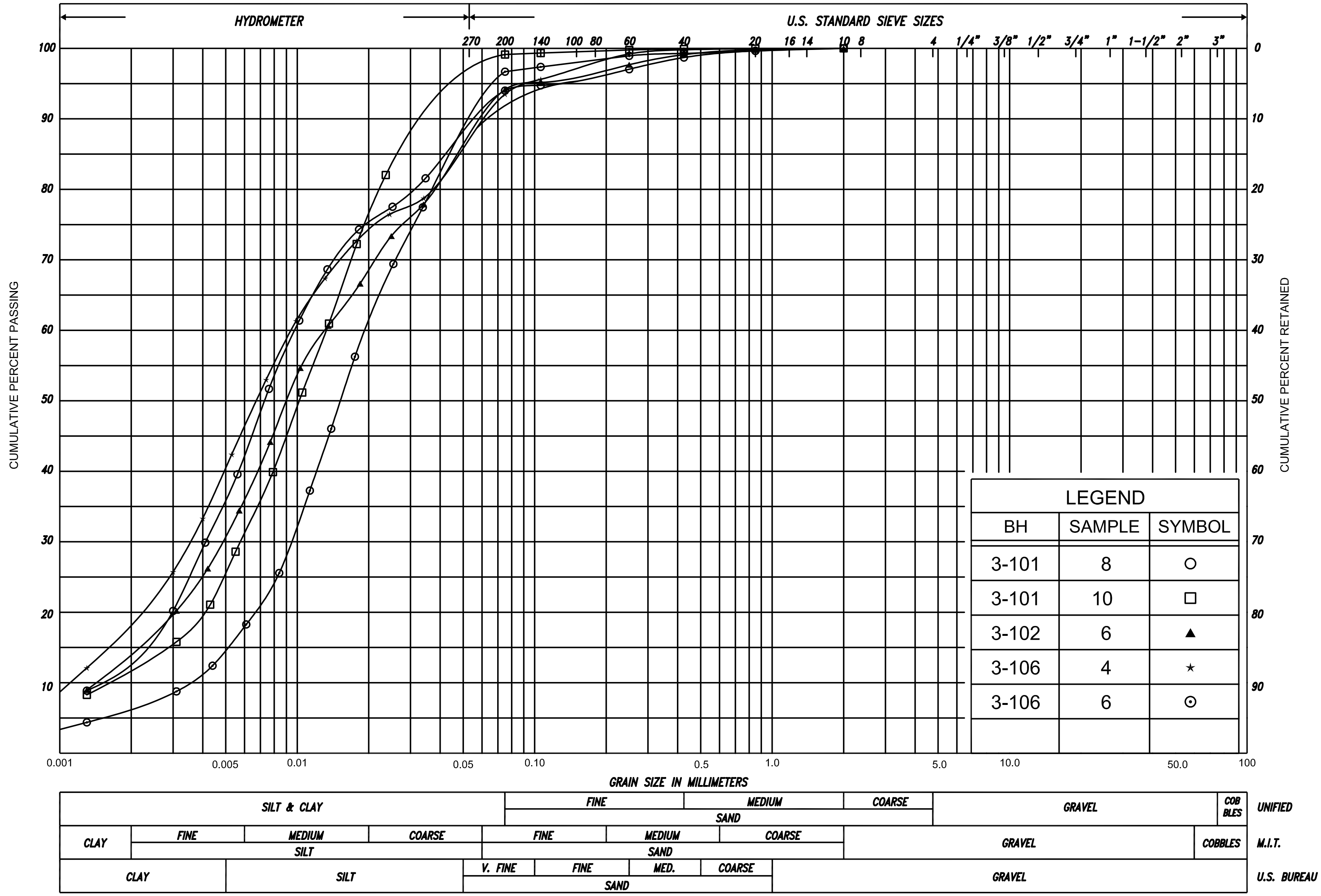


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace to some sand (CL to CL–ML)

FIG No.	3-2
HWY	11
W.P. No.	62-86-02



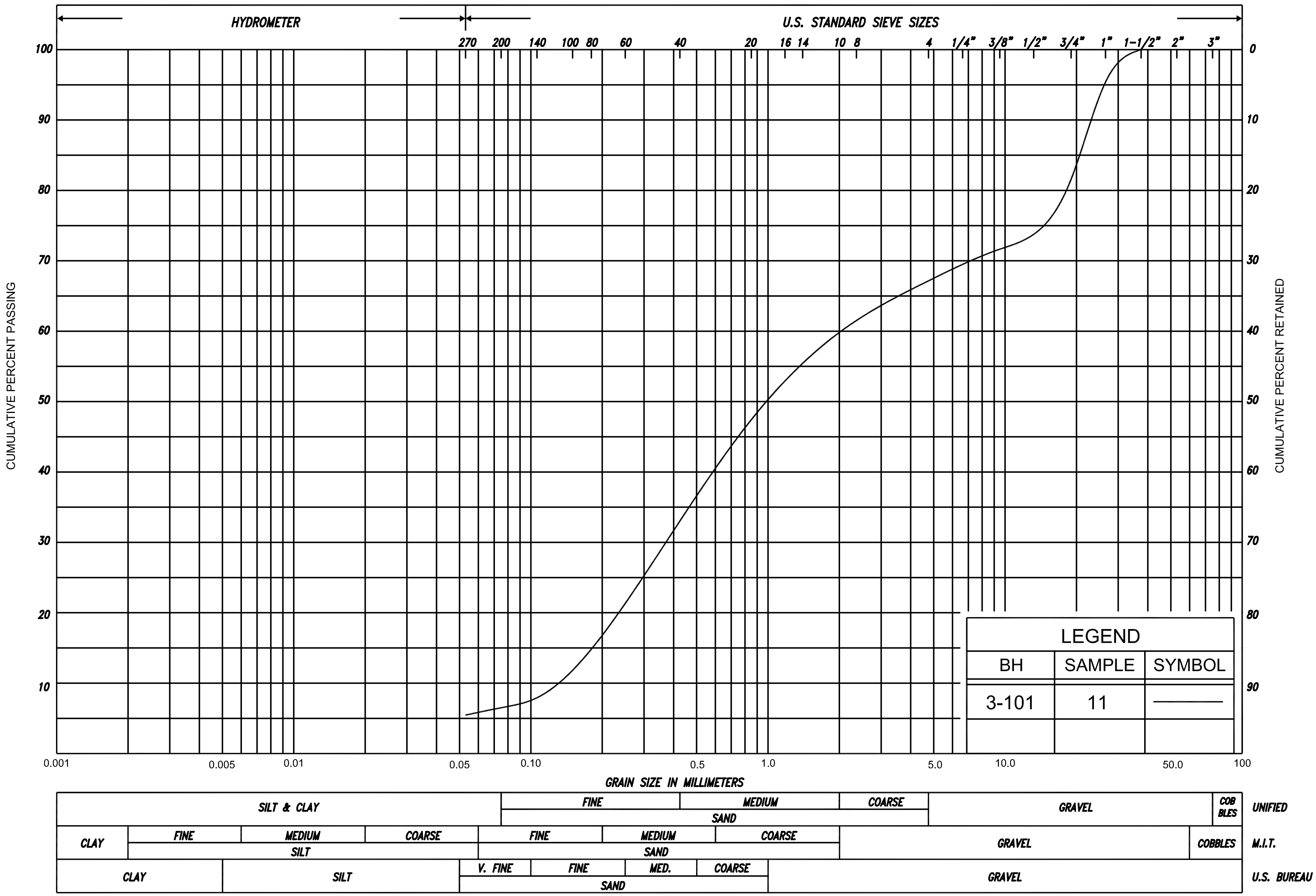
BH	SAMPLE	SYMBOL
3-101	8	○
3-101	10	□
3-102	6	▲
3-106	4	⋆
3-106	6	⊙

Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

SILT, trace sand trace to some clay (ML to CL–ML)

FIG No.	3-3
HWY	11
W.P. No.	62-86-02

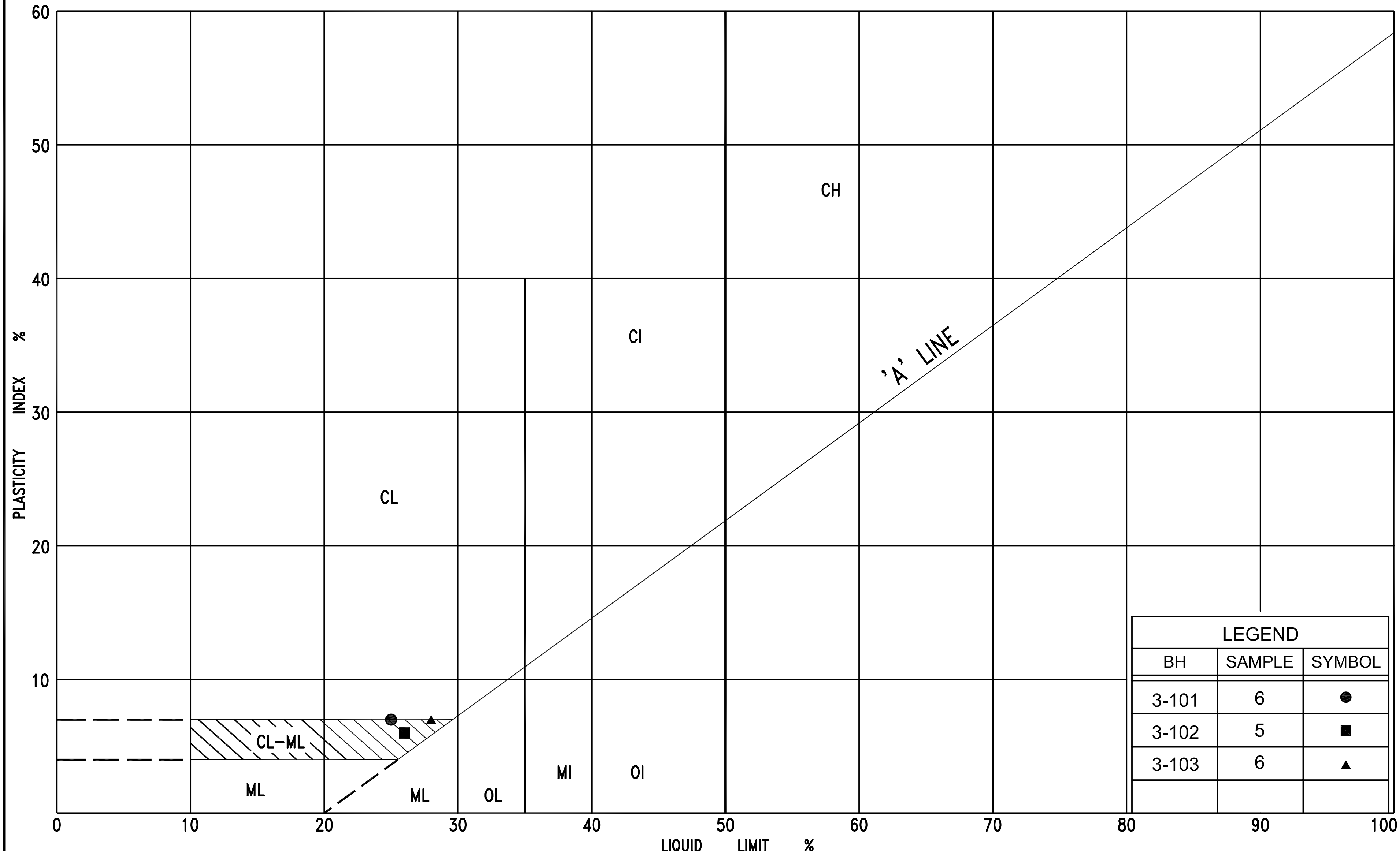


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

GRAVELLY SAND, trace silt

FIG No.	3-4
HWY	11
W.P. No.	62-86-02

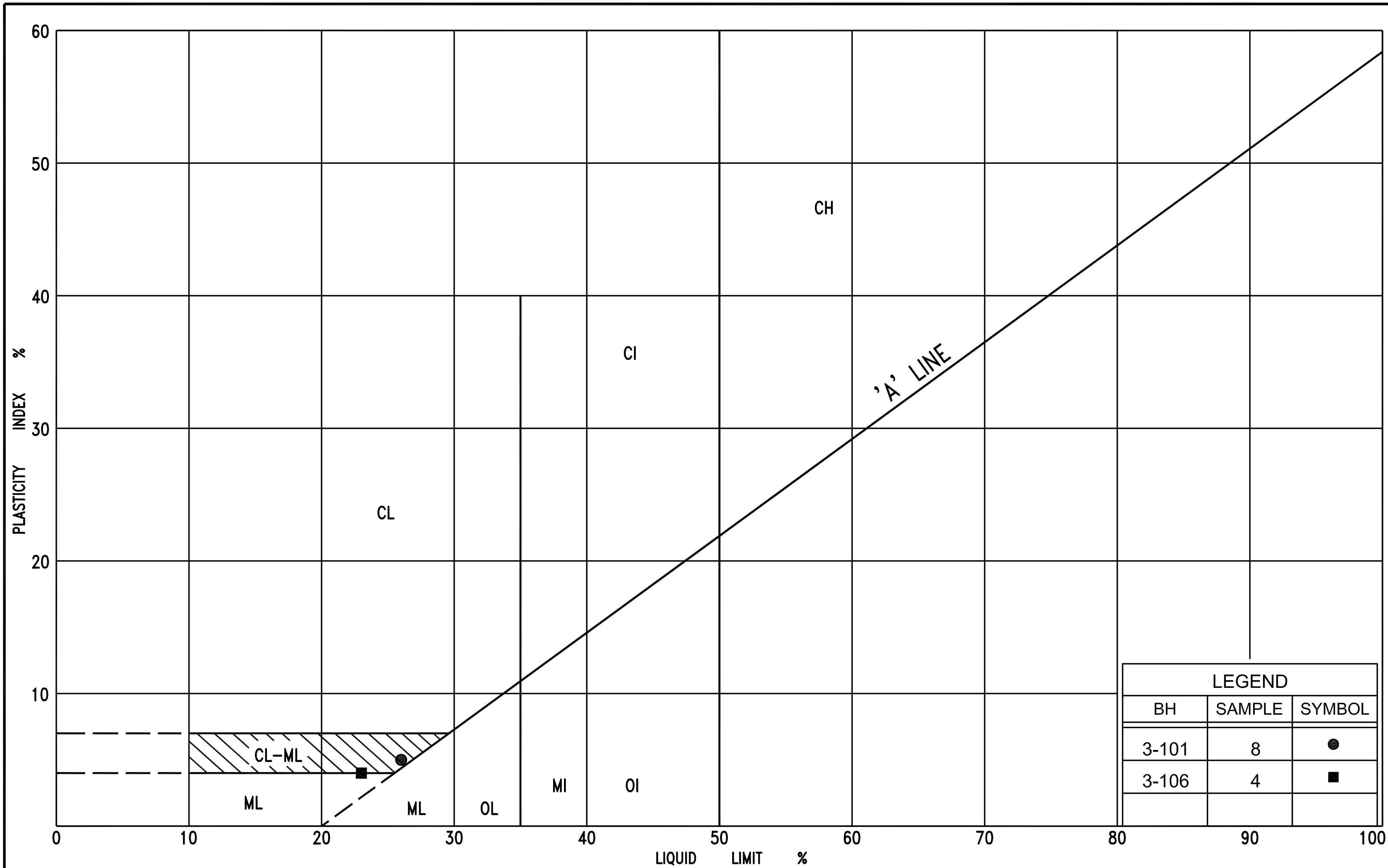


Ministry of Transportation Ontario

PLASTICITY CHART

CLAYEY SILT, trace to some sand (CL to CL–ML)

FIG No.	PC 3-1
HWY	11
W.P. No.	62-86-02



Ministry of Transportation Ontario	PLASTICITY CHART SILT, trace sand trace to some clay (ML to CL-ML)	FIG No.	PC 3-2
		HWY	11
		W.P. No.	62-86-02

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 51mm 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 (51mm 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_u (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/0.3m)	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:

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

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
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
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	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_α	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
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

(Legend Continued)

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
3-105	311.7	5 005 931	319 468
3-106	311.4	5 005 954	319 466
3-107	313.6	5 005 946	319 507
3-108	314.1	5 005 969	319 505
3-109	313.5	5 005 947	319 510
3-110	314.5	5 005 969	319 508
3-111	313.6	5 005 948	319 512
3-112	314.6	5 005 970	319 510
3-113	314.2	5 005 958	319 506
3-114	314.4	5 005 959	319 509
3-115	314.4	5 005 960	319 511
3-116	316.6	5 005 966	319 527

METRIC

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

CONT No

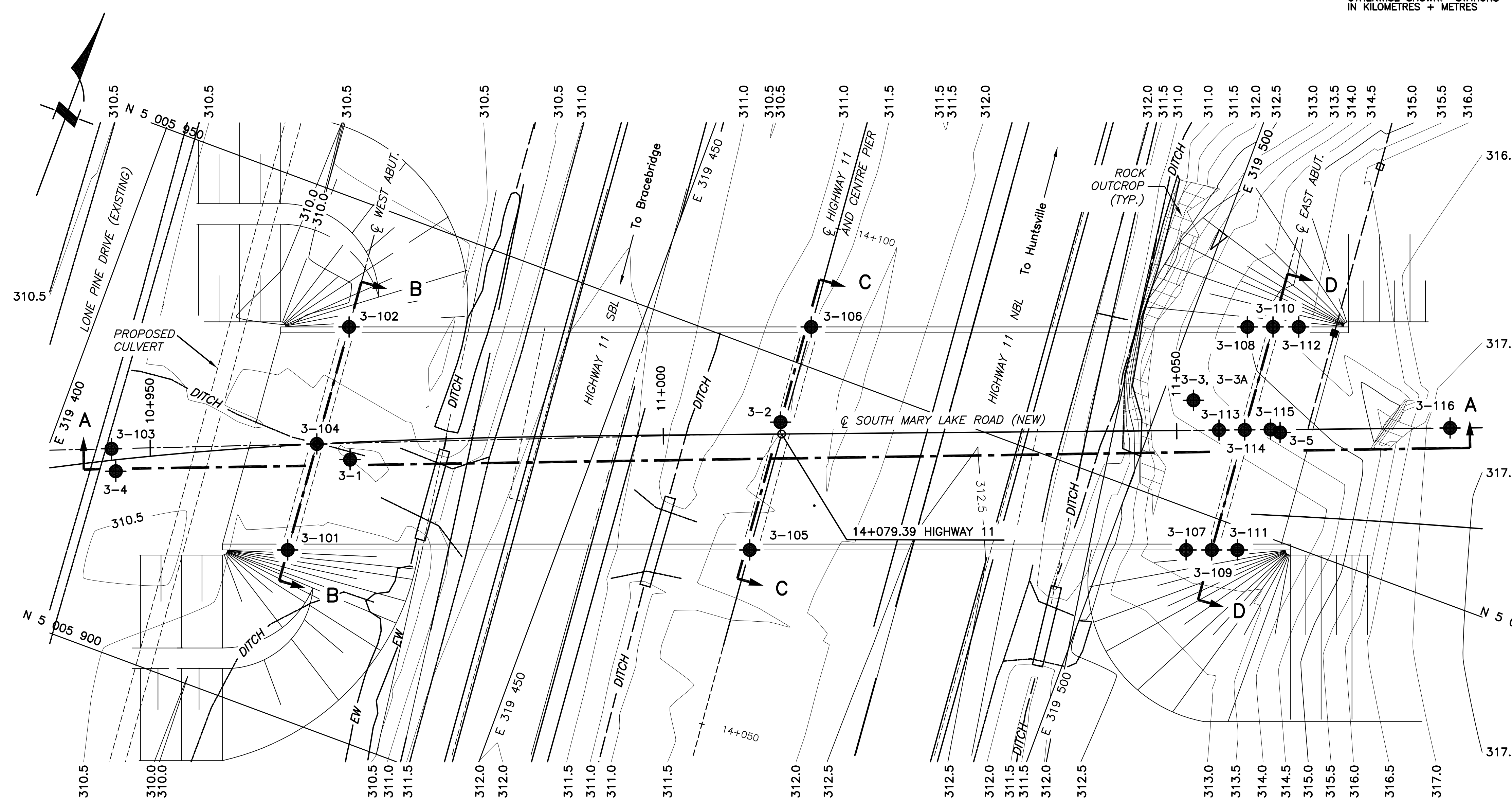
WP No 62-86-02

HIGHWAY 11

SOUTH MARY LAKE ROAD UNDERPASS
BOREHOLE LOCATIONS

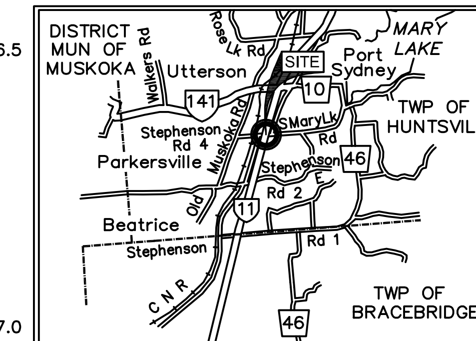
SHEET

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE



KEY PLAN

SCALE

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 (See Note 3)
- Head
- ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
3-1	310.4	5 005 926	319 428
3-2	311.4	5 005 944	319 466
3-3	314.2	5 005 960	319 503
3-3A	314.2	5 005 960	319 503
3-4	310.2	5 005 917	319 407
3-5	314.5	5 005 960	319 512
3-101	310.7	5 005 916	319 425
3-102	310.5	5 005 938	319 423
3-103	310.4	5 005 919	319 406
3-104	310.9	5 005 926	319 424

(Legend Continues)

– NOTE –

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

NOTES:

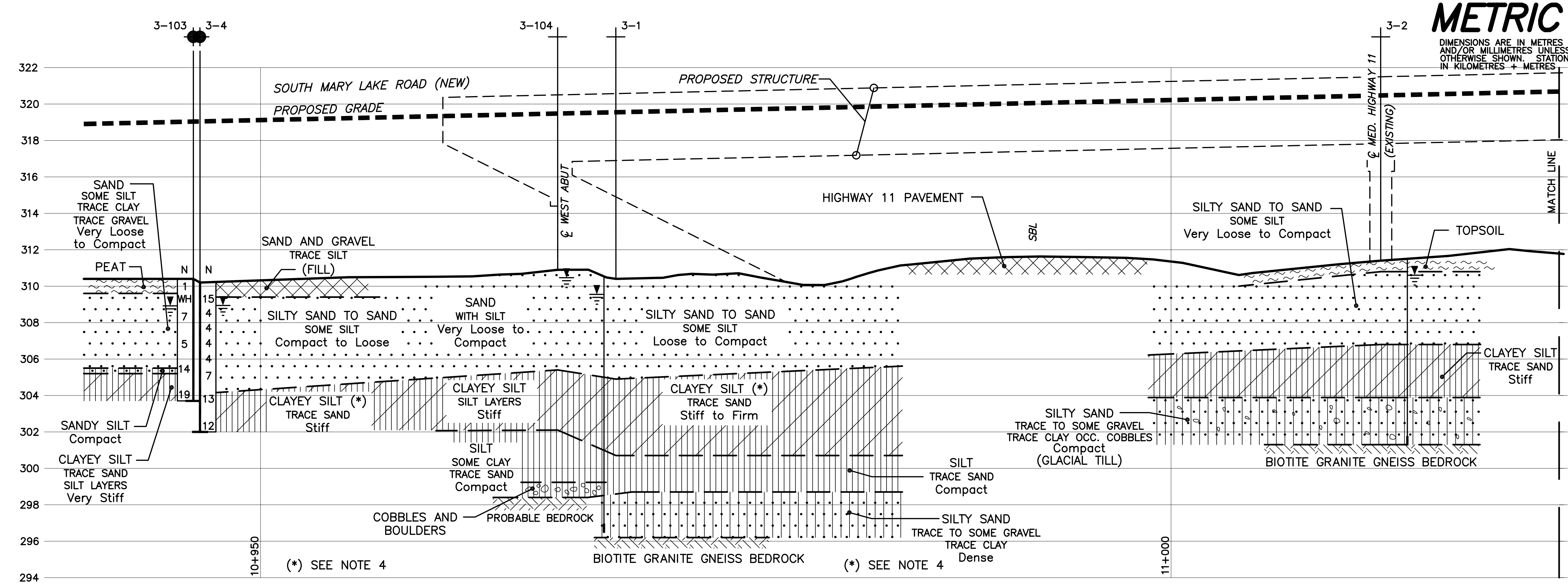
1. BOREHOLES 3-1, 3-2, 3-3, 3-3A, 3-4 AND 3-5 WERE DRILLED BY GOLDER ASSOCIATES; REPORT REFERENCE NO. 011-1104 DATED APRIL 2001
2. REFER TO DRAWING NO. 2 AND 3 FOR SECTIONS A-A, B-B C-C AND D-D.
3. WL OCTOBER 2004 (3-100 SERIES BOREHOLES) WL JANUARY 2001 (3-1 TO 3-5 BOREHOLES)

REF No E-S5568-330GA001.dwg; h5568xB1.dwg; h5568xB2.dwg; October, 2004

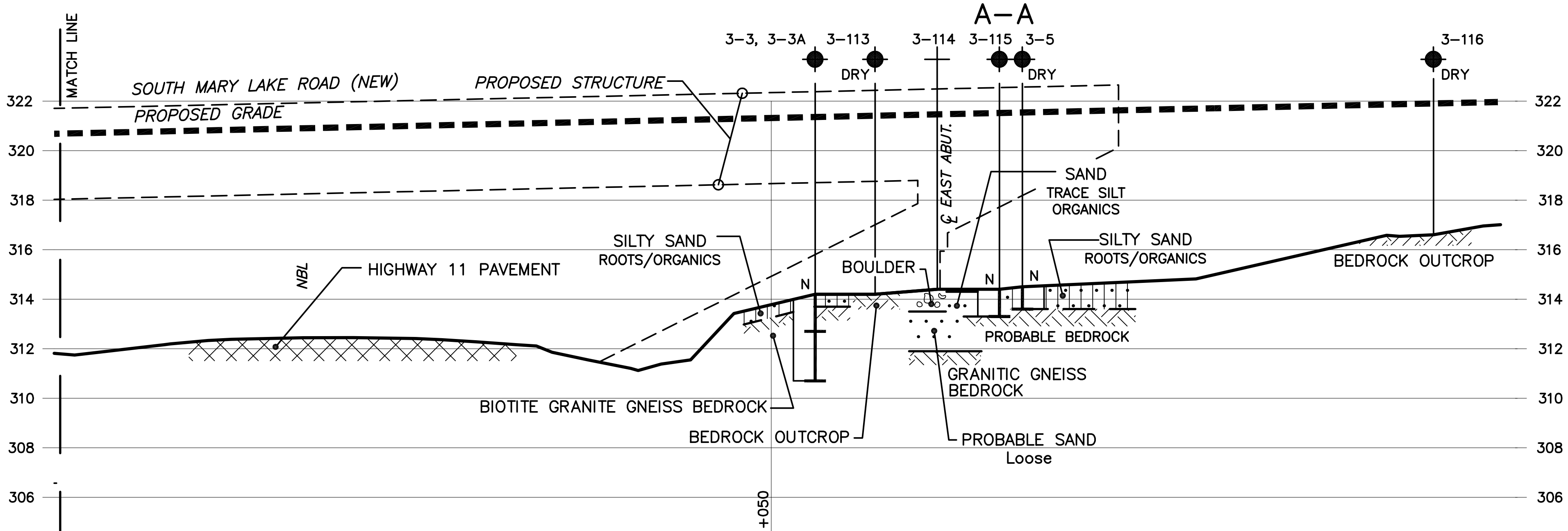
REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31E-228

HWY No 11			DIST 52
SUBM'D FP	CHECKED CN	DATE MAY 12, 2005	SITE 42-193
DRAWN NA/MM	CHECKED BRG	APPROVED DWK	DWG 1



A–A



A–A (Continued)

SECTIONS

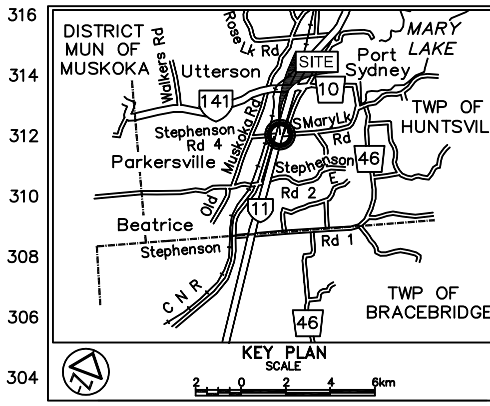
SCALE 2.5 0 2.5 5m

METRIC

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

CONT No WP No 62-86-02	
HIGHWAY 11 SOUTH MARY LAKE ROAD 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 (See Note 3)
- Head
- ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
(Refer to drawing no. 1 for co-ordinates)			

NOTES:

1. BOREHOLES 3-1, 3-2, 3-3, 3-3A, 3-4 AND 3-5 WERE DRILLED BY GOLDER ASSOCIATES; REPORT REFERENCE NO. 011-1104 DATED APRIL 2001
2. REFER TO DRAWING NO. 1 FOR PLAN AND DRAWING NO. 3 FOR SECTIONS B-B TO D-D.
3. WL OCTOBER 2004 (3-100 SERIES BOREHOLES)
   WL JANUARY 2001 (3-1 TO 3-5 BOREHOLES)
4. SOIL IDENTIFIED AS SILTY CLAY IN BOREHOLES 3-1 AND 3-4 CHANGED TO CLAYEY SILT BASED ON REPORTED LIQUID LIMIT OR FOR CONSISTENCY
5. 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.

REVISIONS DATE	BY	DESCRIPTION

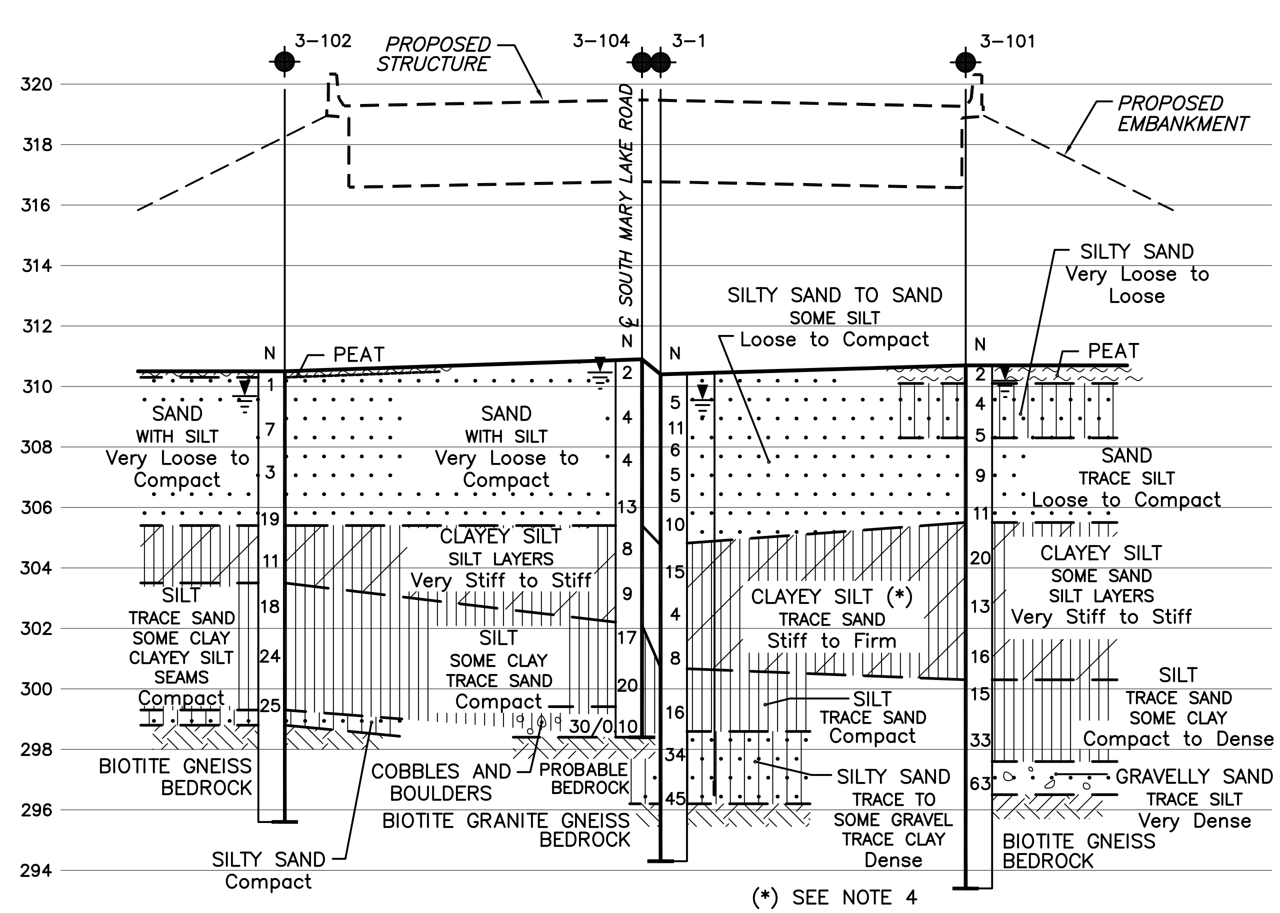
Geocres No. 31E-228

HWY No 11			DIST 52
SUBM'D FP	CHECKED CN	DATE MAY 12, 2005	SITE 42-193
DRAWN NA/MM	CHECKED BRG	APPROVED DWK	DWG 2

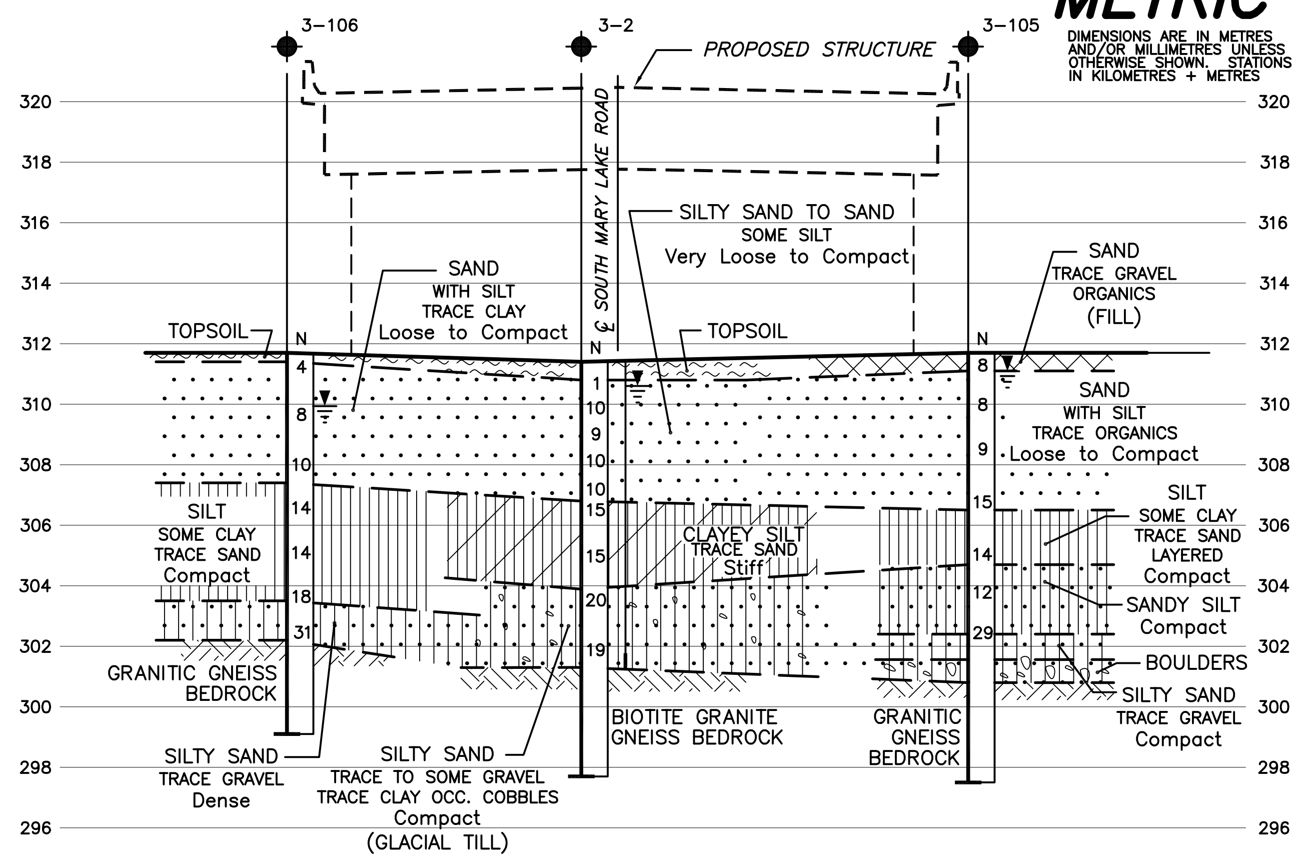
REF No E-S5568-330GA001.dwg; h5568xB1.dwg; h5568xB2.dwg; October, 2004

METRIC

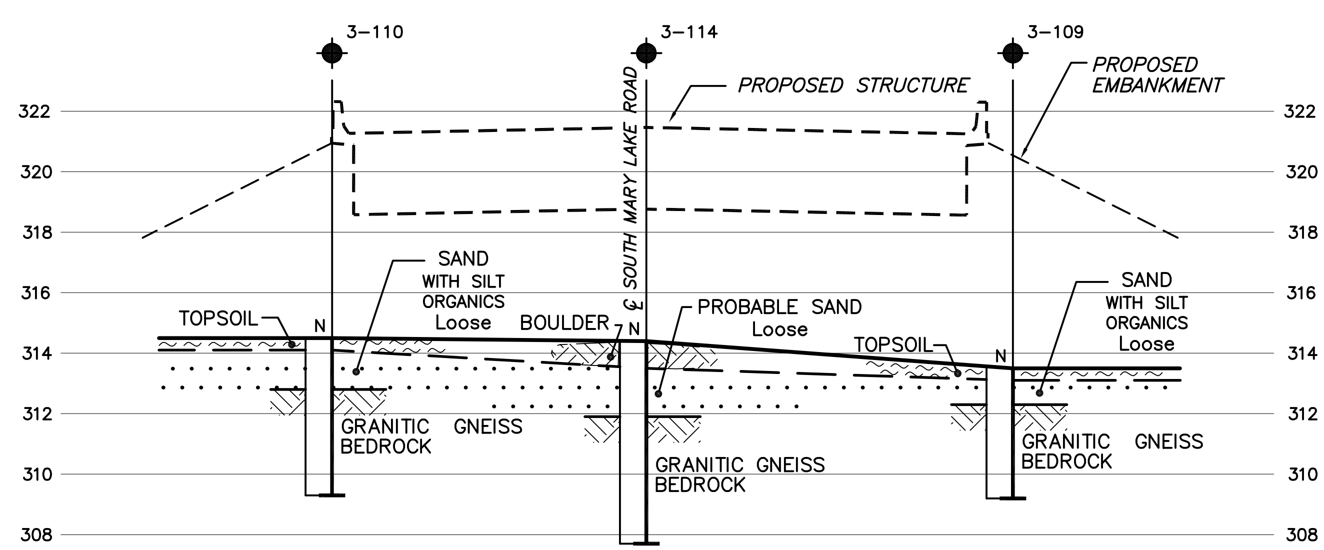
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES



B–B



C–C



D–D

SECTIONS

SCALE 2.5 0 2.5 5m

NOTES:

1. BOREHOLES 3–1, 3–2, 3–3, 3–3A, 3–4 AND 3–5 WERE DRILLED BY GOLDER ASSOCIATES; REPORT REFERENCE NO. 011–1104 DATED APRIL 2001
2. REFER TO DRAWING NO. 1 FOR PLAN AND DRAWING NO. 2 FOR SECTION A–A.
3. WL OCTOBER 2004 (3–100 SERIES BOREHOLES)
   WL JANUARY 2001 (3–1 TO 3–5 BOREHOLES)
4. SOIL IDENTIFIED AS SILTY CLAY IN BOREHOLE 3–1 CHANGED TO CLAYEY SILT BASED ON REPORTED LIQUID LIMIT OR FOR CONSISTENCY
5. 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.

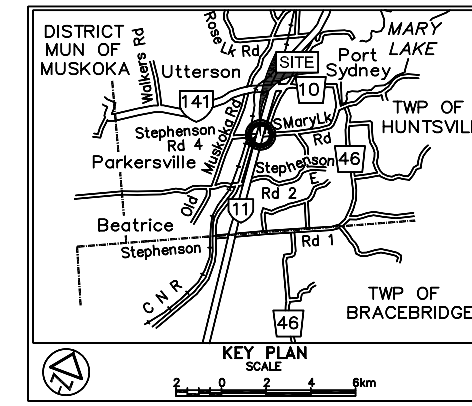
REF No E–S5568–330GA001.dwg; h5568xB1.dwg; h5568xB2.dwg; October, 2004

CONT No
WP No 62–86–02

SHEET

HIGHWAY 11
SOUTH MARY LAKE ROAD UNDERPASS
SOIL STRATA

Peto MacCallum Ltd.
CONSULTING ENGINEERS



KEY PLAN
SCALE 2 0 2 4 6km

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 (See Note 3)
- Head ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
Refer to drawing no. 1 for co-ordinates			

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

REVISIONS DATE	BY	DESCRIPTION

Geocres No. 31E–228

HWY No 11				DIST	52
SUBM'D FP	CHECKED CN	DATE MAY 12, 2005		SITE	42–193
DRAWN NA/MM	CHECKED BRG	APPROVED DWK		DWG	3

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-101

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 916 N; 319 425 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Mud Rotary and NQ Coring COMPILED BY F.P.

DATUM Geodetic DATE October 18 and 19, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.7	Ground Surface				
0.0	Peat, fine fibrous, Dark brown	1	SS	2	
310.1 / 0.6	Silty sand, fine; Very loose, Brown, Wet	2	SS	4	
	Loose, Grey	3	SS	5	
308.3 / 2.4	Sand, some silt; Loose, Grey, Wet	4	SS	9	0 88 (12)
	Compact	5	SS	11	
305.5 / 5.2	Clayey silt, trace sand silt layers; Very stiff, Grey, Wet	6	SS	20	0 5 69 26
	Stiff	7	SS	13	
301.6 / 9.1	Silt, trace sand some clay; Compact, Grey, Wet	8	SS	16	0 6 81 13
		9	SS	15	
	Dense	10	SS	33	0 1 87 12 (Non-plastic)
297.6 / 13.1	Gravelly sand, trace silt; Very dense, Grey, Wet	11	SS	63	32 62 (6)
296.5 / 14.2	Biotite Gneiss Bedrock; Cont'd	12	RC NQ	REC 100%	RQD 31%

Column headings: SOIL PROFILE (ELEV/DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (310 to 296); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 – ○ – 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3-102

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 938 N; 319 423 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers and NQ Coring COMPILED BY F.P.

DATUM Geodetic DATE October 19 & 20, 2004 CHECKED BY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
310.5	Ground Surface
0.0	Peat, fine fibrous, Dark brown
0.2	Sand, with silt, topsoil to 0.9 m Very loose, Brown, Moist Loose, Wet Grey Compact
305.4 / 5.1	Clayey silt, trace sand Very stiff, Grey, Wet Stiff
303.5 / 7.0	Silt, trace sand some clay, clayey silt seams Compact, Grey, Wet
299.3 / 11.2	Silty sand Compact, Grey, Wet
298.8 / 11.7	Biotite Gneiss Bedrock Medium to high strength Unweathered Fair to good quality Refer to Table A for detailed description
295.6	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	1	
2	SS	7	
3	SS	5	
4	SS	19	
5	SS	11	0 4 78 18
6	SS	18	0 6 80 14 (Non-plastic)
7	SS	24	
8	SS	25	
9	RC NQ	REC 98%	RQD 56%
10	RC NQ	REC 91%	RQD 64%
11	RC NQ	REC 83%	RQD 83%

GROUND WATER CONDITIONS: ∇* (approx. elev. 309.6)

ELEVATION SCALE: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 3-102

METRIC

W.P.	62-86-00	LOCATION	Co-ords. 5 005 938 N; 319 423 E	ORIGINATED BY	F.P.
DIST	52 HWY 11	BOREHOLE TYPE	Hollow Stem Augers and NQ Coring	COMPILED BY	F.P.
DATUM	Geodetic	DATE	October 19 & 20, 2004	CHECKED BY	

SOIL PROFILE			SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE					
295.5 / 14.9	End of borehole											

* 2004 10 19

** 2004 10 20

∇ Water level observed during drilling

■ Penetrometer test

Borehole charged with drilling water

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.

Foundation Design

RECORD OF BOREHOLE No 3-103

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 919 N; 319 406 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.

DATUM Geodetic DATE October 20, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
310.4	Ground Surface												
0.0	Peat, coarse fibrous, Dark brown	1	SS	1	∇*	310		272					
309.6 / 0.8	Sand, some silt, trace clay trace gravel organics to 1.3 m; Very loose to loose, Brown, Wet	2	SS	WH**		309							
		3	SS	7	▼*					1	78	19	2
	Grey					308							
		4	SS	5		307							
	Compact					306							
305.5 / 4.9	Sandy silt, Compact, Grey, Wet	5	SS	14									
305.2 / 5.2	Clayey silt, trace sand silt layers; Very stiff, Grey, Wet					305							
		6	SS	19		304	■			0	4	69	27
303.7 / 6.7	End of borehole												

* 2004 10 20

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

WH** Refers to penetration under weight of rods and hammer

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

$+^7$, $\times^5$: Numbers refer to Sensitivity; 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3-101

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 916 N; 319 425 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Mud Rotary and NQ Coring COMPILED BY F.P.

DATUM Geodetic DATE October 18 and 19, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
295.7	Cont'd Biotite Gneiss Medium to high strength Unweathered Poor to fair, becoming excellent quality Refer to Table A for detailed description		13	RC NQ	REC 100%		295 294				RQD 57%
			14	RC NQ	REC 100%						RQD 96%
293.4 17.3	End of borehole										

* 2004 10 18

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Pocket penetrometer test

Borehole charged with drilling water on October 19, 2004

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-104

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 926 N; 319 424 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.

DATUM Geodetic DATE October 19, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS
310.9	Ground Surface							
0.0	Sand, with silt topsoil to 0.6 m Very loose Brown Moist	1	SS	2	▽*			
	Wet					310		
		2	SS	4		309		
						308		
		3	SS	4		307		
	Compact Grey	4	SS	13		306		
305.4 5.5	Clayey silt silt layers Stiff Grey Wet					305		
		5	SS	8			>200	
						304		
		6	SS	9		303	175	
302.1 8.8	Silt, some clay trace sand clayey silt seams Compact Grey APL					302		
		7	SS	17		301		
	no clayey silt seams	8	SS	20		300		
	cobbles and boulders				▼*	299		
		9	SS	30/ 10cm				
298.4 12.5	End of borehole Refusal on probable bedrock							

* 2004 10 19

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Pocket penetrometer tests

** 2.1 m of blow back at 6.1 m depth

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^7$, $\times^5$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-105

METRIC

W.P. 62-86-00	LOCATION Co-ords. 5 005 931 N; 319 468 E		ORIGINATED BY F.P.
DIST 52 HWY 11	BOREHOLE TYPE Hollow Stem Augers and NQ Coring		COMPILED BY F.P.
DATUM Geodetic	DATE October 21, 2004		CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.7	Ground Surface						
0.0	Sand, trace gravel, organics Brown (FILL)	1	SS	8	∇*		
311.1 / 0.6	Sand, with silt, trace clay organics Loose Brown Wet					311	
		2	SS	8		310	
	Grey					309	
		3	SS	9		308	
	Compact Brown						
		4	SS	15		307	
306.5 / 5.2	Silt some clay, trace sand layered Compact Grey Wet					306	
		5	SS	14		305	
304.7 / 7.0	Sandy silt Compact Grey Wet						
		6	SS	12		304	
						303	
302.4 / 9.3	Silty sand, trace gravel Compact Grey Wet	7	SS	29		302	
	Boulders					301	
300.8 / 10.9	Granitic Gneiss Bedrock Medium to high strength Unweathered Fair to excellent quality Refer to Table A for detailed description	8	RC NQ	RCD 91%		300	RQD 55%
		9	RC NQ	RCD 100%		299	RQD 100%
		10	RC NQ	RCD 97%		298	RQD 73%
297.5 / 14.2	End of borehole Cont'd						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ✚ FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3-105

METRIC

W.P. 62-86-00	LOCATION Co-ords. 5 005 931 N; 319 468 E		ORIGINATED BY F.P.
DIST 52 HWY 11	BOREHOLE TYPE Hollow Stem Augers and NQ Coring		COMPILED BY F.P.
DATUM Geodetic	DATE October 21, 2004		CHECKED BY

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
296.7	(Cont'd)										
	* 2004 10 21										
	▽ Water level observed during drilling										
	Borehole charged with drilling water										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-106

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 954 N; 319 466 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augers and NQ Coring COMPILED BY F.P.

DATUM Geodetic DATE October 21, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.4	Ground Surface							
0.0	Topsoil							
0.3	Sand, with silt trace clay organics to 0.6 m; Loose Brown Moist	1	SS	4		311		
	Wet	2	SS	8	∇*	310		0 67 30 3
	Compact; Grey	3	SS	10		309, 308		
307.1 / 4.3	Silt, some clay, trace sand; Compact Grey Wet	4	SS	14		307		0 6 76 18
	sandy silt layers	5	SS	14		306, 305		
	trace clay	6	SS	18		304	Non-plastic	0 4 90 6
303.2 / 8.2	Silty sand, trace gravel; Dense Grey Wet	7	SS	31		303, 302		
301.9 / 9.5	Granitic Gneiss Bedrock; High strength; Unweathered; Fair to excellent quality; Refer to Table A for detailed description	8	RC NQ	REC 100%		301		RQD 64%
		9	RC NQ	REC 100%		300		RQD 100%
298.8		10	RC NQ	REC 67%		299		RQD 67%
12.6	End of borehole							

* 2004 10 21

∇ Water level observed during drilling

Borehole charged with drilling water

+7, ×5: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-107

METRIC

W.P. 62-86-00	LOCATION Co-ords. 5 005 946 N; 319 507 E		ORIGINATED BY F.P.
DIST 52 HWY 11	BOREHOLE TYPE Manual Hand Sampler		COMPILED BY F.P.
DATUM Geodetic	DATE October 22, 2004		CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	*					
313.6	Ground Surface										
0.0	Boulders at surface										
	* Borehole dry										

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 3-108

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 969 N; 319 505 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.1	Ground Surface										
0.0	Peat, coarse fibrous Dark brown						314				
0.2	Sand, trace silt, organics Loose Brown Moist										
313.2 / 0.9	End of borehole Refusal on probable bedrock										

* Borehole dry on completion of drilling

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 –O– 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3-109

METRIC

W.P. 62-86-00 | LOCATION Co-ords. 5 005 947 N; 319 510 E | ORIGINATED BY F.P.

DIST 52 HWY 11 | BOREHOLE TYPE Hollow Stem Augers and NQ Coring | COMPILED BY F.P.

DATUM Geodetic | DATE October 22, 2004 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.5	Ground Surface										
0.0	Topsoil										
313.1 / 0.4	Sand with silt, organics Loose Brown Moist						313				
312.3 / 1.2	Granitic Gneiss Bedrock High strength Unweathered Excellent quality Refer to Table A for detailed description		1	RC NQ	REC 100%		312 311				RQD 96%
			2	RC NQ	REC 100%		310				RQD 100%
309.2 / 4.3	End of borehole										

* Borehole charged with drilling water

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

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No 3-110

METRIC

W.P. 62-86-00	LOCATION Co-ords. 5 005 969 N; 319 508 E		ORIGINATED BY F.P.
DIST 52 HWY 11	BOREHOLE TYPE Hollow Stem Augers and NQ Coring		COMPILED BY F.P.
DATUM Geodetic	DATE October 22, 2004		CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.5	Ground Surface										
0.0	Topsoil										
314.1 / 0.4	Sand with silt, organic Loose Brown Moist						314				
							313				
312.8 / 1.7	Granitic Gneiss Bedrock High strength Unweathered Good to excellent quality Refer to table A for detailed description		1	RC NQ	REC 92%		312				RQD 92%
							311				
			2	RC NQ	REC 99%		310				RQD 80%
309.3 / 5.2	End of borehole										
	* Borehole charged with drilling water										

+7, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-111

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 948 N; 319 512 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.6	Ground Surface										
0.0 / 0.1	Peat, coarse fibrous, Dark brown										
313.0 / 0.6	Sand, trace silt, organics, Loose Brown Moist						313				
	End of borehole										
	Refusal on probable bedrock										
	* Borehole dry on completion of drilling										

ON_MOT VER 3A 04TF006_SMR.GPJ ON_MOT.GDT 12/13/04 8:59:24 AM

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-112

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 970 N; 319 510 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.6	Ground Surface										
0.0 / 0.1	Peat, coarse fibrous Dark brown										
	Sand, trace silt, organics Loose Brown Moist						314				
313.8 / 0.8	End of borehole Refusal on probable bedrock										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-113

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 958 N; 319 506 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE				
314.2	Ground Surface										
0.0	Bedrock at surface										
	* Borehole dry										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-114

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 959 N; 319 509 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Washboring and NQ Coring COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
314.4	Ground Surface				
0.0	Boulder (0.9 m thick)				
313.5 / 0.9	Probable sand Brown				
311.9 / 2.5	Granitic Gneiss Bedrock High strength Unweathered Excellent quality Refer to Table A for detailed description	1	RC NQ	REC 100%	RQD 100%
		2	RC NQ	REC 97%	RQD 97%
		3	RC NQ	REC 100%	RQD 100%
307.7 / 6.7	End of borehole				

* Borehole charged with drilling water

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (314, 313, 312, 311, 310, 309, 308); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+7, ×5: Numbers refer to Sensitivity 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-115

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 960 N; 319 511 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.4	Ground Surface					*					
0.0	Peat, coarse fibrous Dark brown										
0.1	Sand, trace silt, organics Loose Brown Moist						314				
313.3 / 1.1	End of borehole Refusal on probable bedrock										

* Borehole dry on completion of drilling

Legend: SOIL PROFILE; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3-116

METRIC

W.P. 62-86-00 LOCATION Co-ords. 5 005 966 N; 319 527 E ORIGINATED BY F.P.

DIST 52 HWY 11 BOREHOLE TYPE Manual Hand Sampler COMPILED BY F.P.

DATUM Geodetic DATE October 22, 2004 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	*					
316.6	Ground Surface										
0.0	Bedrock at surface										
	* Borehole dry										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

APPENDIX A

Previous Record of Boreholes and Laboratory Test Results

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3-1

METRIC

PROJECT 011-1104
W.P. 62-86-00
LOCATION N 5005926; E 319528
ORIGINATED BY SB
DIST 52 HWY 11
BOREHOLE TYPE 108mm Hollow Stem Augers
COMPILED BY DKB
DATUM Geodetic
DATE Jan.31/01
CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.4	GROUND SURFACE						
0.0	Silty Sand to Sand, some silt, occ. organics to 3.0m depth Loose to compact Brown Moist to wet				310		
		1	SS	5	309		
		2	SS	11		o	
		3	SS	6	308		
		4	SS	5	307		
		5	SS	5	306	o	0 82 18 0
		6	SS	10	305		
304.9 5.5	Silty Clay, trace sand, occ. thin sand interlayers Stiff to firm Grey Moist to wet						
		7	SS	15	304	o	
		8	SS	4	303	├─┤ o	0 7 72 21
		9	SS	8	302 / 301		
300.7 9.7	Silt, trace sand Compact Grey Wet				300		
		10	SS	16	299	o	0 2 98 0
298.7 11.7	Silty Sand, trace to some gravel, trace clay, occ. cobbles (Glacial Till) Dense Grey Wet						
		11	SS	34	298		
		12	SS	45	297	o	
296.2 14.2					296		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 3-1

METRIC

PROJECT 011-1104

W.P. 62-86-00

LOCATION N 5005926; E 319528

ORIGINATED BY SB

DIST 52 HWY 11

BOREHOLE TYPE 108mm Hollow Stem Augers

COMPILED BY DKB

DATUM Geodetic

DATE Jan.31/01

CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
294.3 16.1	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 14.2m to 16.1m depth. For bedrock coring details refer to Record of Drillhole 3-1						295				
	END OF HOLE Note: 1. Water level measured in piezometer at 6.5m (El. 303.9m) upon completion of installation. 2. Water level measured in piezometer at 1.0m depth (El. 309.4m) on February 27,2001. 3. Water level measured in piezometer at 0.8m depth (El. 309.6m) on April 21,2001.										

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

ON_MOT_OLD 011-1104.GPJ ON_MOT.GDT 5/3/04

RECORD OF DRILLHOLE: 3-1

PROJECT: 011-1104
LOCATION: N 5005926, E 319428
INCLINATION: -90°   AZIMUTH: —

DRILLING DATE: January 31, 2001
DRILL RIG: CME 55 BOMBARDIER
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH / COLOUR % RETURN	RECOVERY: TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				296.20											
15	Rotary Drilling NQ Size	Biotite granite GNEISS Dark grey-black with occasional 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.		14.20	1							J,PR J,PR			
16				294.36 16.05	2							J,PR			
17–24		END OF HOLE													

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: PDG

RECORD OF BOREHOLE No 3-2

METRIC

PROJECT 011-1104			
W.P. 62-86-00	LOCATION N 5005944; E 319466	ORIGINATED BY SB
DIST 52	HWY 11	BOREHOLE TYPE 108mm Hollow Stem Augers	COMPILED BY DKB
DATUM Geodetic	DATE Feb.20/01	CHECKED BY ASP	

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
311.4	GROUND SURFACE
0.0	Topsoil
310.8 0.6	Silty Sand to Sand, some silt, occ. organics to 2.1m depth Very loose to compact Brown, becoming grey below 3.0m depth Moist to wet
306.8 4.6	Clayey Silt, trace sand Stiff Grey Moist
303.9 7.5	Silty Sand, trace to some gravel, trace clay, occ. cobbles (Glacial Till) Compact Grey Wet
301.3 10.1	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 10.1m to 13.7m depth. For bedrock coring details refer to Record of Drillhole 3-2
297.7 13.7	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	1	
2	SS	10	
3	SS	9	0 78 22
4	SS	10	
5	SS	10	
6	SS	15	
7	SS	15	0 6 94 0
8	SS	20	
9	SS	19	

GROUND WATER CONDITIONS

ELEVATION SCALE: 311, 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 3-2

METRIC

PROJECT 011-1104

W.P. 62-86-00 LOCATION N 5005944; E 319466 ORIGINATED BY SB

DIST 52 HWY 11 BOREHOLE TYPE 108mm Hollow Stem Augers COMPILED BY DKB

DATUM Geodetic DATE Feb.20/01 CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	END OF HOLE Note: 1. Water level measured in piezometer at 10.0m (El. 301.4m) upon completion of installation. 2. Water level measured in piezometer at 1.0m depth (El. 310.4m) on February 27,2001. 3. Water level measured in piezometer at 0.8m depth (El. 310.6m) on April 21,2001.										

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

PROJECT: 011-1104

LOCATION: N 5005944; E 319466

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 3-2

DRILLING DATE: February 1, 2001

DRILL RIG: CME 55 BOMBARDIER

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				301.35												
	Rotary Drilling NQ Size	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.		10.05	1											
11					2								J,PR J,PR			
12													J,PR (FeO) J,PR (FeO)			
13					3											
				297.70 13.70												
14		END OF HOLE														
15																
16																
17																
18																
19																
20																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: PDG

Ministry of Transportation Ontario

Foundation Design

PROJECT 011-1104

RECORD OF BOREHOLE No 3-3

METRIC

W.P. 62-86-00 | LOCATION N 5005960; E 319503 | ORIGINATED BY SB

DIST 52 HWY 11 | BOREHOLE TYPE 108mm Hollow Stem Augers | COMPILED BY DKB

DATUM Geodetic | DATE Feb.1/01 | CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.2	GROUND SURFACE										
0.0	Silty Sand with roots/organics Blackish brown Wet						314				
313.7 0.5	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 0.5m to 1.5m depth. For bedrock coring details refer to Record of Drillhole 3-3						313				
312.7 1.5	END OF HOLE										

Note:
Core barrel burnt out at 1.5m depth; unable to continue hole.
Water used during coring operations; water level on completion of drilling not representative of groundwater conditions.

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

ON_MOT_OLD 011-1104.GPJ ON_MOT.GDT 5/3/04

PROJECT: 011-1104

LOCATION: N 5005960; E 319503

INCLINATION: -90°   AZIMUTH: —

RECORD OF DRILLHOLE: 3-3

DRILLING DATE: February 1, 2001

DRILL RIG: CME 55 BOMBARDIER

DRILLING CONTRACTOR: Marathon Drilling

SHEET 2 OF 2

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				313.74												
1	Rotary Drilling NQ Size	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.		0.46 312.68 1.52	1								J,R,P			
2		END OF HOLE Note: Core barrel burnt out at 1.5m depth; unable to continue hole.														
3																
4																
5																
6																
7																
8																
9																
10																

MISS_ROCK 1104ROCK.GPJ GLDR_CAN.GDT 5/3/04 MMZ

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB

CHECKED: PDG

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 3-3A

METRIC

PROJECT 011-1104
W.P. 62-86-00 LOCATION N 5005960; E 319503 ORIGINATED BY SB
DIST 52 HWY 11 BOREHOLE TYPE 108mm Hollow Stem Augers COMPILED BY DKB
DATUM Geodetic DATE Feb.1/01 CHECKED BY ASP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.2	GROUND SURFACE										
0.0	Silty Sand with roots/organics Blackish brown Wet						314				
313.7 0.5	Biotite granite GNEISS Dark grey-black with white speckle 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 0.5m to 3.5m depth. For bedrock coring details refer to Record of Drillhole 3-3A						313 312 311				
310.7 3.5	END OF HOLE Note: Water used during coring operations; water level on completion of drilling not representative of groundwater conditions.										

ON_MOT_OLD 011-1104.GPJ ON_MOT.GDT 5/3/04

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

PROJECT: 011-1104

LOCATION: N 5005960; E 319503

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 3-3A

DRILLING DATE: February 1, 2001

DRILL RIG: CME 55 BOMBARDIER

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
				313.74			
0–3.5	Rotary Drilling NQ Size	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.		0.46	1	J,PR	
2					2	J,PR J,PR J,PR J,PR J,PR J,PR J,PR J,PR	
				310.70 3.50			
		END OF HOLE					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: PDG

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 3-4

METRIC

PROJECT 011-1104

W.P. 62-86-00

LOCATION N 5005917; E 319407

ORIGINATED BY SB

DIST 52 HWY 11

BOREHOLE TYPE 108mm Hollow Stem Augers

COMPILED BY DKB

DATUM Geodetic

DATE Jan.29/01

CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.2	GROUND SURFACE					310	
0.0	Sand and Gravel, trace silt (Fill) Brown Moist						
309.4 0.8	Silty Sand to Sand, some silt Compact to loose Brown becoming grey below 4.5m depth Moist to wet	1	SS	15	∇	309	
		2	SS	4		308	
		3	SS	4		307	
		4	SS	4			
		5	SS	4		306	
		6	SS	7		305	
304.1 6.1	Silty Clay, trace sand Stiff Grey Moist	7	SS	13		304	0 6 66 28
						303	
		8	SS	12			
302.0 8.2	END OF BOREHOLE Note: 1. Water level measured in open hole at 1.1m (El. 309.1m) upon completion of drilling.					302	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3-5

METRIC

PROJECT 011-1104

W.P. 62-86-00 | LOCATION N 5001898; E 319605 | ORIGINATED BY SB

DIST 52 HWY 11 | BOREHOLE TYPE 108mm Hollow Stem Augers | COMPILED BY DKB

DATUM Geodetic | DATE Feb.1/01 | CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.5	GROUND SURFACE										
0.0	Silty Sand with roots/organics Blackish brown Moist						314				
313.6 0.9	END OF BOREHOLE Refusal to further auger penetration; probable bedrock Borehole dry on completion of drilling.										

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

RECORD OF BOREHOLE No 3-6

METRIC

PROJECT 011-1104			
W.P. 62-86-00	LOCATION N 5005898; E 319374		ORIGINATED BY SB
DIST 52	HWY 11	BOREHOLE TYPE 108mm Hollow Stem Augers	COMPILED BY DKB
DATUM Geodetic	DATE Jan.29-30/01		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.2	GROUND SURFACE					
0.0	Ice				310	
0.2	Fibrous Peat Dark brown Wet					
309.1 1.1	Silty Sand to Sand, some silt Loose Brown becoming grey at 3.0m depth Wet	1	SS	3	309	
		2	SS	5	308	
		3	SS	6		
		4	SS	7	307	
306.4 3.8	Silt, trace sand and clay Loose to compact Grey Moist	5	SS	22	306	
	non-plastic Atterberg Limit result measured for Sample 6.	6	SS	9	305	0 6 84 10
		7	SS	10	304	
303.5 6.7	END OF BOREHOLE					

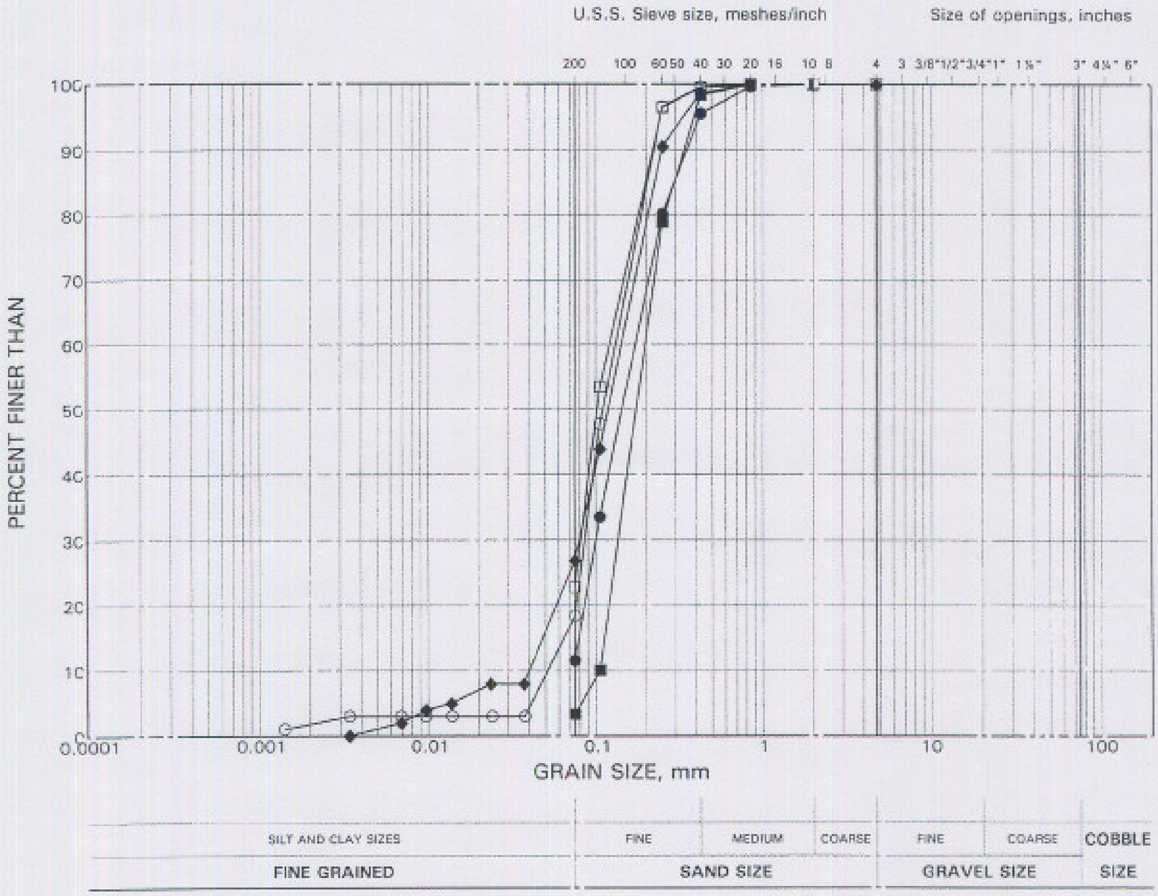
Chart column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ kN/m³.

Note:
1. Water level measured in piezometer at 1.7m (El. 308.5m) upon completion of installation.
2. Water level measured in piezometer at 0.1m depth (El. 310.1m) on February 28,2001.
3. Water level measured in piezometer at ground surface (El. 310.2m) on April 21,2001.

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

GRAIN SIZE DISTRIBUTION
Sand to Silty Sand

FIGURE 1



LEGEND

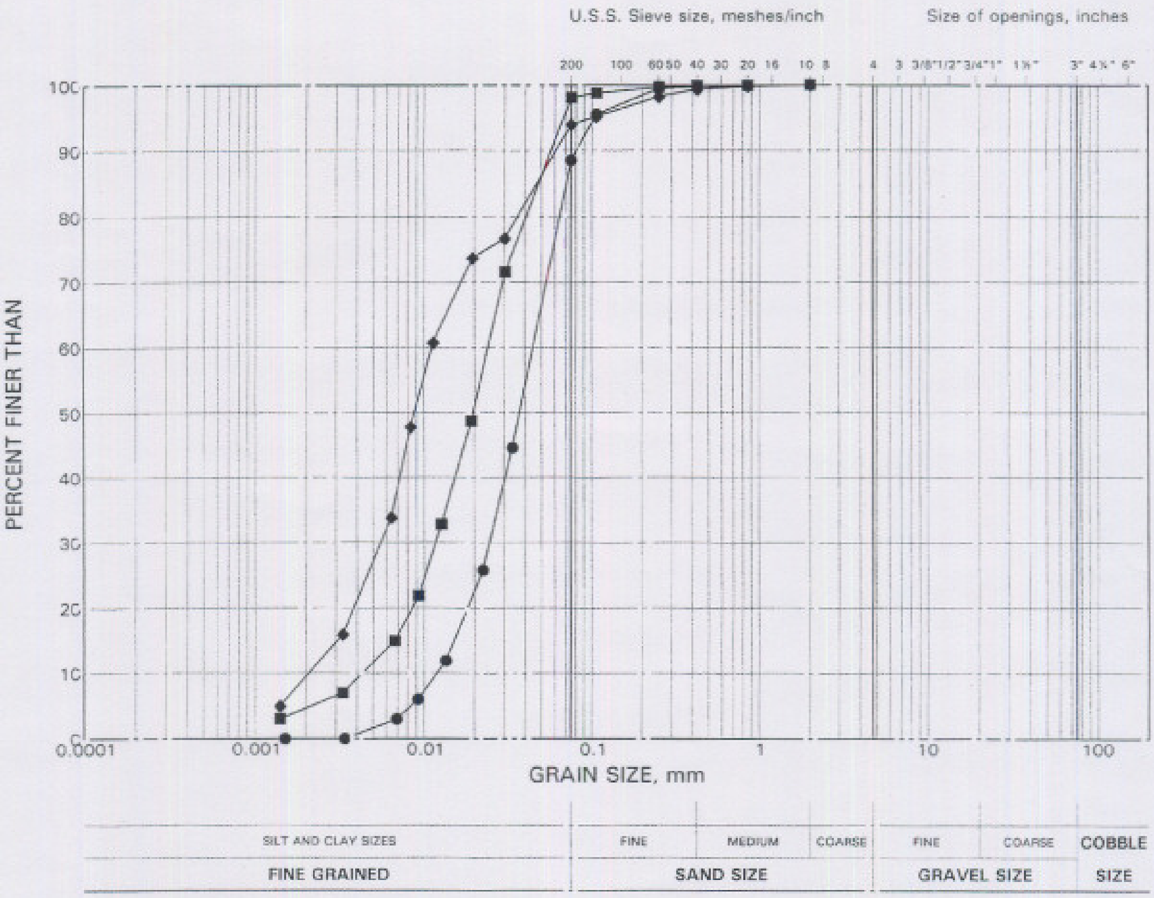
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1-8	3	318.6
■	2-2	5	310.2
◆	2-3	1	311.8
○	3-1	5	306.0
□	3-2	3	308.5

Project 011-1104

Golder Associates

GRAIN SIZE DISTRIBUTION
Silt

FIGURE 3



LEGEND

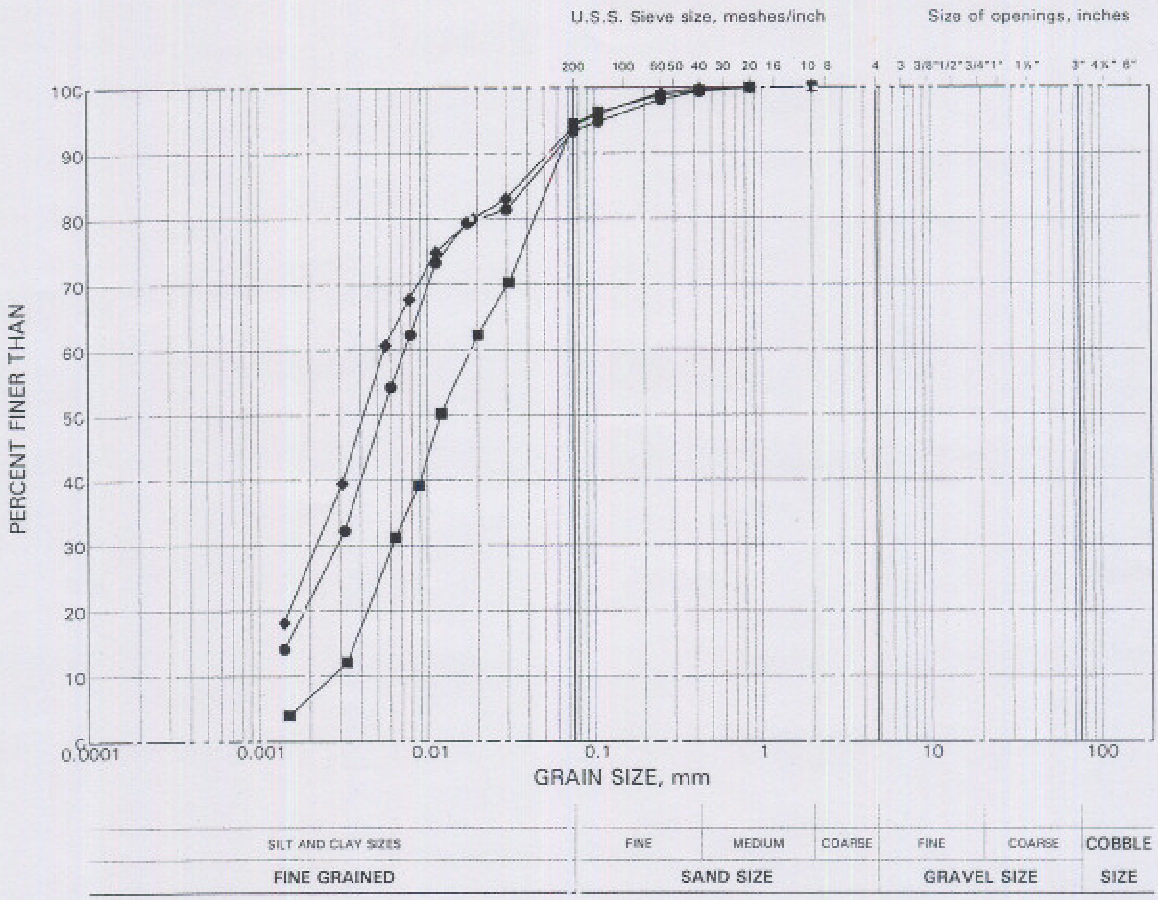
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	2-2	8	307.2
■	3-1	10	299.1
◆	3-6	6	305.0

Project 011-1104

Golder Associates

GRAIN SIZE DISTRIBUTION
Silty Clay to Clayey Silt

FIGURE 6

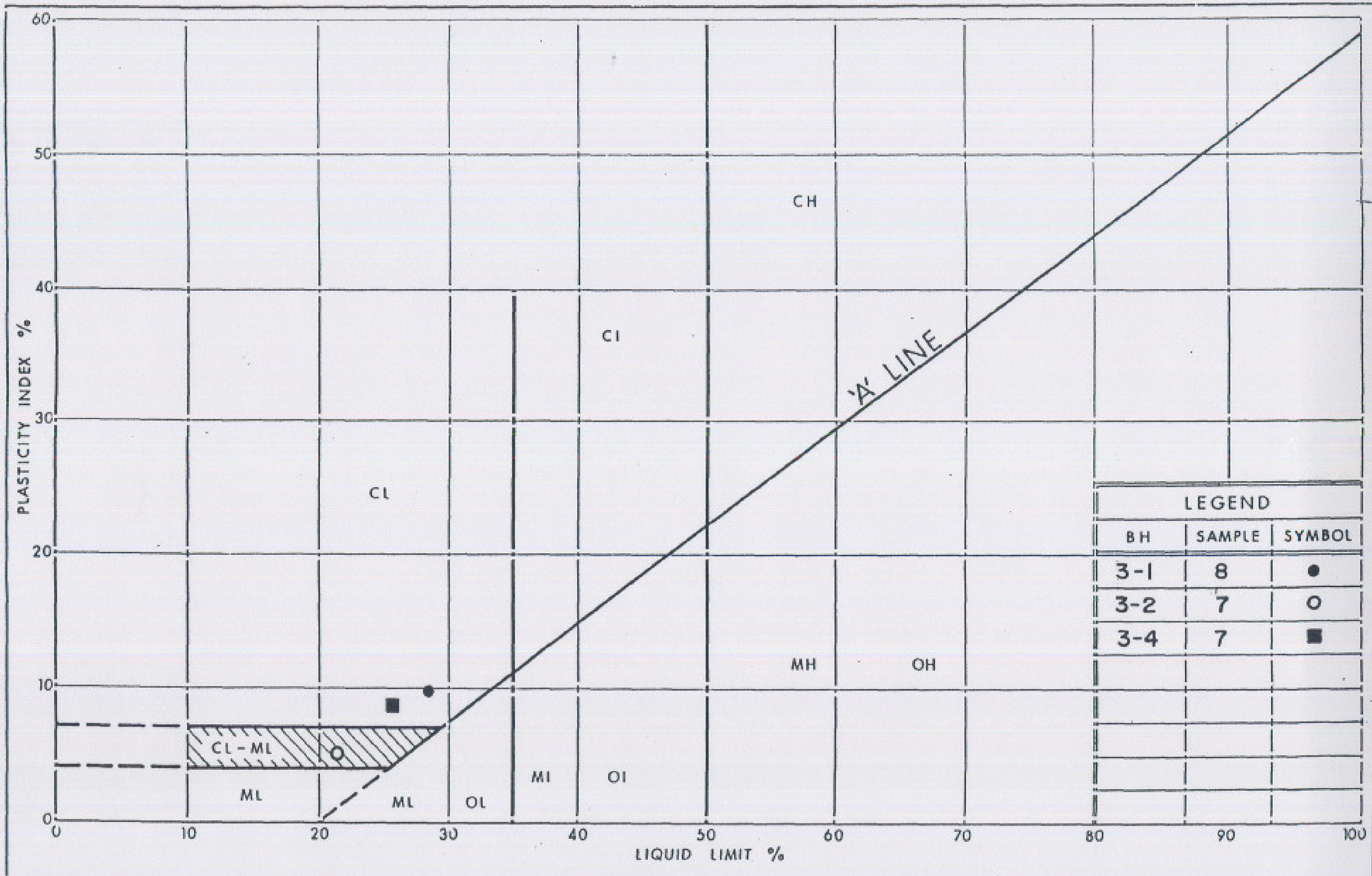


LEGEND

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

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation
Ontario

PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY

FIG No	7
W P	62-86-00

APPENDIX B

Rock Core Photographs

Plate 1. Biotite Gneiss Borehole 3-101, Rock Core 12 at west abutment (South Mary Lake Road)

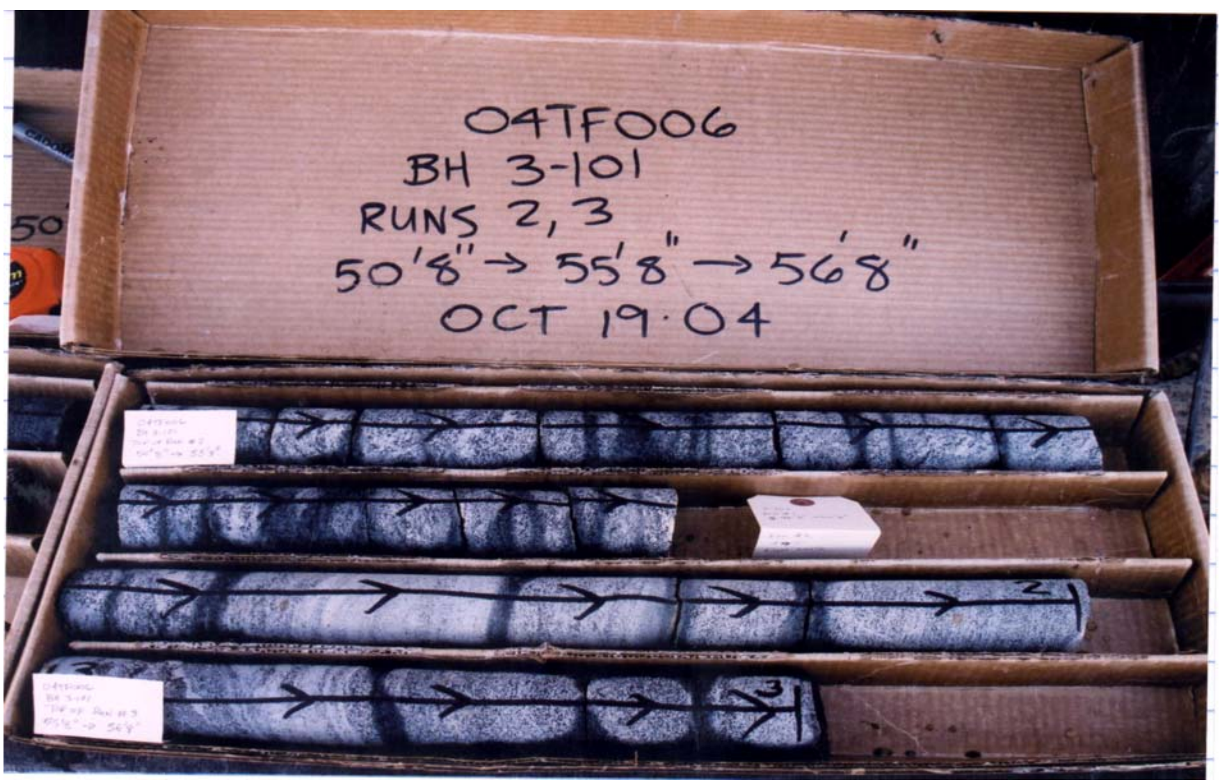


Plate 2. Biotite Gneiss Borehole 3-101, Rock Cores 13 and 14 at west abutment (South Mary Lake Road)

Plate 3. Granitic Gneiss Borehole 3-105, Rock Cores 8 and 9 at centre pier (South Mary Lake Road)

Plate 4. Granitic Gneiss Borehole 3-105, Rock Core 10 at Centre pier (South Mary Lake Road)

Plate 5. Granitic Gneiss Borehole 3-114, Rock Core 1 at east abutment (South Mary Lake Road)

Plate 6. Granitic Gneiss Borehole 3-114, Rock Cores 2 and 3, at east abutment (South Mary Lake Road)

APPENDIX C

SRS Rock Survey Report

(Geophysics GPR 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: T04670

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

RE: South Mary Lake Road Seismic Refraction Soundings

Geophysics GPR International Inc. was requested by Peto MacCallum Limited to perform seismic refraction soundings at South Mary Lake Road west abutment in the District of Muskoka, Ontario.

The surveys were performed November 4, 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 2 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.

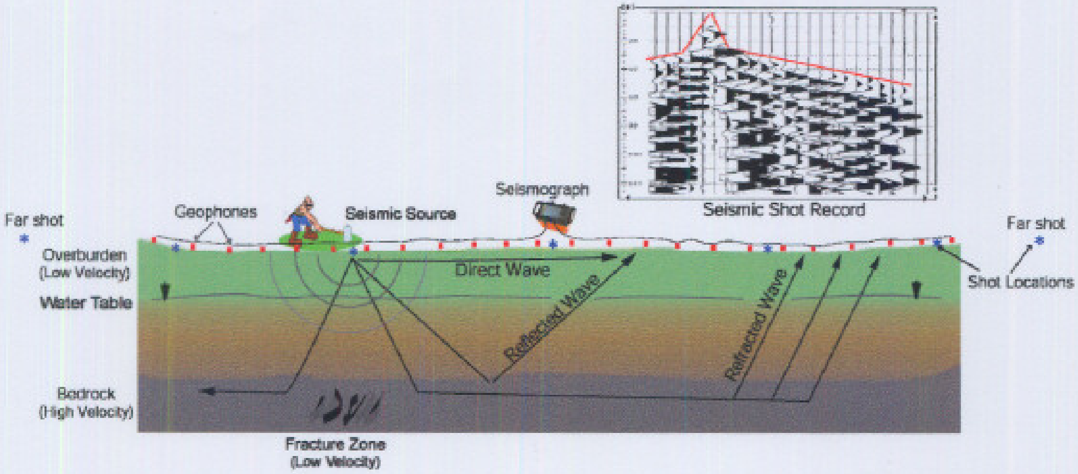


Figure 1: Seismic Method Operating Principle

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:

One seismic profile was 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.

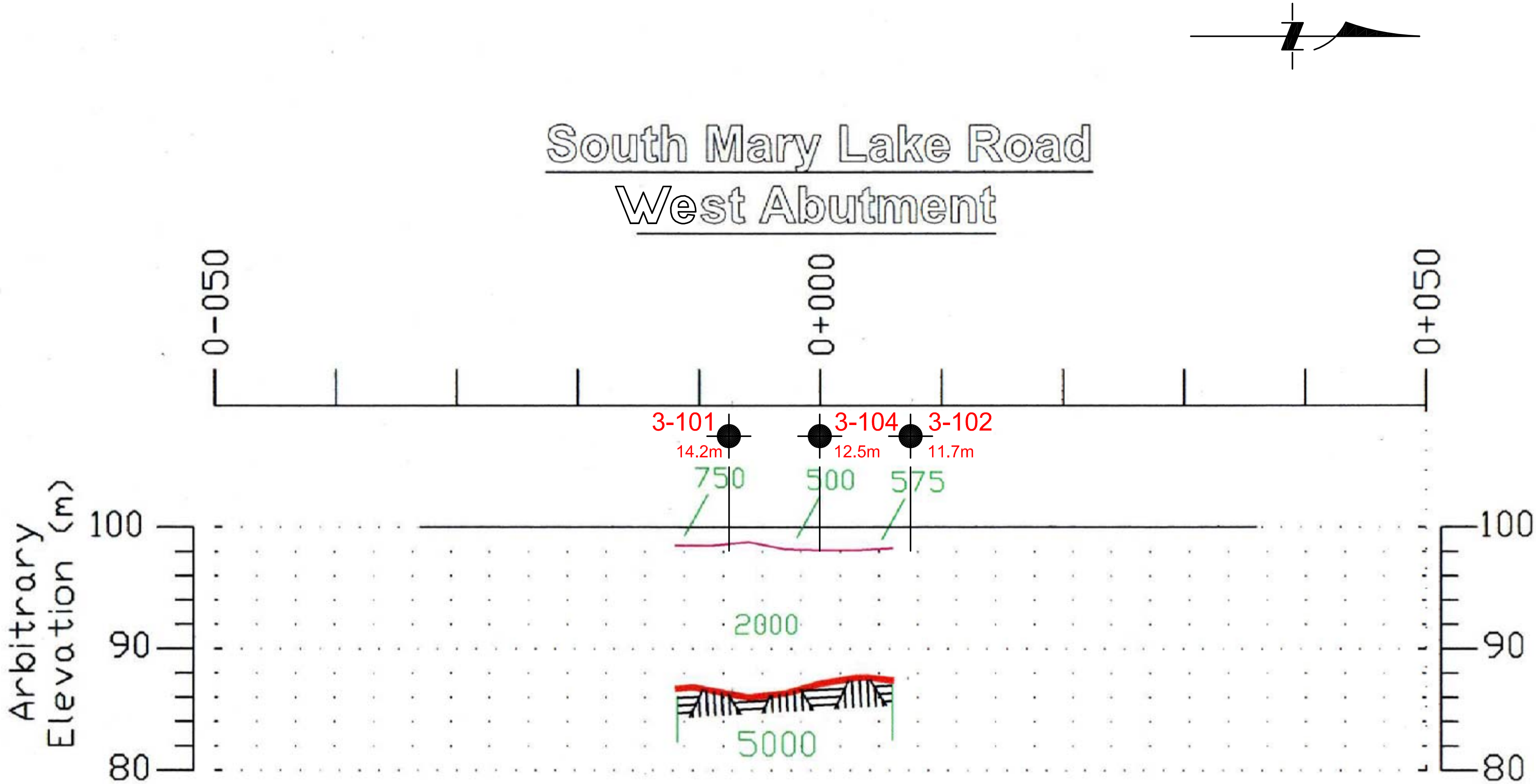
Ideally it is possible to determine the depth to bedrock beneath each of the 24 geophones; however, at this site this was not possible due to a combination of the strength of the energy source, the seismic noise from the highway and the depth of the rock.

A borehole was available at 0.0 where a depth to bedrock of 12.5 meters was measured. The original interpretation identified two layers above the bedrock. There was a suspected third layer with a very high velocity directly over the bedrock. The estimated velocity of the hidden layer could be as high as 2,500 m/s, which is typical of basil till. The hidden layer is not indicated because the exact thickness is not known. As a result the velocity of the second layer was increased from the measured value of 1525 m/s to 2000 m/s in order to adjust the bedrock depth to match the borehole.

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

Sincerely,

Milan Situm, P.Geo.
Manager



NOTES:

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

FOUNDATION DESIGN REPORT

for

SOUTH MARY LAKE ROAD UNDERPASS
WP 62-86-02, SITE 42-193
HIGHWAY 11
TOWNSHIP OF STEPHENSON
DISTRICT 52, HUNTSVILLE

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
3 cc: McCormick Rankin Corporation for distribution to MTO, North Bay + 1 digital copy
1 cc: McCormick Rankin Corporation for distribution to MTO, Downsview + 1 digital copy
2 cc: McCormick Rankin Corporation + 1 digital copy
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref.: 04TF006
Index No.: 073FDR
Geocres No.: 31E-228
May 17, 2005

TABLE OF CONTENTS

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
for
South Mary Lake Road Underpass
WP 62-86-02, Site 42-193
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 South Mary Lake Road and Highway 11 located some 14 km south of Huntsville, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

The South Mary Lake Road alignment passes over Highway 11 at Station 14+079.36, Highway 11 Chainage, in the Township of Stephenson (ref. Drawing 1 "South Mary Lake Road 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 South Mary Lake Road/Highway 11 underpass comprises a two-span structure with two equal spans of 45 m for a total length of 90 m. The road grade at the west abutment of the new underpass is planned at elevation 319.5 and at elevation 321.5 the east abutment. Based on the existing ground levels, there will be about 9 m and 7 m of embankment fill placed at the west and east abutments, respectively. No fill or cut is anticipated at the centre pier foundation that is planned on the existing Highway 11 median centreline.

The subsurface stratigraphy varies from deep soil cover in excess of 9.5 m at the west approach embankment, west abutment and centre pier to shallow 0.0 to 2.5 m deep soil cover (typically less than 1.0 m deep), with bedrock outcrops exposures and boulders at the east abutment and approach embankment.

The stratigraphy at the west approach embankment, west abutment and centre pier foundations generally comprises localized fill or peat deposits covering typically loose to compact sandy silt/

165 Cartwright Avenue, Toronto, Ontario M6A 1V5
Tel: (416) 785-5110 Fax: (416) 785-5120
E-mail: toronto@petomaccallum.com
BARRIE, BRAMPTON, HAMILTON, KITCHENER, TORONTO

silty sand interbedded with stiff to very stiff clayey silt layers and compact to dense silt/sandy silt mantling bedrock. The clayey silt layers are 1.9 to 4.2 m thick at the west abutment and 2.9 m thick at the centre pier. (The clayey silt was classified as silty clay in the previous GAL boreholes.)

The stratigraphy at the east approach abutment boreholes comprises topsoil and loose sand with varying amounts of silt over bedrock and exposed bedrock outcrops. The soil cover contains an isolated large 900 mm thick boulder at the surface near the centre of the abutment area.

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	3-103	> 6.7	< 303.7
	3-4	> 8.2	< 302.0
West Abutment	3-101	14.2*	296.5
	3-102	11.7*	298.8
	3-104	12.5	298.4
	3-1	14.2*	296.2
Centre Pier	3-105	10.9*	300.8
	3-106	9.5*	301.9
	3-2	10.1*	301.3
East Abutment	3-107	0.0**	313.6
	3-108	0.9	313.2
	3-109	1.2*	312.3
	3-110	1.7*	312.8
	3-111	0.6	313.0
	3-112	0.8	313.8
	3-113	0.0	314.2
	3-114	2.5*	311.9
	3-115	1.1	313.3
	3-3	0.5*	313.7
	3-5	0.9	313.6
East Approach	3-116	0.0	316.6

* - Confirmed by rock core   ** - Boulders at surface

2. FOUNDATIONS

2.1 General

2.1.1 West Abutment and Centre Pier

The soil/bedrock interface at the boreholes drilled for the west abutment is 11.7 to 14.2 m below grade and 9.5 to 10.9 m at the centre pier of the proposed bridge structure. The relative density/consistency of the soils at the west abutment and centre pier is typically loose to compact/stiff to very stiff therefore the soils are not considered suitable to support these structure foundations on spread footings. It is recommended to support the west abutment and centre pier on piles driven to bedrock. The presence of boulders within the soil immediately above the bedrock should be noted since the boulders may damage the piles and/or cause the piles to reach false refusal. Drilled caissons designed to support the foundations on rock is not considered a feasible alternative, due to the presence of boulders immediately above the bedrock and the wet condition of the native soils that would cause difficulties during construction.

2.1.2 East Abutment

The soil/bedrock interface at the boreholes drilled for the east abutment of the proposed bridge structure is 0.0 to 2.5 m below existing grade. Foundation alternatives for the abutment consist of spread footings constructed on bedrock or engineered fill. All boulders such as the 900 mm thick boulder that was encountered at the centre of the east abutment should be removed from the footing location.

Use of steel H-piles to support the east abutment foundation loads will be dictated by structural design considerations. The feasibility of employing integral abutments supported on steel H-piles will also be subject to structural design considerations and require excavation of a trench in the bedrock to accommodate the minimum pile length required for integral abutments as indicated in the MTO Report Ref. No. SO-96-01.

2.1.3 Comparison of Foundation Alternatives

Footings bearing on bedrock or structural fill and piles driven to bedrock were considered to be feasible for this project. Caisson foundations were not considered feasible or practical. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives is summarized on the following table:

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	APPLICABLE FOUNDATION ELEMENT
Spread Footings on Rock	• Ease of construction • Lower cost than deep foundations • High bearing resistance	• Need to remove rock to foundation elevation • Needs mass concrete to achieve a level bearing surface	East Abutment
Spread Footings on Structural Fill	• Ease of construction • Lower cost than pile foundation • Level bearing surface • No need for rock excavation	• Requires the construction of a structure fill pad • Low bearing resistances • Needs erosion protection	East Abutment *
Piles on Bedrock	• Technique and specialized construction equipment to drive piles is available in the industry • High bearing resistance	• Requires construction of a fill pad ahead of the approach embankment construction • Higher installation cost than spread footings	West Abutment * And Centre Pier * East abutment (in rock trench)

Note: * Preferred foundation type for structure foundation element.

Conventional, semi-integral and integral abutments are considered feasible at this site, based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundation engineering perspective, use of piles driven to bedrock is the preferred type of foundation for the west abutment and center pier; spread footings constructed on a pad of structural fill is the preferred foundation type for the east 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 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 east abutment of the underpass structure on conventional spread footings founded on either bedrock or structural fill placed directly on bedrock is considered to be feasible.

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

2.2.2 Footings Constructed on Bedrock

Footings construction on the existing bedrock surface at the east abutment should be designed using a factored bearing resistance at ULS of 8,000 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. It is considered that a reduction of the bearing resistance for inclined loads in accordance with the requirements of clause 6.7.4 of the CHBDC is not required in view of the tight partings of the rock encountered in the boreholes.

The anticipated depths/elevations to bedrock at the east abutment are indicated in the table provided in a previous section of this report. The bedrock surface ranges from elevations 311.9 to 314.2 at the east abutment. Mass concrete could be placed to provide a level founding surface for the footings. Alternatively, or the rock surface could be “stepped” in the bedrock surface thereby creating a level founding subgrade by a combination of rock excavation and placement of mass concrete. The width of the "steps" should be greater than or equal to the height of the "steps".

Mass concrete could also be employed to raise the subgrade to the design level of the footings. The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use of high strength concrete to prevent overstressing will be dictated by the actual thickness of the mass concrete and structural design considerations.

Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 35 MPa.

Comments concerning excavation of bedrock to enable construction of the footings are provided in subsequent sections of the report.

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 added in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. A NSSP should be prepared for the shear key. Fractured rock should be removed from these areas.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC. If anchors are installed, a factored bond stress at the rod/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The minimum recommended bond length is 3.0 m according to clause 6.10.2.3 of the CHBDC. A factored rock/grout bond stress of 400 kPa should be employed for the first metre of embedment since poor to fair quality of rock was identified in this zone and 800 kPa for the remaining embedment length.

2.2.3 Footings Constructed on Structural Fill

Footings constructed on structural fill placed on the west approach embankment are not recommended due to the underlying loose to compact and stiff to very stiff native soils. Footings constructed on structural fill could be employed to support the east abutment loads provided that all loose soil and boulders are removed and the structural fill pad is constructed on the exposed bedrock (elevation 311.9 to 313.8).

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 rock fill. However, rock fill 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 (bearing resistance independent of fill thickness due to shallow bedrock at this site) is as follows:

Factored Bearing Resistance at ULS	=	900 kPa
Bearing Resistance at SLS	=	350 kPa

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

The centre pier and west abutment should be supported on piles. Conventional or semi-integral abutment design is anticipated if the east abutment is founded on spread footings.

To allow an alternative integral abutment design for the bridge structure, the east abutment should also be supported on piles installed in a trench excavated/blasted into the existing rock outcrop.

The general pile foundation design recommendations are provided on the following paragraphs followed by additional recommendations for integral abutment foundations.

Piles for the centre pier and west abutment should be driven to refusal on bedrock at the estimated range of reference founding levels that are provided on the following table:

LOCATION	DEPTH TO ROCK (m)	PILE FOUNDING ELEVATION	RELEVANT BOREHOLES
West Abutment	11.7 to 14.2	296.2 to 298.8	3-101, 3-102, 3-104, 3-1
Centre Pier	9.5 to 10.9	300.8 to 301.9	3-105, 3-106, 3-2

The recommended factored axial resistance at ultimate limit states (ULS) for the four pile sections noted is considered to be appropriate if the installation proceeds **after** the full consolidation of the native soils has occurred.

FACTORED AXIAL RESISTANCE AT ULS, kN

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 stress imposed on the sand, clayey silt and underlying silt with clayey layers by the approximate 9 m high embankment near the east abutment will result in some "consolidation" of the soils that surrounds the piles since the sand is loose, the liquidity index of the clayey silt is 0.8 and clayey layer exist in the underlying silt. While consolidation of the sand will occur rapidly as the fill is placed, it is anticipated that primary consolidation of the clayey silt will be completed in 5 months. Since it is likely that some fill will be placed after the piles are driven and the magnitude of movement required to mobilize the negative skin friction is in the order of 5 mm, the axial resistance of the piles used for design should be reduced to account for the downdrag force.

The downdrag force developed on the piles due to negative skin friction from consolidation of the native clayey silt and overlying cohesionless soils under the embankment loading at the west abutment is computed as follows:

DOWNDRAG FORCE, kN	
HP 310 x 110	260
HP 310 x 152	270
HP 360 x 108	300
HP 360 x 152	310

Refer to Section 4.3 for a discussion and recommendations on the treatment of approach embankment settlements.

The presence of cobbles/boulders was identified above bedrock at depth in boreholes 3-104 and 3-105. Since these deposits appear to be typically compact, the risk of damage during driving is considered to be low and, as a consequence, application of a reduction factor is not employed. Nevertheless, a NSSP should be prepared to advise the contractor of the presence of boulders 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 include specific direction for the contractor to provide experienced full time foundation engineering supervision to monitor the driving operations over the complete length of the pile during driving below elevation 300 (west abutment) and elevation 303 (central 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 over the total length of the pile below elevation 300 or 303 as applicable 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 and the need to drive replacement piles if evidence of damage is detected.

The compacted granular fill pad placed as a working platform for construction equipment during installation of the abutment piles should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular 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 in which cobbles and/or boulders were identified. 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 piles will set on or into bedrock and should be equipped with "Rock Points". Clause 3.1.2 and 3.3.1-6 of the Structural Manual (Division 1 - Exceptions to the Canadian Highway Bridge Design Code) and SP 903S01 call for the use of Oslo Point (OPSD 3304) or Titus H Bearing Pile Points Rock Injector Model (Titus Point) on piles driven to bedrock. The Titus Point should be used at this site since the slope of the bedrock revealed at the borehole locations is about 14^{o} and to minimize the potential for damage to the pile toe when driving through cobbles/boulders overlying bedrock at the site.

2.3.1 Integral Abutments on Piles

The bedrock surface level at the east abutment ranges from elevations 311.9 to 314.2, that is 7.4 to 9.7 m below the proposed top of pavement elevation 321.6.

The depth of excavation of a trench into rock to accommodate the use of integral abutments will be dictated by structural design details. The excavation width should be at least 1 m wider than the plan area of the piles; side slopes in the rock could be excavated near vertical. The excavation should be backfilled with Granular A, following the procedures outlined in the section titled "Approach Embankments". Further comments concerning bedrock excavation are provided in the section titled "Excavation and Groundwater Control".

Placement of concrete in the trench could also be employed to provide temporary support to the piles during construction in the alternative pile installation scheme shown on the attached Figure 2. The piles must however bear on the bedrock surface directly or as designed by the structural engineer.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

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. These soils are interbedded with stiff/very stiff clayey silt layers at the west abutment.

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

	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 assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended in the Section 4. If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexture, the coefficient of horizontal subgrade reaction, k_s (MN/m^3) should be computed using the following equations (Canadian Foundation Engineering Manual, 3rd Edition):

a) Cohesionless Soils

k_s = $n_h\ z/b$

where n_h = coefficient related to soil density

= 10.0 MN/m^3 for granular backfill

= 2.0 MN/m^3 for native silty/sandy soils above groundwater table (elevation 310.0)

= 1.3 MN/m^3 for native silty/sandy soils below elevation 310.0

z = depth, m

b = pile width, m

b) Cohesive Soils (between elevations 301.5 and 305.5)

k_s = $67\ \tau_u/d$

where τ_u = Undrained shear strength of clayey silt, use 0.15 MPa

d = Pile diameter or width, m

For design purposes, the groundwater should be considered at 1.0 m depth below existing grade at both east and west abutments.

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

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.

$$p = K(\gamma h + q) + Cp + Cs$$

where K = coefficient of lateral earth pressure (dimensionless)
γ = unit weight of free-draining granular material, kN/m^3
h = depth below final grade, m
q = surcharge load, kPa, if present.
Cp = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = angle of internal friction of retained soil (35^o for Granular A or Granular B Type II)
δ = angle of friction between the soil and wall (23.5^o 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 rock fill should be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETERS	GRANULAR A	GRANULAR B TYPE II	ROCK FILL
Internal Friction Angle, Ø (degrees)	35	35	42
Unit weight, γ (kN/m^3)	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

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral 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^o 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 these computations. The backfill should be considered as medium dense sand for this 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.

3.2 Retained Soil System Wall

A retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 4.3 Approach Embankments Settlements 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 wall footings and should be removed from the RSS wall footprint together with all soil containing organic materials.

The bearing resistances recommended previously for footings founded on bedrock (at levels ranging from elevations 311.9 to 314.2) or structural fill constructed on bedrock at the east abutment are considered to be suitable for the RSS wall footings. The anticipated width of the RSS footing is 600 mm.

The RSS wall footings for the west abutment should be placed on a minimum 1.0 m thick engineered fill pad founded below the organic and very loose upper zones of the subgrade. Based on the borehole data, the engineered fill pad should be founded at elevations 309.6. The recommended bearing resistance for the west abutment RSS wall footings (maximum 0.6 m wide) constructed on a minimum 1 m thick engineered fill placed on the native soil is as follows:

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

The earth pressure coefficients provided previously 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 is considered to be appropriate for both situations at this site.

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, 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 the design of the structure (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. Rockfill is recommended at the west abutment in view of the embankment fill requirements for the road and intersection ramp construction through the adjacent swamp. The east and west approach fill embankments will be about 7 and 9 m high, respectively near the structure and taper down away from the structure.

The 200 to 800 mm thick layer of peat identified in boreholes 3-101, 3-102, 3-103 (west approach) and the 100 to 400 mm layer of peat and topsoil in boreholes 3-108 to 3-112 and 3-115 (east approach) should be stripped prior to placement of the embankment fill. Construction of the fill on the native sandy soils and/or bedrock that underlie surficial deposits of peat or topsoil is considered feasible.

It is considered that the approach embankments constructed in accordance with the recommendations in this report will be stable with minimum factors of safety against overall global stability of 1.4 during construction and 1.5 post construction.

4.2 Embankment Design and Construction Considerations

The width of the embankment platforms should be widened by a minimum of 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 settlements of the embankment 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 rock fill. The 2 m wide mid-height berm called for in the OPSD 202.010 should be incorporated for earth embankments higher than 8 m and rockfill embankments higher than 10 m. Where the embankment is constructed with earth fill placed over a rock fill base, the 8 m height limit should govern.

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 3, appended. OPSS Granular B Type II should be used for the drainage gaps.

4.3 Approach Embankment Settlements

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

Settlement under the east 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 maximum computed settlement under the 9 m high west embankment fill due to "consolidation" of the underlying native sandy/silty soil and clayey silt deposits is 100 to 150 mm. Consolidation of the cohesionless soils (75 to 100 mm) should be completed within one month of the placement of the fill and the remaining 25 to 50 mm due to consolidation of the clayey silt completed within five months following placement of the fill.

Where 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 40 and 30 mm at the west and east approaches, respectively). About 50% of these settlements should occur during the initial 12 months following construction and 50% during the ten year period following completion of construction.

If the approach embankments are constructed with granular materials, some settlement of the road surface adjacent to the abutments should also be expected due to "consolidation" of the backfill. The granular backfill placed adjacent to the abutments will be about 9 and 7 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 and, if placed in 200 mm thick lifts compacted to 100% of the maximum dry density determined by the MTO test method LS-706 (standard Proctor) in accordance with the requirements of SP 206S03 and OPSS 501 (Method A) should be less than 20 mm. These estimated total settlements of the approach fill surface near the abutments should be 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 east abutment where a shallow soil cover was encountered. At the west abutment, where a layer of clayey silt up to 4 m thick was interbedded with cohesionless soils, placing a 2 m high surcharge of granular soils for a period of 3 months or a 3 m high surcharge for a period of 2 months 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 east abutment foundations if supported on spread footings founded on bedrock or engineered fill will extend through peat/topsoil, boulders and sandy soils to depths of up to 2.5 m. Excavation of the peat/topsoil and native soils is expected to be relatively straightforward. Large boulders should be expected at the site as encountered in borehole 3-114.

The soil at the abutment sites is classified as Type 3 soil above the water table according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes over the full depth of the excavation should be inclined at 45° to the horizontal.

Excavation of flatter side slopes mat be required if soft/wet materials or concentrated seepage zones are encountered locally.

At the west abutment and central pier, the soil below the water table (about 1.0 m) is classified as Type 4 soil. Side slopes should be cut at 3H:1V or suitable shoring used if space restrictions exist. If required, a suitable shoring protection scheme following SP 539S01 should be implemented. Since the anticipated maximum excavation depth will be 1.8 m (for frost protection of foundations) several protection scheme alternatives such as sheet piling, sheeting supported on rakers or bracing, cantilever soldier piles and lagging may be considered. The schemes should be designed for performance level 2 where the toe of slope of the Highway 11 embankment is more than 1.8 m away from the excavation. For smaller separation distances, a performance level 1 system such as soldier piles and lagging with anchored tie-backs is recommended to prevent movements of the existing embankments.

Alternatively, the groundwater level could be lowered in the surficial sand deposits locally for structure foundations 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 an elevation as high as possible with provision of insulation or fill to provide adequate frost protection.

Excavation of a bedrock trench at the east abutment will necessitate conventional rock excavation techniques such as blasting (OPSS 120) and jack-hammering. 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 excavate bedrock at the structure foundation.

Mechanical means should be employed to excavate the loosened rock at the footing. Mass concrete could be employed to level minor variations in the bedrock surface as indicated previously.

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 east abutment boreholes during or upon completion of drilling and the existing piezometer was dry. However, minor seepage should be anticipated locally at the soil/bedrock interface and within depressions in the bedrock surface. Groundwater levels are subject to seasonal fluctuations and rainfall patterns. It is anticipated therefore that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations.

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.

REGISTERED PROFESSIONAL ENGINEER
C. M. P. Nascimento
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
PROVINCE OF ONTARIO

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

CN/DWK:mi-lr

TABLE 1

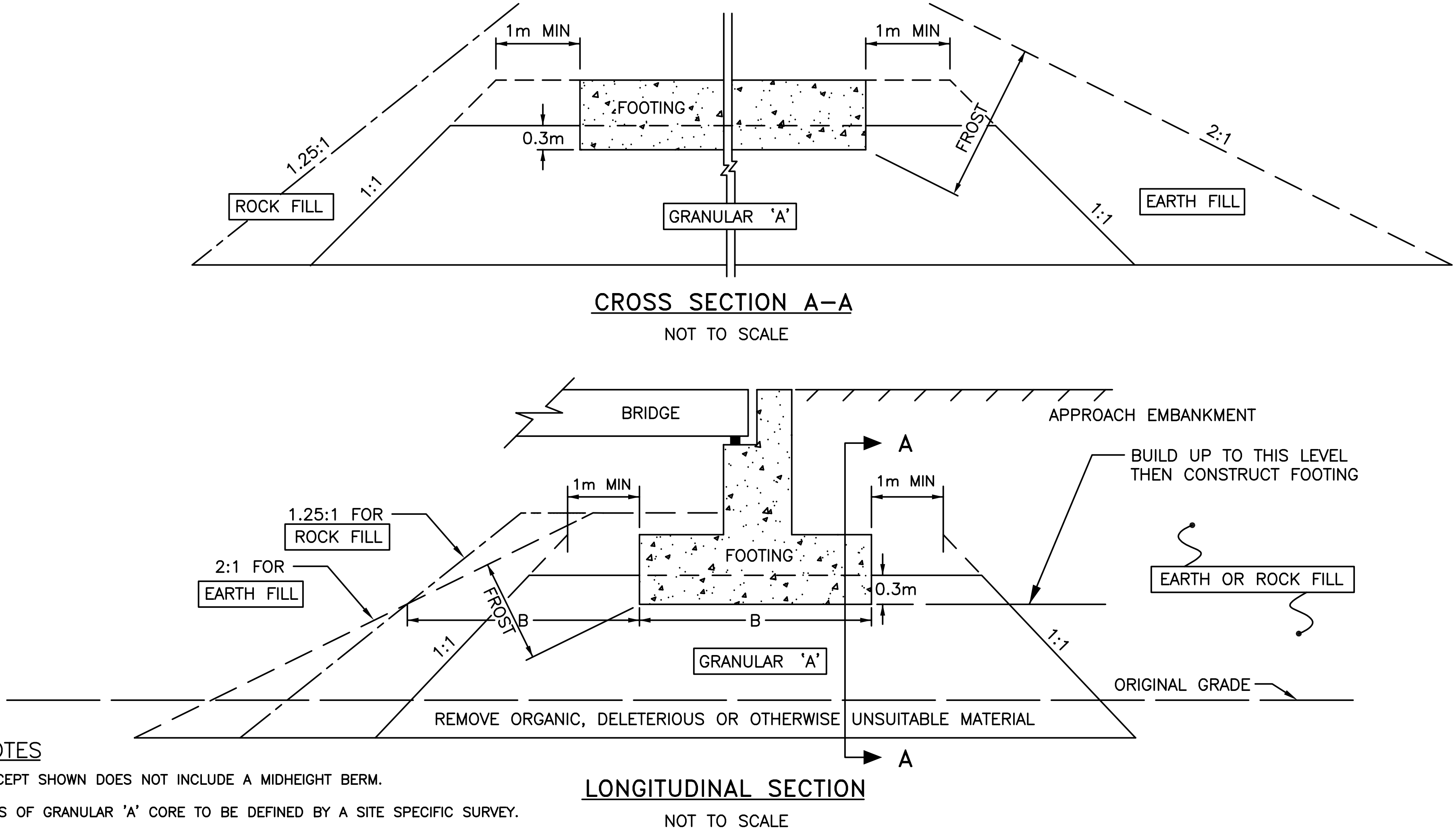
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

TITLE	NO.	DATE
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
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
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
Northeastern Region Directive - Platform Widening	NRE 98-200	October 28, 1998
Dowels Into Concrete	NSSP	December 2002

TABLE 2

Gradation Specification for Sand Fill in Pre-Augered Holes at Integral Abutments

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 μm	#30	80 – 100
425 μm	#40	40 – 80
250 μm	#60	5 – 25
150 μm	#100	0 – 6



NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

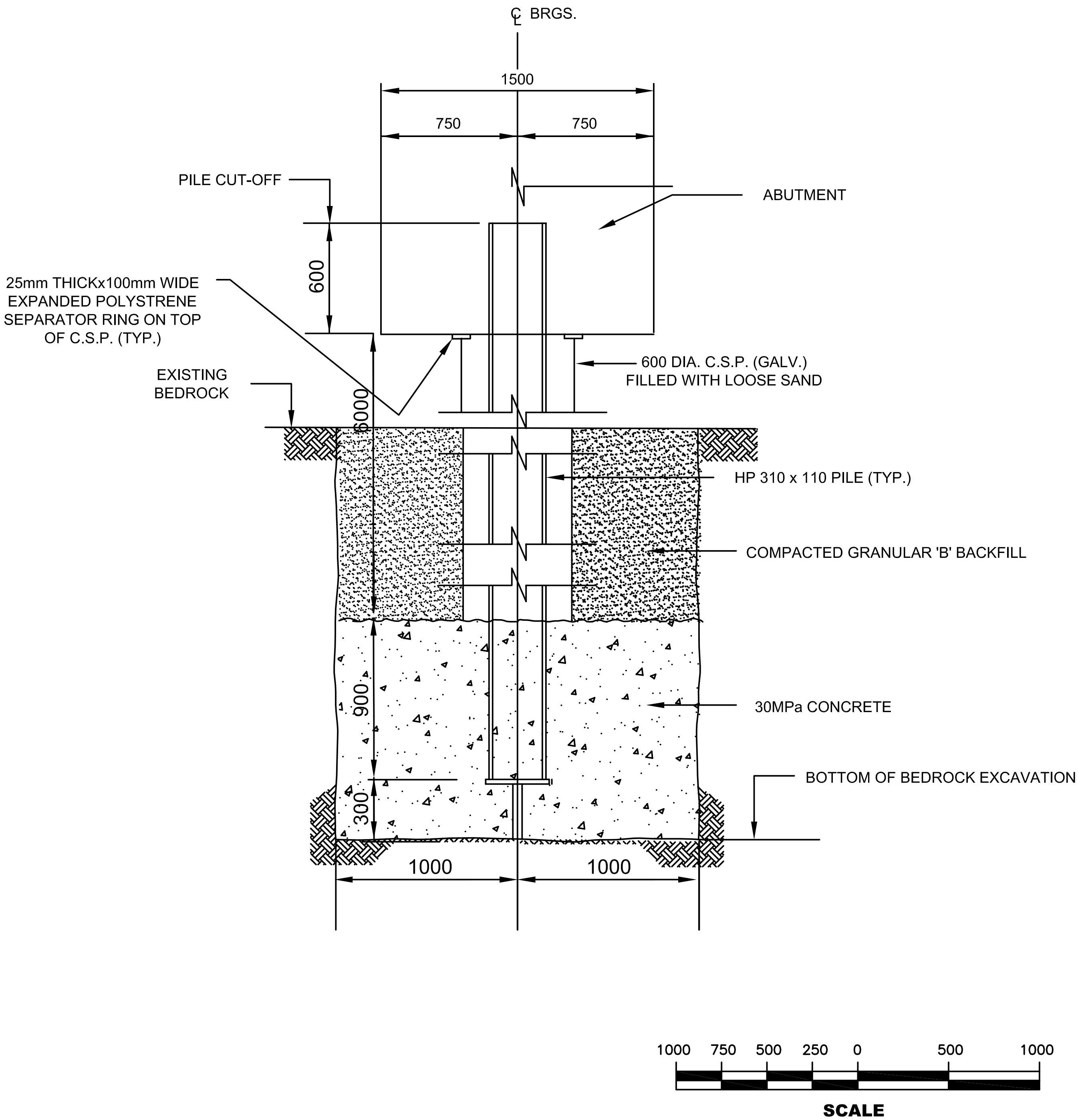
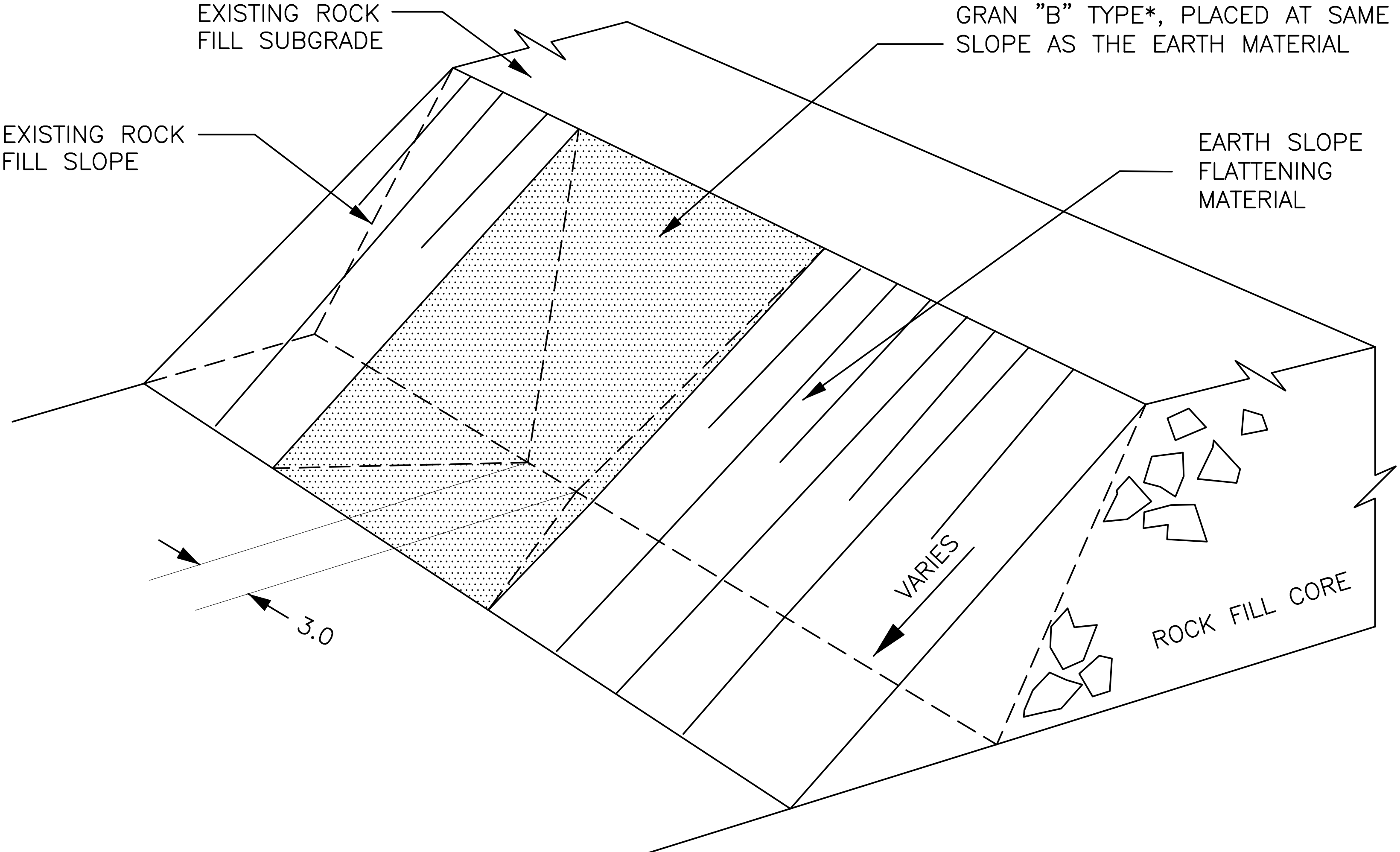


FIGURE 2: ALTERNATIVE PILE INSTALLATION SCHEME (INTEGRAL ABUTMENTS)

REFERENCE:
SKETCH PROVIDED BY McCORMICK RANKIN CORP. (MAY 2005)

SCALE: AS SHOWN
DIMENSIONS IN MILLIMETRES



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

FIGURE 3: ROCK FILL DRAINAGE IN SLOPE FLATTENED AREAS

NOT TO SCALE