

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 124 I/C UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
ONTARIO
G.W.P. 759-93-00, W.P. 5039-03-01, SITE 44-418**

Geocres Number: 31E-201

Report to

Marshall Macklin Monaghan

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

AEG/C:\Proj\19\1423\12 Hwy 11\Bridges\418 Hwy 124\Hwy 124 FIDR 2nd
DRAFT.doc

April 20, 2004
File: 19-1423-12

TABLE OF CONTENTS

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
3.1	General.....	2
3.2	Drilling and Sampling.....	2
3.3	Installations and Backfilling	3
4	LABORATORY TESTING	4
5	DESCRIPTION OF SUBSURFACE CONDITIONS	4
5.1	General.....	4
5.2	Topsoil	4
5.3	Sand	4
5.4	Silt.....	5
5.5	Clayey Silt	5
5.6	Sand and Gravel.....	6
5.7	Bedrock.....	6
5.8	Water Levels	7
6	INTRODUCTION	9
7	STRUCTURE FOUNDATIONS.....	10
7.1	Foundation Alternatives.....	10
7.2	Spread Footings on Native Soil	10
7.3	Spread Footings on Bedrock.....	10
7.4	Lateral Resistance on Bedrock	11
7.5	Spread Footings on Engineered Fill	11
7.6	Drilled Shafts (Caissons or Bored Piles)	12
7.7	Driven Piles	13
7.8	Recommended Foundation System	14

7.9	Downdrag	14
7.10	Pile Tips	14
7.11	Pile Installation	15
7.12	Pile Driving.....	15
7.13	Lateral Resistance of Piles.....	15
7.14	Frost Protection.....	16
7.15	Abutment Considerations	17
7.15.1	Retained Soil Systems	17
7.15.2	Integral and Conventional Abutments	17
7.16	Backfill to Abutments.....	18
8	EXCAVATION	18
9	APPROACH EMBANKMENTS	18
9.1	General.....	18
9.2	Stability Analysis.....	19
9.2.1	General.....	19
9.2.2	Stability analysis Results:	21
9.3	Settlement Analysis	23
9.3.1	General.....	23
9.3.2	One-Dimensional Consolidation Analysis - Methodology.....	23
9.3.3	One-dimensional secondary consolidation analysis - Methodology	24
9.3.4	Settlement Analysis - Results	24
9.4	Lateral Displacements at the Abutment Piles.....	25
10	STATIC EARTH PRESSURE	26
11	SEISMIC CONSIDERATIONS.....	27
11.1	Seismic Design Parameters.....	27
11.2	Embankment Stability and Spread.....	27
11.3	Retaining Wall Dynamic Earth Pressures.....	27
12	EMBANKMENT DESIGN ALTERNATIVES AND RECOMMENDATIONS...	28
12.1	General.....	28
12.2	West Approach Embankment	28
12.3	East Approach Embankment	29

12.4	Hwy 11 Embankments.....	29
12.5	Monitoring Program	31
13	CONSTRUCTION CONCERNS	31

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Embankment Stability Analysis
Appendix D	Foundation comparison
Appendix E	Special Provisions

Drawings

Appendix F	Borehole Locations and Soil Strata
	Soil Strata



FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 124 I/C UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
ONTARIO
G.W.P. 759-93-00, W.P. 5039-03-01, SITE 44-418
Geocres Number: 31E-201

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation at the Highway 124 Underpass structure over the proposed four-lane Highway 11 near Sundridge, Ontario. No preliminary foundations data was available for this site.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed through considering the data obtained in the course of the present investigation.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The site lies at the intersection of proposed alignments of Highway 11 and Highway 124 in Strong Township. The site lies to the north of existing Highway 124, to the west of existing Highway 11 and approximately 2 km southwest of Sundridge.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally, however, the land surrounding the site is low-lying and swampy. The immediate bridge site lies within a wooded area with the west abutment and approach lying in a swamp.

The site environs drain northward through a series of wetlands and small creeks. Local drainage is poorly developed and the groundwater table was found to be at or slightly above the ground surface.

Outside the wooded portions the area is generally farmland, mostly pasture. Development is scattered and there are no buildings within the immediate site area.

3 SITE INVESTIGATION AND FIELD TESTING

3.1 General

The site investigation and field testing for this project were carried out between November 12 and December 17, 2003. The site investigation consisted of drilling and sampling a total of eight sampled boreholes to depths ranging from 3.5m to 16.2m. Initially, a total of twenty potential exploration points were staked in the field but on the basis of the soil conditions encountered a total of eight sampled boreholes were drilled. The boreholes numbers and locations are shown on the attached Borehole Locations and Soil Strata Drawing at the end of this report and summarize in Table 3.1 below.

Table 3.1 – Borehole Locations and Elevations

Borehole	Northing (m)	Easting (m)	Elevation (m)
BH418-1A	5068807.0	311334.8	350.0
BH418-4	5068833.9	311386.9	351.6
BH418-5	5068808.4	311390.8	351.3
BH418-6A	5068800.8	311351.6	350.3
BH418-7	5068825.3	311349.4	350.1
BH418-15	5068817.8	311430.4	353.8
BH418-18	5068842.7	311421.8	353.7
BH418-20	5068834.9	311446.2	354.8

The borehole locations were surveyed by Marshall Macklin Monaghan Ltd. and Thurber obtained utility clearances prior to any drilling being carried out.

3.2 Drilling and Sampling

Some boreholes were drilled and sampled by All-Terrain Drilling Limited of Waterloo, Ontario using a CME 75 drill rig mounted on a Nodwell tracked carrier. The remaining boreholes were drilled and sampled by Malone's Soil Samples Co. Ltd. of Etobicoke,

Ontario using a Bombardier-mounted drill rig. Auger drilling and mud-rotary techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) and Shelby tube sampling. Insitu vane shear strength tests were conducted in cohesive soils.

Four of the boreholes were advanced into bedrock by diamond coring for the distances shown below:

Table 3.2 – Length of Rock Core

Borehole	Length of Core (m)
BH418-4	3.1
BH418-6A	3.0
BH418-7	1.1
BH418-15	3.0

A member of Thurber's technical staff supervised the drilling and sampling operations on a full-time basis. The supervisor logged the boreholes and the recovered samples and processed the samples for transport to Thurber's Oakville office.

3.3 Installations and Backfilling

One standpipe piezometer was installed in each of the foundation elements to allow monitoring of groundwater levels. The piezometers were installed in BH418-4, BH418-6A and BH418-15. The tip of the standpipe piezometer was surrounded with sand prior to backfilling with Benseal, bentonite and drill cuttings

Boreholes BH418-5, BH418-18 and BH418-20 were backfilled using Benseal grout and mixtures of bentonite and drill cuttings.

Artesian conditions were encountered in Boreholes BH418-1A, BH418-6A and BH418-7.

These boreholes were sealed using bentonite grout that successfully sealed off the artesian flow.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg limits. One oedometer test was carried out on a firm silty clay sample retrieved from BH418-1A beneath the west approach embankment.

Point Load tests were carried out on selected portions of the rock cores.

The test results are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are also presented on the “Borehole Locations and Soils Strata” and “Soil Strata” drawings inserted at the end of the report. A description of the stratigraphy is given in the following paragraphs.

5.2 Topsoil

Dark brown sandy topsoil was encountered in all boreholes to depth of 0.2m to 0.7m.

5.3 Sand

A layer of sand was encountered underlying the topsoil in all boreholes except in BH418-4 where it was absent. Where present in the boreholes the sand layer extended to depths ranging from 1.5m to 3.5m below ground surface. These depths corresponded to the underside of the sand lying at Elevation 347.7 at the west abutment, Elevation 348.6 at the pier and Elevation 351.5 at the east abutment.

The sand was brown to grey, with variable amounts of fines, ranging from 18% to 34%. The SPT blow count values typically ranged from 16 to 80, with most values in the range of 20 to 40, reflecting compact to very dense state. The moisture content ranged from 15% to 20%.

Grain size distributions for selected samples are shown in Figure B1 in Appendix B.

5.4 Silt

A layer of silt was encountered below the topsoil or below the sand layer to depths up to 4.1m below ground surface in the boreholes located at and west of the pier. The silt was classified as sandy to some sand and some to trace clay.

The underside of this silt stratum lies at elevation 346.0 at the west abutment and at Elevation 347.3 at the pier. It is absent at the east abutment.

Gradation tests carried out on selected soil samples showed sand content in the order of 13% to 22%, silt content ranging from 65% to 74% and clay content of 10% to 13%. Occasional sand layers were encountered in some of the boreholes. The thickness of the silt layer varied from 1.2m at BH418-1A to 3.9m at BH418-4. The SPT blow count values ranged from 10 to 36 with an average value of 21 indicating compact to dense conditions. The moisture content ranged from 15% to 27% with most values in the order of 20%.

Grain size distributions for selected samples are shown in Figure B2 in Appendix B.

5.5 Clayey Silt

With the exception of BH418-20, a silty clay to clayey silt deposit was encountered in all boreholes, underlying the silt or the sand deposits. The thickness of this deposit increases westwards being 1.1m to 1.5m thick at the east abutment and Pier locations and 5.1m to 7.3m thick beneath the West Abutment.

The underside of the silt and clay lies at Elevation 338.8 to 334.9 at the west abutment, Elevation 345.5 to 346.6 at the pier and at Elevation 350.0 at the east abutment.

Sand lenses typically 100mm thick were encountered in this deposit. Gradation tests carried out on selected soil samples showed sand content in the order of 3% to 7%, silt content ranging from 58% to 74% and clay content ranging from 21% to 38%. The clay content typically increased westwards. Water content values typically ranged from 25% to 40% with values below this range encountered in sand lenses. Atterberg Limit tests carried out on this material showed Plastic Limit values of 19% to 20% and Liquid Limit values of 23% to 25%.

SPT blow counts ranged from 1 to 8 beneath the west abutment and Pier and increased from 7 to 21 beneath the East Abutment. In Situ Vane tests indicated undrained shear strength values ranging from 35kPa to 120kPa beneath the west abutment and Pier. It is believed that the higher values of undrained shear strength are associated with the presence of sand layers. No vane tests were carried out east of the Pier due to the stiff consistency of the layer in that area.

Grain size distributions for selected samples are shown in Figure B3 and Atterberg limit results in Figures B4 through B6 in Appendix B.

5.6 Sand and Gravel

A layer of non-plastic soil grading from silty sand to sand and gravel was encountered beneath the silt and clay layer described above. This deposit was relatively thin in most boreholes, ranging from 0.1m to 1.7m in thickness, except in BH418-1A where it was 2.9m thick.

The underside of the sand and gravel coincided with the top of bedrock at Elevation 337.1 to 340.8 at the west abutment, Elevation 346.0 at the pier and Elevation 349.8 at the east pier.

The SPT blow count values ranged from 5 to more than 100. Cobbles were encountered at the base of this deposit in BH 418-6A. Local experience shows that a layer of very dense sand and gravel containing cobbles and boulders commonly mantles the bedrock.

The low SPT values are attributed to disturbance at the bottom of the borehole due to piping. The high SPT values are probably associated with the presence of cobbles and boulders commonly encountered on the bedrock surface.

5.7 Bedrock

The soils described above were underlain by bedrock consisting of pink with black bands granitic gneiss of the Central Gneiss Belt of the Pre-Cambrian Canadian Shield. The rock was generally fresh and the discontinuities were mostly sub-horizontal and slightly to medium rough. The bedrock was proved by coring in Boreholes BH418-4, BH418-6A, BH418-7 and BH418-15. The bedrock surface was inferred from refusal to auger penetration in other boreholes drilled at this site.

The bedrock is relatively shallow at the East Abutment (approximate depth of 4.0m – EL.349.6m to EL.349.8m) and it dips southeastward along the bridge alignment to depths of 9.3m (EL.340.8m) and 13.1m (EL.337.1m) in Boreholes BH418-7 and BH418-6A, respectively. The bedrock surface was encountered or inferred at the elevations shown in Table 5.1.

Table 5.1 – Ground Surface and Bedrock Elevations

Borehole	Ground Surface Elevation (m)	Bedrock Elevation (m)
BH418-1A	350.0	336.9 (**)
BH418-6A	350.2	337.1 (*)
BH418-7	350.1	340.8 (*)
BH418-4	351.6	346.6 (*)
BH418-5	351.3	344.6 (**)
BH418-15	353.8	349.8 (*)
BH418-18	353.7	349.6 (**)
BH418-20	354.8	351.3 (**)

(*) Top of bedrock proven by coring

(**) Top of bedrock inferred by auger refusal

Core recovery in the bedrock ranged from 77% to 100% and the RQD values ranged from 53% (near the surface) to 98% and for most part lay between 70 and 85%, indicating generally good quality rock.

The condition of the joints ranged from planar to uneven and were generally rough though some smooth, planar joints were noted. The joints were mostly tight with no infilling or secondary weathering material.

Strength values of the intact rock obtained from Point Load Tests in selected rock cores ranged from 112MPa to 205MPa, with most values larger than 150MPa. The rock is generally described as very strong. A summary of the Point Load Test results is presented in Table B1, Appendix B.

5.8 Water Levels

The groundwater level data recorded at this site is shown in Table 5.2.

Table 5.2 – Groundwater Level Measurements

Location	Borehole	Depth / Elevation of Groundwater Surface (m)	
		18/Nov/2003	18/Dec/2003
West Abutment	BH418-6A	-	-1.0 / EL.351.2 (*)
Pier	BH418-4	-	-0.91 / EL.352.5 (*)
East Abutment	BH418-15	0.99 / EL.352.8	1.21 / EL.352.6

(*) Negative depth refers to artesian conditions: water level in standpipe above ground surface. Water inside the standpipe was frozen.

Table 5.2 above shows that groundwater level in the standpipe piezometers installed at the west abutment and Pier locations was up to 1m above the ground surface. It should be noted, however, that the water was frozen in the standpipe when these measurements were taken. Notwithstanding this fact, the artesian groundwater flow encountered in BH418-7 when this borehole was advanced into bedrock confirms that small artesian conditions are present at this site.

At the East Abutment, the groundwater level was encountered 1.0m to 1.2m below the ground surface in BH418-15.

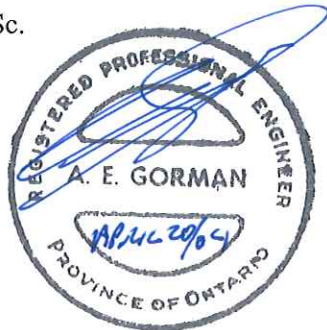
All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

Thurber Engineering Ltd.

Report prepared by:

Alastair E. Gorman, P.Eng., M.Sc.

Senior Foundations Engineer



Report reviewed by:

Paulo J. Branco, P.Eng., Ph.D.

Principal



FOUNDATION INVESTIGATION AND DESIGN REPORT**HIGHWAY 124 I/C UNDERPASS****HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER****ONTARIO****G.W.P. 759-93-00, W.P. 5039-03-01, SITE 44-418****Geocres Number: 31E-201****PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS****6 INTRODUCTION**

This report presents the interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

The proposed bridge will consist of a two-span (38:38) underpass structure with a total of three foundation elements: two abutments and one pier. The foundation elements will be skewed at 6° with respect to the centreline of the structure.

At the bridge site, the mainline of Highway 11 will run on an embankment approximately 3.5m to 6.5 m high under the NBL and SBL, respectively. Highway 124 will approach the site from higher ground to the west. At the west abutment, the finished grade will be Elevation 366.8 and the original ground lies at Elevation 350.1 ±, resulting in an approach fill approximately 16m to 17 m above original ground level or approximately 7 m to 8 m above the Highway 11 mainline.

At the east abutment, the finished grade will be Elevation 364.9 and the original ground lies at Elevation 353.8 ±, resulting in an approach fill approximately 11 m above original ground surface and 7.5 m above the Highway 11 mainline.

The ground at the west abutment and immediately to the west of the structure is wet and compressible soils underlie that portion of the site.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation and available from the preliminary investigation.

7 STRUCTURE FOUNDATIONS

7.1 Foundation Alternatives

Five foundation types have been considered:

- Spread footings on native soil
- Spread footings on bedrock
- Spread footings on engineered fill
- Drilled shafts (also referred to as caissons or bored piles)
- Driven piles

These foundations alternatives are discussed below.

7.2 Spread Footings on Native Soil

Spread footings bearing on the native soils are not considered to be suitable for the support of the foundations at this structure on account of the low values of geotechnical resistance that would be available and on account of the potential for immediate and long-term settlements.

The problems related to spread footings are compounded by the fact that the thickness of the compressible soils is not uniform along the length of the structure, being appreciable thicker at the west abutment than at the east. This would result in differential settlements that would impose further serviceability constraints on the structure.

7.3 Spread Footings on Bedrock

A spread footing founded on bedrock is considered feasible at the east abutment, where the bedrock is at approximately 4m depth. The footing may be designed on the basis of a concentric, vertical geotechnical resistance of 10,000kPa at factored ULS. The SLS condition will not govern on the bedrock. The established bedrock elevations at the east abutment are shown in Table 7.1.

Table 7.1 – Bedrock Elevations

Borehole	Bedrock Elevation
418-15	349.8
418-18	349.6

In practical terms, the underside of the concrete footing should be designed to lie approximately 200 mm above the local top of rock and the difference made up using mass concrete fill. Typically, the footing may be stepped down across the width of the structure to accommodate changes in the elevation of the top of bedrock.

The top surface of the bedrock should be stripped of all overburden and be cleaned. All shattered and loosened rock fragments should be removed from the footprint of the footing or mass concrete fill.

This option is not considered feasible at the Pier and west abutment, where the bedrock lies up to 7m and 12m, respectively, below ground surface.

The above resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clauses 6.7.3 and 6.7.4.

This option will involve excavating through the layer of fine grained sand below the groundwater table. Prior dewatering will be required to prevent the soil flowing into the excavation and to achieve a sufficiently dry base on which to construct the footing. Dewatering the silty, fine grained sand will be difficult and it may be necessary to install a sheet pile cut-off around the perimeter of the excavation.

7.4 Lateral Resistance on Bedrock

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors should be included in the design.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock is exceeded. Using lower bound values for the strength of the rock, an ultimate horizontal resistance of 2.6 MN may be assumed for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing.

The shearing resistance of the selected dowel must be checked structurally.

7.5 Spread Footings on Engineered Fill

The use of spread footings bearing on engineered fill pads is considered to be feasible at the east abutment provided all overburden is removed and the engineered fill is constructed on the bedrock.

Given the soils shown to exist at the site, footings on engineered fill pads bearing on the native soil are not recommended due to the possibility of unacceptably large settlements. The depths of overburden soil at the pier and west abutment make founding the engineered fill on bedrock impracticable and the use of spread footing in these elements is not recommended.

If a footing on engineered fill bearing on bedrock is used, it may be designed on the basis of the following concentric, vertical geotechnical resistances:

- 900kPa at factored ULS
- 400kPa at SLS

The engineered fill must be founded on the bedrock at the elevations given in Table 7.1 and a minimum thickness of 1.0 m of engineered fill must be maintained between the underside of the concrete and the top of the bedrock.

The engineered fill must consist of OPSS Granular “A” or Granular “B” Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content and generally conforming to the geometry illustrated in Figure 1.

The resistance values above are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 15 mm across the width of the structure.

The sliding resistance of mass concrete poured on a compacted Granular “A” pad may be computed on the basis of an ultimate friction factor of 0.7.

This option will involve excavating through the layer of fine grained sand below the groundwater table. Prior dewatering will be required to prevent the soil flowing into the excavation and to achieve a sufficiently dry base on which to construct the footing. Dewatering the silty, fine grained sand will be difficult and it may be necessary to install a sheet pile cut-off around the perimeter of the excavation.

7.6 Drilled Shafts (Caissons or Bored Piles)

The foundations may also be supported on drilled shafts founded in the bedrock.

Due to the relatively high groundwater table and presence of a layer of water bearing sand and gravel immediately above the bedrock, it will be necessary to advance a temporary liner into the top of the bedrock to exclude soil from the drilled shaft. In addition, a qualified geotechnical engineer or technician should visually inspect the top of bedrock to

assess the bedrock quality and resistance. Hence, a temporary liner and dewatering will be required for quality control.

Drilled shafts may be designed on the basis of a concentric, vertical geotechnical resistance of 10,000 kPa at factored ULS. The SLS condition will not govern on the bedrock. The caisson should be founded on fresh, intact bedrock, which is anticipated to be encountered at or within 0.5m of the bedrock surface.

The caissons must be installed in accordance with 903SP01.

As it is not considered practical to install caissons on a batter, lateral loads must be resisted by earth pressure as described in Section 7.13 or by socketing the caissons into bedrock.

7.7 Driven Piles

The stratigraphy encountered at this site is considered to be suitable for the use of steel piles driven to bedrock to support the foundations. The use of H-section piles is recommended and the following concentric, vertical, geotechnical resistances (factored ULS) will be available for the pile sections shown in Table 7.2.

Table 7.2 – Pile Resistances

Pile Section	ULS _f
HP 310 X 110	2,000 kN
HP 310 X 132	2,400 kN
HP 310 X 152	2,750 kN
HP 360 X 132	2,400 kN

The SLS case will not govern for piles driven to bedrock.

The expected pile tip elevations, based on the bedrock elevations, are shown in Table 7.3.

Table 7.3 – Pile Tip Elevations

Foundation	Borehole	Bedrock Elevation
West Abutment	418-6A	337.1
	418-7	340.8
Pier	418-4	346.0
	418-5	344.6
East Abutment	418-15	349.8
	418-18	349.6

In the case of a conventional abutment, it is anticipated that the design will result in piles 6 to 8 m long at the east abutment and progressively longer at the pier and west abutment.

In the case of an integral abutment, the piles must be provided with a free length of 3 m below the underside of the abutment stem. If the integral abutment design results in less than 5 m of embedment, i.e. a total pile length of less than 8 m, it is recommended that the

piles be socketed into rock for a distance of 500 mm with the rock socket backfilled with concrete. Construction of sockets will require mobilization of a separate rig and will add significantly to the cost of the foundation.

7.8 Recommended Foundation System

A comparison of foundation alternatives based on advantages and disadvantages of each foundation alternative is included in Table D1, Appendix D. Based on the local experience and the difficulty in inspecting the base of drilled shafts seated on bedrock, the recommended foundation system is steel H-piles driven to bedrock.

The following sections provide additional recommendations for driven piles. Additional design recommendations for drilled shafts may be provided in the final report if this foundation type is selected for this site.

Depending on the selected abutment type and the final arrangement, the piles at the east abutment may have to be socketed into bedrock.

7.9 Downdrag

Long-term settlements at the approach embankment will result in downdrag forces on the piles. Estimates of downdrag forces per pile are summarized in Table 7.4.

Table 7.4 – Downdrag Forces on Abutment Piles

West Abutment				
Pile Type	HP 310x110	HP 310x132	HP 310x152	HP 360x132
Estimated downdrag force (*)	1,500 kN	1,500 kN	1,500 kN	1,775 kN
Factored downdrag force (f = 1.25)	1,875 kN	1,875 kN	1,875 kN	2,200 kN
East Abutment				
Pile Type	HP 310x110	HP 310x132	HP 310x152	HP 360x132
Estimated downdrag force (*)	320 kN	320 kN	320 kN	360 kN
Factored downdrag force (f = 1.25)	400 kN	400 kN	400 kN	450 kN

(*) Downdrag forces have been calculated assuming that the negative skin friction will be mobilized at the outside perimeter of the “H” pile, between the underside of the pile cap, at approximate E.356 for the West Abutment and EL.355 for the East Abutment, and the Neutral Plane at the base of the silty clay layer. The analysis was carried out assuming the subsurface conditions and drained shear strength values calculated based on “Beta” values shown in Table 9.1

7.10 Pile Tips

The H-piles for the recommended foundation scheme will be driven to bedrock and will penetrate the thin layer of very dense sand and gravel with cobbles that overlies the

bedrock. Due to the possibility of a sloping bedrock surface and the comparatively low lateral confinement afforded by the native soils at this site, it is recommended that the pile tips be fitted with rock points. A suitable rock point is that shown in OPSD 3304 or, alternatively, the Titus Steel Company Rock Injector, APF Hard Bite, or equal from another approved manufacturer.

7.11 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

7.12 Pile Driving

The piles should be driven to bedrock. Note No. 6 from Article 3.3.3 “Pile Driving Notes” in the MTO Structural Manual should be used on the Foundation Drawing, i.e. “Piles to be fitted with rock points and driven into bedrock in accordance with 903S01”.

7.13 Lateral Resistance of Piles

Horizontal loads may be resisted either by batter piles or through the lateral resistance developed by the pile acting against the surrounding soil.

The lateral resistance of the piles must be calculated based on the following horizontal coefficients of subgrade reaction (k_s) and taking account of the ultimate lateral resistance (p_{ult}):

- Granular engineered fill (below the 600mm CSP for integral abutments):

- $k_s = f \cdot z / D \text{ (MN/m}^3 \text{)}$

Where $f = 18 \text{ MN/m}^3$

z = depth below ground surface

D = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma \cdot z^3 \cdot K_p \text{ (kPa)}$

Where $\gamma = 22 \text{ kN/m}^2$

z = depth below ground surface (m)

$K_p = 3.7$ (passive earth pressure coefficient)

- Sand / Silt:

- $k_s = f \cdot z / D \text{ (MN/m}^3 \text{)}$

Where $f = 3 \text{ MN/m}^3$

z = depth below ground surface

D = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma \cdot z \cdot 3 \cdot K_p \text{ (kPa)}$

Where $\gamma = 20 \text{ kN/m}^2$

z = depth below ground surface (m)

$K_p = 3.3$ (passive earth pressure coefficient)

- Silty Clay:

- $k_s = 6.5 / D \text{ (MN/m}^3 \text{)}$

Where D = pile diameter or width in a direction perpendicular to the pile movement (m)

- $p_{ult} = 500 \text{ kPa } (\cong 9 \cdot C_u)$

The appropriate elevations for each soil type are shown in Table 7.5.

Soil	Stratigraphic Elevations	
	West Abutment	East Abutment
Fill	Above 350.0	353.8
Sand/Silt	350.0 to 346.0	353.8 to 351.5
Silty Clay/Clayey Silt	346.0 to 340.0	351.5 to 350.0

Spring constant (K_s) and ultimate spring load (P_{ult}) values for numerical analysis of the integral abutment piles can be obtained by multiplying the k_s and p_{ult} values above, respectively, by the pile diameter or width (in a direction perpendicular to the pile movement) and the vertical distance between nodal points of the numerical model mesh along the pile.

7.14 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.9 m.

It is recommended that the full depth of soil cover be provided unless adequate synthetic insulation is incorporated in the design to replace the soil cover. It is recommended that the frost protection not be reduced at this site, due to the nature of the native soils and the high groundwater table.

It should be noted that rock fill does not provide the insulation value of soil cover. Where rock fill is used as backfill or for the construction of forward slopes in frost sensitive locations, consideration should be given to incorporating synthetic insulation.

Rigid, extruded polystyrene (EPS) insulation may be used and it may be assumed that 25 mm of this insulation provide protection equivalent to 600 mm of soil cover.

7.15 Abutment Considerations

7.15.1 Retained Soil Systems

Retained Soil System walls are not considered suitable for the West Abutment due to the potential for relatively large settlement during construction and long-term settlements due to secondary consolidation.

Retained soil system (RSS) walls may be used at the East Abutment provided that the levelling pad for the RSS wall is formed directly on the exposed bedrock, mass concrete fill or on a pad of engineered fill seating on bedrock. Engineered fill should be designed in the same manner as the engineered fill to support foundations as described elsewhere in this report. The geotechnical resistance of the bedrock or engineered fill is as stated elsewhere in this report.

RSS walls should be specified to be “High Performance” and “High Appearance”.

The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The global stability of an RSS wall founded as described above will be satisfactory.

The internal stability of the RSS should be analysed by the supplier/designer of the proprietary product selected for this site.

The settlement of a wall founded on engineered fill pad is expected to be small and should to occur essentially as the RSS is constructed.

7.15.2 Integral and Conventional Abutments

Integral and conventional abutments are considered suitable for this site. To provide the required flexibility in the piles of integral abutments, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP filled with sand in accordance with standard integral abutment design procedures.

7.16 Backfill to Abutments

In the case of integral abutments, the backfill should consist of granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. The rock fill used as backfill to the abutment should be limited to fragments no greater than 300 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and the abutment wall is backfilled with granular fill, the granular backfill should consist of OPSS Granular “B” Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

8 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site and any earth fill are classed as Type 3 soils. Unwatering will be required prior to excavation below the existing ground surface.

9 APPROACH EMBANKMENTS

9.1 General

The approach embankments to the proposed underpass structure are up to 16.8m high and will be founded on compressible deposits of silty clay to clayey silt. The geotechnical issues associated with these embankments are:

- Stability of the embankments, particularly during construction and during a seismic event
- Large long-term settlements due to primary and secondary consolidation
- Large lateral deflections or spreading of the underlying soils in the area of the abutment piles associated with settlements due to consolidation and during a seismic event

An engineering analysis was carried out in order to address the issues above as follows:

- \$ Analysis of embankment cross sections behind the abutments and forward slope to investigate stability and the possible requirements for stabilizing berms and/or construction staging to achieve a factor of safety of 1.3 during construction and 1.5 long term
- \$ In the event that staging is required, to investigate the necessary degree of consolidation between construction stages to allow sufficient gain in strength to maintain a minimum factors of safety of 1.3 during construction
- \$ Analysis to investigate the need for the use of wick drains and different light or ultra-lightweight fill to accommodate the construction schedule and to reduce long-term settlements
- \$ Analysis of lateral deflection at the abutment pile locations for structure pile design by others

Stability analysis was carried out using the alternatives of SSM or rock fill. SSM was assumed rather than earth fill for the following reasons:

- Construction control – to exclude earth fill that would not satisfy the assumed strength parameters
- Settlement concerns

The top of an embankment should be expected to experience settlement resulting from the consolidation of the fill. The settlement using SSM is expected to be equivalent to approximately 1% of the fill height, 160 mm in a 16 m high embankment. If uncontrolled earth fill is used the settlement could exceed 2% of the fill height, more than 320 mm.

The timing of the settlement due to self consolidation of the embankment fill depends on many factors including placement moisture content, degree of compaction, time for saturation/stabilization of internal moisture content and the initial freeze-thaw cycles.

9.2 Stability Analysis

9.2.1 General

Simplified stratigraphic profiles were selected for the subsurface conditions and embankment geometries behind the abutment locations in order to analyse the embankment stability. Table 9.1 presents simplified subsurface conditions for the west and east approach embankments.

The stability analysis was carried out assuming the following:

- Embankment Side Slopes:
 - Select Subgrade Material (SSM): 2H:1V
 - Rock Fill: 1.25H:1V
- Embankment Forward Slopes
 - 1.5H:1V during construction
 - Vertical abutment wall for long-term stability
- Berms:
 - The berms heights ranging from 4m to 8m above the original ground surface and with widths ranging from 20m to 25m were used in the analysis
- Geosynthetics Reinforcement:
 - Reinforcement of the base of the embankment using geosynthetics (geogrid or woven geotextile) was included in the analysis if required to increase the factor of safety against global instability
- Surcharge:
 - A surcharge of 2m above the top of pavement was assumed in accordance with the RFP
- Staging:
 - If required, construction staging for dissipation of excess pore pressures was considered in order to allow for soil shear strength gain
- Site Preparation:
 - All organic and soft soils will be removed within the footprint of the embankment and side berms
- Limit Equilibrium Analysis:
 - Bishop Modified using G-Slope, developed by Mitre Software
- Shear Strength of Native Soils:
 - Undrained shear strength (S_u) for cohesive soils as shown in Table 9.1. For vertical effective stresses (σ'_v) larger than the pre-consolidation pressure (p'), S_u was assumed equal to $0.25 \cdot \sigma'_v$. This assumption was

based on S_u/p' values measured in normally consolidated clays elsewhere near Bernard Lake.

- Drained shear strength (ϕ') as shown in Table 9.1
- Pore pressure generation:
 - Generation of excess pore pressures (EPP) upon undrained loading of the compressible and cohesive deposits is calculated assuming a B_{bar} (ratio of EPP over vertical total stress) of 0.9
- Piezometric Head:
 - West Approach: 1m artesian at the underside of the clay layer and hydrostatic in the cohesionless soils above the clay layer
 - East Approach: hydrostatic

Selected stability analysis results are presented in Appendix C. The analysis indicates the following requirements for stability:

9.2.2 Stability analysis Results:

Selected output is included in Appendix C.

9.2.2.1 West Approach

Fill Material: Select Subgrade Material (SSM)

- Construction Stages: One to the top of surcharge.
- West Forward Slope:
 - 1.5H:1V during construction
 - Vertical abutment wall for long-term stability
 - SBL of proposed Hwy11 embankment must be constructed to EL.357 before construction of the West Approach Embankment above this elevation
- West Approach Embankment Side Slope: 2H:1V
- Side Berms:
 - Slope: 2H:1V
 - Width: 21m
 - Height: 6m above original ground surface at EL.357
 - Reinforcement: Not required. The analysis showed that the use of geosynthetic reinforcement in order to reduce the berm height and width is

not an effective way of increasing the factor of safety against stability due to the deep seated nature of the critical slip surfaces

Fill Material: Rock Fill (except at forward slope and 2m surcharge)

- Construction Stages: One to the top of surcharge
- West Forward Slope (SSM during construction):
 - 1.5H:1V during construction
 - Vertical abutment wall for long-term stability
 - SBL of proposed Hwy11 embankment must be constructed to EL.355 before construction of the west approach embankment above this elevation
- West Approach Embankment Side Slopes: 1.25H:1V
- Side Berms:
 - Slope: 1.25H:1V
 - Width: 21m
 - Height: 4m above original ground surface at EL.355
 - Reinforcement: Not required. The analysis showed that the use of geosynthetic reinforcement in order to reduce the berm height and width is not an effective way of increasing the factor of safety against stability due to the deep seated nature of the critical slip surfaces

9.2.2.2 East Approach

Fill Material: Select Subgrade Material (SSM)

- Construction Stages: One to the top of surcharge.
- East Forward Slope:
 - 1.5H:1V during construction
 - Vertical abutment wall for long-term stability
 - NBL of proposed Hwy11 embankment must be constructed to EL.357 before construction of the west approach embankment above this elevation
- East Approach Embankment Side Slope: 2H:1V
- Side Berms:
 - Slope: 2H:1V
 - Width: 4m
 - Height: 3m above original ground surface at EL.357

- Reinforcement: Not required.

Fill Material: Rock Fill (except at forward slope and 2m surcharge)

- Construction Stages: One to the top of surcharge
- East Forward Slope (SSM during construction):
 - 1.5H:1V during construction
 - Vertical abutment wall for long-term stability
 - NBL of proposed Hwy11 embankment must be constructed to EL.357 before construction of the west approach embankment above this elevation
- East Approach Embankment Side Slope: 1.25H:1V
- Side Berms:
 - Slope: 1.25H:1V
 - Width: 2m
 - Height: 3m above original ground surface at EL.357
 - Reinforcement: Not required.

9.3 Settlement Analysis

9.3.1 General

The settlement analysis was carried out for the approach embankments within 20m of the east and west abutments and for the proposed Highway 11 mainline embankment, within 20m of the bridge centreline.

The settlement analysis was carried out in the following steps:

- One-dimensional primary consolidation analysis
- One-dimensional secondary consolidation analysis

The soil layer thickness and soil properties used in the analysis are shown on Table 9.1

9.3.2 One-Dimensional Consolidation Analysis - Methodology

One-dimensional consolidation analyses were carried out in order to assess the total settlement and time required for dissipation of pore water pressures in excess of hydrostatic. The analysis was carried out using Geocalc, a spreadsheet based software developed by Thurber. The program allows the one dimensional consolidation analysis of

multi-layered soil masses, taking into account two dimensional stress distribution in the foundations soils, non-linear constitutive law, soil parameters varying as a function of the over-consolidation ratio and variable boundary conditions.

The analyses were carried out assuming that the embankment construction will occur in one stage, instantly at the start of construction. This is a simplified model of the actual construction process in which several days or weeks are typically required to complete each of the construction stages. This approach over predicts excess pore pressures since it assumes that there will be no dissipation of pore pressures during fill placement.

9.3.3 One-dimensional secondary consolidation analysis - Methodology

Settlements due to secondary consolidation of normally consolidated to lightly over consolidated clayey soils (over-consolidation ratio less than 1.2) have been assessed based on the following equation:

$$\Delta T_{cs} = C\alpha\epsilon \cdot T \cdot \text{Log } t_{sc}/t_p,$$

Where:

ΔT_{cs} = settlement due to secondary consolidation

$C\alpha\epsilon$ = secondary compression ratio

T = initial thickness of compressible layer

t_{sc} = time over which secondary consolidation is to be calculated

t_p = time to complete primary consolidation

Clayey soils with an OCR equal to or larger than 1.2 the anticipated settlements due to secondary consolidation were calculated using the method by Mesri and Feng¹ (1991).

The results of the one-dimensional settlement analysis are presented in Table 9.2.

9.3.4 Settlement Analysis - Results

West Approach

Table 9.2 shows that large settlements due to primary consolidation, in the order of 500mm to 600mm, are anticipated at the West Approach Embankment. 98% of these settlements are anticipated to occur within 190 days of the end of the embankment construction to the top of surcharge. The anticipated settlement due to primary consolidation of the Hwy11 – SBL embankment in front of the West Approach Embankment is 55mm and the time required for 98% consolidation is 220 days. Therefore, it is anticipated that between 6 to 8

¹ Mesri G and Feng.T.W., 1991. "Surcharge to Reduce Secondary Consolidation" Geo-Coast '91, 3-6 Sept., 1991, Yokohama, pp.359-364

months after the end of the embankment construction will be required before the surcharge at the West Approach Embankment can be removed, the abutment piles installed and the pavement constructed. The post construction settlements due to secondary consolidation are expected to be in the order of 30mm to 60mm, 10 years after the end of the pavement construction.

East Approach

Table 9.2 shows that settlements due to primary consolidation up to 70mm are anticipated at the East Approach Embankment and that they are expected to stabilize relatively quickly, within two weeks of the end of construction. The anticipated settlement due to primary consolidation of the Hwy11 – NBL embankment in front of the East Approach Embankment is small, in the order of 10mm. The post construction settlements due to secondary consolidation are expected to be in the order of 5mm to 20mm for both, the East Approach and the Hwy11 – NBL embankments, 10 years after the end of the pavement construction.

9.4 Lateral Displacements at the Abutment Piles

Provided that the abutment piles are installed after most of the foundation soil settlements due to primary consolidation under the abutment fills have taken place, relatively small time dependent lateral displacements are anticipated to occur along the piles. For monitoring purposes and verification of the structural capacity of the abutment piles, the maximum outstanding pile long-term lateral deflection should be equal to 15mm and 5mm for the West and East Abutments, respectively (about 20%, Ladd², of the maximum outstanding settlement of the embankment). The maximum lateral deflection values above are anticipated to occur at the centre of the silty clay/clayey silt layer, at approximate EL.344 and EL.351, at the West and East Abutments, respectively. The lateral deflections should be assumed decreasing to zero above and below the point of maximum deflection, at the underside of the abutment (or underside of the CSP in case of integral abutments) and at the top of the sand and gravel layers, at EL.340 and EL.350, for the West and East Abutments, respectively.

² Ladd, C.C. (1991). A Stability Evaluation During Staged Construction@, ASCE Journal of Geotechnical Engineering, Vol.17, No.4, 1991

10 STATIC EARTH PRESSURE

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

$$P_h = K*(\gamma h + q)$$

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see below)

γ = unit weight of retained material (see table)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or 2.0 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown below in Table 10.1.

Table 10.1 – Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 20 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.31	0.47*	0.20	0.26*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a

lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

11 SEISMIC CONSIDERATIONS

11.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 2 and the following seismic parameters should be used for design (Table 3.1.7 of the CHBDC: Sundridge):

- Velocity Related Seismic Zone: 2
- Zonal Velocity Ratio: 0.1
- Acceleration Related Seismic Zone: 2
- Zonal Acceleration Ratio (pha): 0.11g

The Soil Profile Type at this site is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC is, associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1. Therefore a Peak Horizontal Ground Acceleration (PHA) at ground surface of 0.10g (where g is the gravity acceleration) should be used for design of the bridge.

11.2 Embankment Stability and Spread

The structure foundations will bear either on bedrock or on engineered fills founded on bedrock and consequently there is no potential for soil liquefaction.

The approach embankments will be founded on compact to dense sands and silts and on clayey soils that are not considered to be in danger of liquefaction.

A limit equilibrium analysis of the embankment constructed with SSM and 21m wide and 6m high berms resulted in a yield acceleration ratio of 0.11g. A comparison of the peak ground acceleration (PGA) with the yield acceleration indicates that, when subject to the design earthquake loading conditions the embankment lateral spread will be negligible.

11.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause C4.6.4 of the CHBDC 2000 retaining structures should be designed using active (K_{AE}) and passive earth pressure (K_{PE}) coefficients that include earthquake loading. The following design parameters should be used to calculate K_{AE} and K_{PE} according to the CHBDC:

ϕ'	= angle of internal friction of backfill
ϕ'	= 35° for OPSS Granular A or Granular B Type II
ϕ'	= 32° for OPSS Granular B Type I
ϕ'	= 42° for Rock fill
k_h	= horizontal acceleration coefficient
k_h	= 0.5* 0.10 (PHA) = 0.05 for yielding structures (integral abutments)
k_h	= 1.5* 0.10 (PHA) = 0.15 for non-yielding structures (rigid retaining walls and abutments founded on battered piles)

Recommended values are shown in Table 11.1.

12 EMBANKMENT DESIGN ALTERNATIVES AND RECOMMENDATIONS

12.1 General

The stability and settlement analysis presented in this report show that the approach embankments along the proposed Highway 124 within 20m of the bridge abutments may be constructed in one stage to the top of the surcharge using either SSM or rock fill materials. It is critical, however, that the embankments for the proposed Hwy11 be constructed to the EL.357 and EL.355 for the SBL and NBL, respectively, prior to the start of the construction of the approach embankments.

A summary of the approach embankment geometry, including required berm width and height, and an approximate cost estimate for the approach embankments are presented in Table 9.3. Table 9.3 shows that the design alternatives have approximately the same costs.

From the geotechnical point of view the use of rock fill is preferable due the associated low potential for settlements due to the compression of the rock fill embankments.

12.2 West Approach Embankment

The presence of relatively thick compressible plastic soils beneath the west approach combined with large embankment loads will result in large settlements time dependent settlements due to primary consolidation and secondary consolidation. In order to reduce the potential for large post-construction settlements, the West Approach Embankment should be constructed with a surcharge 2m above the top of pavement elevation and the excess pore pressures generated during the embankment construction should be allowed to dissipate prior to removal of the surcharge and installation of the abutment piles. It is

anticipated that up to 8 months after the end of the embankment construction will be required for dissipation of excess pore pressures. This period may be reduced with the use of wick drains and/or lightweight fill materials such as Rigid Expanded Polystyrene (REP).

Notwithstanding the use of 2m of surcharge and a waiting period of 8 months for dissipation of excess pore pressures, post-construction settlements up to 55mm are anticipated at the West Approach Embankment 10 years after the end of construction. This is due to the relatively low degree of over consolidation achieved in the foundation soils after the end of primary consolidation and removal of the surcharge. It is not considered practical to achieve higher degrees of consolidation, and consequently to reduce post-construction settlements, by increasing the thickness of the surcharge. REP fill should be considered as a design alternative if post-construction settlements of the West Approach Embankment smaller than 55mm are required over a period of 10 years.

In order to reduce the potential for long-term settlements close to the bridge abutments, it is critical that the surcharge be extended to the back of the abutments and sloped down towards the Hwy11 embankment at the slope angles indicated earlier in this report.

12.3 East Approach Embankment

Table 9.2 shows that relatively small post-construction settlements (less than 25mm 10 years after the end of construction) are anticipated for the East Approach Embankment constructed with a surcharge 2m above the top of pavement. It is anticipated that the excess pore pressures generated during construction will dissipate relatively quickly (in less than 30 days) at which point the surcharge can be removed and abutment piles constructed. A combination of surcharging the foundation soils and wick drains should be considered as design alternatives if either the post-construction settlements are considered too large or if the time required for consolidation does not meet the construction schedule requirements.

In order to reduce the potential for long-term settlements close to the bridge abutments, it is critical that the surcharge be extended to the back of the abutments and sloped down towards the Hwy 11 embankment at the slope angles indicated earlier in this report.

12.4 Hwy 11 Embankments

The Hwy 11 embankments should be constructed to EL.357 and EL.355 for the SBL and NBL, respectively, before the beginning of construction of the Hwy 124 approach embankments. This is required to maintain stability of the forward slopes of the Hwy 124 approach embankments. In order to reduce post-construction settlements at the SBL of the Hwy 11, the fill material should be placed to the top of the pavement and be left in place for approximately 8 months for dissipation of excess pore pressures. The calculated post-construction settlements 10 years after the end of construction associated with this method are up to 30 mm.

Table 11.1
Earth pressure Coefficients for Seismic Design

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.30	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})*	6.3	-	5.4	-	12.0	-
At Rest (K_{OE})**	0.59	-	0.63	-	0.33	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

12.5 Monitoring Program

The design proposed above was based on assumptions that should be confirmed with a geotechnical instrumentation and monitoring program implemented during construction of the embankments. The monitoring program is considered critical to reduce the risk of instability of the embankment during construction and to establish the appropriate time to remove the surcharge and start installation of the abutment piles.

13 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

Potential construction concerns include, but are not necessarily limited to:

- The proposed Hwy11 embankments should be in place with minimum heights specified above, before the construction of the approach embankments. They will act as stabilizing berms for the approach embankment slopes
- The results of the monitoring program will control the rate of the embankment construction and consequently the construction schedule. Although not anticipated, there is a risk that the pore pressure dissipation will be slower than anticipated. If this situation occurs, the embankment construction may have to be slowed down which may impact the overall construction schedule. A detailed and regular analysis of the monitoring program during construction is considered critical to:
 - Reduce potential of an embankment failure
 - Reduce the risk of a premature removal of the surcharge
 - Reduce the risk of installing the abutment piles too early
- Completion of the approach embankments and stabilization of settlements due to primary consolidation must take place before the installation of the abutment piles
- Dewatering of excavations at East Abutment due to the presence of soils with high fine contents

Thurber Engineering Ltd.

Report prepared by:

Alastair E. Gorman, P.Eng., M.Sc.

Senior Foundations Engineer



Report reviewed by:

Paulo J. Branco, P.Eng., Ph.D.

Principal

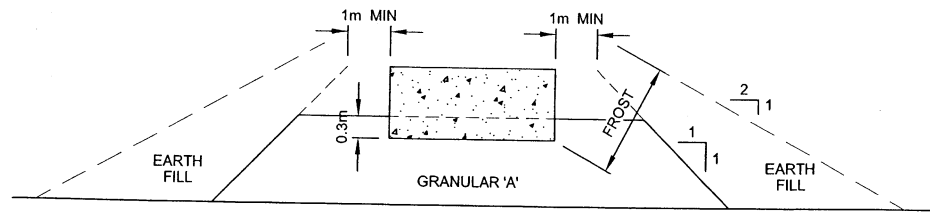


HIGHWAY 11 - ROBINS ROAD/BLACK CREEK
SOIL PROPERTIES FOR STABILITY AND SETTLEMENT ANALYSIS

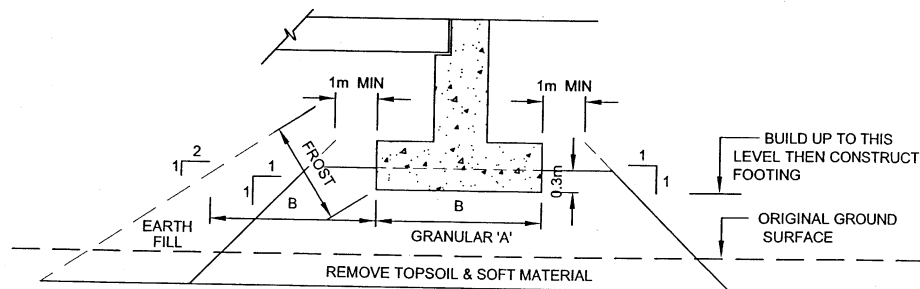
Location	Soil Layer	Depth Interval		Unit Weight (kN/m3)	Undrained Shear Strength		Drained Shear Strength		Shear Strength at Pile/Soil Interface (Beta)	Poisson's Ratio	Young's Modulus (MPa)	Compression Ratio		Pre-Consolidation Pressure (kPa)	Coeff. Of Consolidation (m2/y)				Secondary Compression Ratio $C_{\alpha}/(1+e_0)$
		From (m)	To (m)		Cohesion (kPa)	Friction Angle (deg)	Cohesion (kPa)	Friction Angle (deg)				Cc/(1+eo)	Cr/(1+eo)		Cv		Ch		
															O.C.	N.C.	O.C.	N.C.	
West Approach BH418-1A	Rock or SSM	top of fill	1	20/22	---	---	0	42/30	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand/Silt	1	3	20	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Silty Clay Top	3	6	18	70	0	0	28	0.33	0.45	21	0.15	0.015	---	40	30	80	60	0.006
	Silty Clay Middle	6	8	17	35	0	0	28	0.27	0.45	11	0.20	0.020	120	40	30	80	60	0.006
	Silty Clay Bottom	8	10	18	55	0	0	28	0.31	0.45	16.5	0.15	0.015	140	40	30	80	60	0.006
West Abutment BH418-6A BH418-7	Sand & Gravel	10	13	21	---	---	0	33	---	0.30	40	---	---	---	---	---	---	---	---
	Rock or SSM	top of fill	1	20/22	---	---	0	42/30	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand/Silt	1	4	20	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Silty Clay Top	4	9	18	75	0	0	28	0.27	0.45	23	0.15	0.015	250	40	30	80	60	0.006
	Silty Clay Bottom	9	11.5	17	50	0	0	28	0.31	0.45	15	0.15	0.015	300	40	30	80	60	0.006
East Abutment BH418-18 BH418-15	Sand & Gravel	11.5	12	21	---	---	0	33	---	0.30	40	---	---	---	---	---	---	---	---
	Rock or SSM	top of fill	0.5	20/22	---	---	0	42/30	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand/Silt	0.5	2.5	20	---	---	0	32	0.3	0.35	30	---	---	---	---	---	---	---	---
	Clayey Silt	2.5	4	18	80	0	0	28	0.33	0.45	24	0.15	0.015	---	---	---	---	---	---
East Approach BH418-20	Sand & Gravel	4	4.5	21	---	---	0	33	---	0.30	40	---	---	200	40	30	80	60	0.006
	Rock or SSM	top of fill	0.3	20/22	---	---	0	42/30	0.3	0.30	150	---	---	---	---	---	---	---	---
	Sand/Gravel	0.3	3.5	21	---	---	0	33	---	0.30	40	---	---	---	---	---	---	---	---

Notes: O.C.: Over Consolidated Soil
N.C.: Normally Consolidated Soil

March, 2004



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

TED35146.DWG

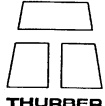
ENGINEER	AEG	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	 THURBER DWG. NO.
DRAWN	SS		
DATE	April, 2004		
APPROVED	PKC		
SCALE	NTS		

FIGURE 1



Appendix A

Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

$\frac{\Delta}{C_{\text{pen}}}$ Water Level
Shear Strength Determination by Pocket Penetrometer






- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.



UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>
Fresh (FR)	No visible signs of weathering.	 CLAYSTONE  SILTSTONE  SANDSTONE  COAL  BENTONITE
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.	
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.	
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.	
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.	
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.	

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 200	Greater than 29,200	Requires many blows of geological hammer to break.
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-200	14,600 to 29,200	Requires a few blows of geological hammer to break.
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,300 to 14,600	Breaks under single blow of geological hammer.
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Moderately Strong	12.5 to 50.0	1,825 to 7,300	¼” indentations with sharp end of geological pick.
<u>TERMS</u>		Moderately Weak	5.0 to 12.5	730 to 1,825	Too hard to cut by hand into triaxial specimen.
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	1.25 to 5.0	182 to 730	Crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.				
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Very Weak (Rock)	0.60 to 1.25	85 to 182	May be broken in the hand with difficulty.
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				



RECORD OF BOREHOLE No 418-1A

1 OF 2

METRIC

W.P. 5039-03-01 LOCATION N 5 068 807.0 E 311 334.8 (Hwy 124) ORIGINATED BY DP
DIST HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 17.12.03 - 17.12.03 CHECKED BY JL

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
350.0	Sandy TOPSOIL, some rootlets												
349.3													
0.7	SAND, very fine grained, some silt, trace organics Compact Grey Wet		1	SS	16								
348.6													
1.5	SILT, some sand, some clay Compact Grey Wet		2	SS	14								0 13 74 13
347.3													
2.7	Clayey SILT, trace sand Firm to Stiff Grey Wet		3	SS	10								
			1	TW	PH								
			4	SS	3								0 6 65 29
			2	TW	PH								
			3	TW	PH								
			5	SS	1								0 4 58 38
			4	TW	PH								0 4 77 19

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

ONTMT4 418HWY 124.GPJ 22/01/04

RECORD OF BOREHOLE No 418-1A										2 OF 2		METRIC							
W.P. 5039-03-01		LOCATION N 5 068 807.0 E 311 334.8 (Hwy 124)				ORIGINATED BY DP													
DIST HWY 11		BOREHOLE TYPE Hollow Stem Augers				COMPILED BY SS													
DATUM Geodetic		DATE 17.12.03 - 17.12.03				CHECKED BY JL													
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80	100	20	40
339.8	SAND, fine to very fine grained, some clay, some gravel Compact Brown Wet		6	SS	5														
10.2																			
336.9			7	SS	16														
13.1	END OF BOREHOLE AT 13.11m. AUGER REFUSAL AT 13.11m. PROBABLE BEDROCK BOREHOLE BACKFILLED WITH DRILL CUTTINGS AND DRY BENTONITE, SEAL AT SURFACE.																		

ONTMT 4 418HWY 124.GPJ 22/01/04

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 418-4

1 OF 1

METRIC

W.P. 5039-03-01 LOCATION N 5 068 833.9 E 311 386.9 (Hwy 124 Pier N-E) ORIGINATED BY DP
DIST HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NW Casings/ NQ Core COMPILED BY SS
DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY JL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
351.6								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
350.8	Sandy TOPSOIL							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
0.2	Sandy SILT, some to trace clay Dense to Compact Brown Wet		1	SS	36		351								
			2	SS	28		350								0 22 65 1
			3	SS	15		349								
	Grey		4	SS	11		348								
347.5															
4.1	Clayey SILT, trace sand, some clay Firm Grey Wet (CL-ML)		5	SS	5		347								0 3 73 2
346.4															
5.2	SAND and GRAVEL, trace silt														
346.0	Very Dense														
5.6	Brown														
	BEDROCK, GRANITIC GNEISS, pink with visible black banding, fresh, slightly weathered at joints, very strong		1	RUN			346								RUN #1 TCR = 90%, SCR = 78%, RQD = 70% UCS=132.5MPa
			2	RUN			345								RUN #2 TCR = 100%, SCR = 88%, RQD = 83% UCS=160.5MPa
			3	RUN			344								RUN #3 TCR = 96%, SCR = 96%, RQD = 80% UCS=166.0MPa
342.9							343								
8.7	END OF BOREHOLE AT 8.74 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 18/12/03 0.91 above GS														

+ 3, x 3: Numbers refer to
Sensitivity 20
15 5 10 (%) STRAIN AT FAILURE

ONTM14 418HWY 124 GPJ 07/01/04

RECORD OF BOREHOLE No 418-5

1 OF 1

METRIC

W.P. 5039-03-01 LOCATION N 5 068 808.4 E 311 390.8 (Hwy 124 Pier S) ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NW Casings/ NQ Core COMPILED BY WM
DATUM Geodetic DATE 17.11.03 - 17.11.03 CHECKED BY JL

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
351.3												
350.2	Sandy TOPSOIL											
0.2	Silty SAND, fine grained Compact Brown Wet	1	SS	22	351							
		2	SS	16	350							0 69 31 (SI+CL)
348.6		3	SS	25	349							
2.7	Sandy SILT, some clay Compact Grey Wet	4	SS	23	348							0 21 67 12
347.2												
4.1	Clayey SILT, trace sand Stiff Grey Wet (CL-ML)	5	SS	8	347							0 7 64 29
345.5					346							
5.8	Silty SAND, fine grained Loose Wet	6	SS	7	345							
	Auger shifting south from 6.7m to 7.7m. Possible sloping bedrock contact at 6.7m (344.6).				344							
343.6		7	SS	52								
7.7	END OF BOREHOLE AT 7.72 m. AUGER REFUSAL AT 7.72m PROBABLE BEDROCK WATER LEVEL ON COMPLETION OF DRILLING AT 3.66 m. BOREHOLE OPEN TO 5.03 m. BOREHOLE BACKFILLED WITH BENSEAL/BENTONITE AND CUTTINGS TO SURFACE.			.102								

ONTMT4 418HWY 124.GPJ 2006/04

+ 3, × 3 : Numbers refer to
Sensitivity 15 20
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 418-6A										1 OF 2		METRIC	
W.P. 5039-03-01		LOCATION N 5 068 800.8 E 311 351.6 (Hwy 124 West Abutment S-E)				ORIGINATED BY DP							
DIST HWY 11		BOREHOLE TYPE Hollow Stem Augers/ NQ Core				COMPILED BY SS							
DATUM Geodetic		DATE 10.12.03 - 10.12.03				CHECKED BY JL							
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W _p W _L	20 40 60			
350.2	Sandy TOPSOIL, some rootlets												
349.5	Silty SAND, very fine to fine grained, trace clay Compact Grey Wet		1	SS	20							0 66 32 2	
347.7	Sandy SILT, trace to some clay, occasional sand layers Compact Grey Wet (ML-nonplastic)		2	SS	24								
346.1	Clayey SILT, trace sand, occasional silt layers Firm Grey Wet		3	SS	34							0 21 70 10	
			4	SS	28								
			5	SS	5							0 6 67 26	
			1	TW	PH								
			2	TW	PH							0 8 66 25	
			6	SS	6								
			3	TW	PH							0 7 64 30	
			7	SS	7								

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 418-6A										2 OF 2		METRIC		
W.P. 5039-03-01		LOCATION N 5 068 800.8 E 311 351.6 (Hwy 124 West Abutment S-E)				ORIGINATED BY DP								
DIST HWY 11		BOREHOLE TYPE Hollow Stem Augers/ NQ Core				COMPILED BY SS								
DATUM Geodetic		DATE 10.12.03 - 10.12.03				CHECKED BY JL								
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	20	40	60	80			100
	becoming soft		4	TW	PH									
	medium plasticity (CI)		8	SS	2									
338.8														
11.4	SAND and GRAVEL		9	SS	75/ 176									
	cobbles from 12.19m to 13.06m													
337.1			1	RUN										
13.1	BEDROCK, GRANITIC GNEISS , pink with visible black banding, fresh, slightly weathered at joints, very strong													
			2	RUN										
334.1			3	RUN										
16.2	END OF BOREHOLE AT 16.15m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 18/12/03 1.0 above ground surface (frozen)													

ONTMT4 418HWY 124.GPJ 22/01/04

+³, ×³: Numbers refer to
Sensitivity 20
15 (5-5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 418-7

1 OF 2

METRIC

W.P. 5039-03-01 LOCATION N 5 068 825.3 E 311 349.4 (Hwy 124) ORIGINATED BY MF
DIST HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY WM
DATUM Geodetic DATE 21.11.03 - 21.11.03 CHECKED BY JL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
350.1												
350.0	Sandy TOPSOIL											
0.2	Dark Brown Silty SAND, fine grained, some organics, trace rootlets Loose to Dense Brown to Grey Wet		1	SS	8							
			2	SS	20							
			3	SS	32							
347.9												
2.2	SILT, some sand, some clay, some sand layers Compact Grey Wet		4	SS	15							0 15 73 13
			5	SS	13							
346.0												
4.1	Clayey SILT, trace sand Firm Grey Wet		6	SS	6							
			7	SS	7							0 5 69 26
			8	SS	6							
	occasional oxide staining		9	SS	100/							
340.9												
9.2	Silty SAND, fine to medium grained, some gravel, occasional cobbles				.150							
340.8	Very Dense											
9.3	Brown		1	RUN								RUN 1# TCR=100%,

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity 20
15 10
10 (%) STRAIN AT FAILURE

ONTM14 418HWY 124.GPJ 08/01/04

RECORD OF BOREHOLE No 418-7

2 OF 2

METRIC

W.P. 5039-03-01 LOCATION N 5 068 825.3 E 311 349.4 (Hwy 124) ORIGINATED BY MF
DIST HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY WM
DATUM Geodetic DATE 21.11.03 - 21.11.03 CHECKED BY JL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
339.7 10.4	<p>Wet</p> <p>BEDROCK, GRANITIC GNEISS, pink with visible black banding, fresh, slightly weathered at joints, very strong</p> <p>END OF BOREHOLE AT 10.39 m. ENCOUNTERED ARTESIAN CONDITION IN THE BEDROCK. BOREHOLE BACKFILLED WITH HOLE PLUG AND SAND TO SURFACE.</p>					340							SCR=100%, RQD=72%, UCS=182.6MPa

ONTM14 418HWY 124.GPJ 06/01/04

+³, ×³: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE)

RECORD OF BOREHOLE No 418-15

1 OF 1

METRIC

W.P. 5039-03-01 LOCATION N 5 068 817.8 E 311 430.4 (Hwy 124 East Abutment S-E) ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NW Casings/ NQ Core COMPILED BY WM
DATUM Geodetic DATE 14.11.03 - 14.11.03 CHECKED BY JL

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20 40 60 80 100	W _p	W	W _L	γ	GR SA SI CL
353.8												
350.0	Sandy TOPSOIL											
0.2	SAND, fine grained, some silt Dense to Compact Brown Moist		1	SS	37							
			2	SS	26							0 82 18 (SI+CL)
351.5												
2.3	Clayey SILT, trace fine grained Very Stiff to Firm Grey Wet		3	SS	21							0 6 73 21
			4	SS	7							
350.0												
3.8	SAND and GRAVEL											
349.8	Brown											
4.0	BEDROCK, GRANITIC GNEISS, pink with visible black banding, fresh, slightly weathered at joints, very strong		1	RUN								RUN #1 TCR = 100%, SCR = 96%, ROD = 84%, UCS=165.8MPa
			2	RUN								RUN #2 TCR = 97%, SCR = 93%, ROD = 85%, UCS=155.6MPa
			3	RUN								RUN #3 TCR = 100%, SCR = 100%, ROD = 85%, UCS=156.6MPa
346.8												
7.0	END OF BOREHOLE AT 7.04 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 18/11/03 0.99 below ground surface 18/12/03 1.21 below ground surface											

+ 3, x 3: Numbers refer to
Sensitivity 20
15 10
(%) STRAIN AT FAILURE

ONTMT4 418HWY 124.GPJ 2004/04

RECORD OF BOREHOLE No 418-18

1 OF 1

METRIC

W.P. 5039-03-01 LOCATION N 5 068 842.7 E 311 421.8 (Hwy 124 East Abutment N-W) ORIGINATED BY DP
DIST HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY JL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								○ UNCONFINED	+ FIELD VANE									
								● QUICK TRIAXIAL	× LAB VANE									
353.7																		
350.0	Sandy TOPSOIL																	
0.2	Silty SAND, fine grained, trace gravel Dense to Compact Brown Moist		1	SS	38		353											
			2	SS	23		352							6 64 31 (SH+CL)				
351.4	Clayey Silt, trace sand																	
2.3	Stiff Grey Wet (ML)		3	SS	13		351											
			4	SS	11		350							0 3 74 22				
349.9	SAND and GRAVEL, trace silt																	
3.8	Brown																	
349.6	Wet																	
4.1	AUGER REFUSAL AT 4.14 m ON ASSUMED BEDROCK. WATER LEVEL ON COMPLETION OF DRILLING AT 1.22 m. BOREHOLE OPEN TO 3.96 m. BOREHOLE BACKFILLED WITH BENSEAL AND DRILL CUTTINGS TO SURFACE.																	

+ ³, × ³: Numbers refer to Sensitivity 20 15 10 5 10 (%) STRAIN AT FAILURE

ONTM14 418HWY 124.GPJ 06/01/04

RECORD OF BOREHOLE No 418-20										1 OF 1	METRIC				
W.P. 5039-03-01		LOCATION N 5 068 834.9 E 311 446.2 (HWY 124)		ORIGINATED BY MF											
HWY 11		BOREHOLE TYPE Solid Stem Augers		COMPILED BY WM											
DATUM Geodetic		DATE 20.11.03 - 20.11.03		CHECKED BY JL											
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
354.8	TOPSOIL Dark Brown Silty SAND, fine grained, trace gravel Compact to Very Dense Brown Moist		1	SS	3										
354.0			2	SS	23										
352.6			3	SS	80										
352.2	SAND and GRAVEL, occasional cobbles, possible boulders Very Dense Brown Wet		4	SS	50/										
351.3			5	SS	50/										
3.5	END OF BOREHOLE AT 3.51 m. AUGER REFUSAL AT 3.51 m. PROBABLE BEDROCK. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. WATER LEVEL IN OPEN BOREHOLE AT 1.52m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE.														

ONTMT4 418-HWY 124 GPJ 2004/04

+ 3, x 3: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE

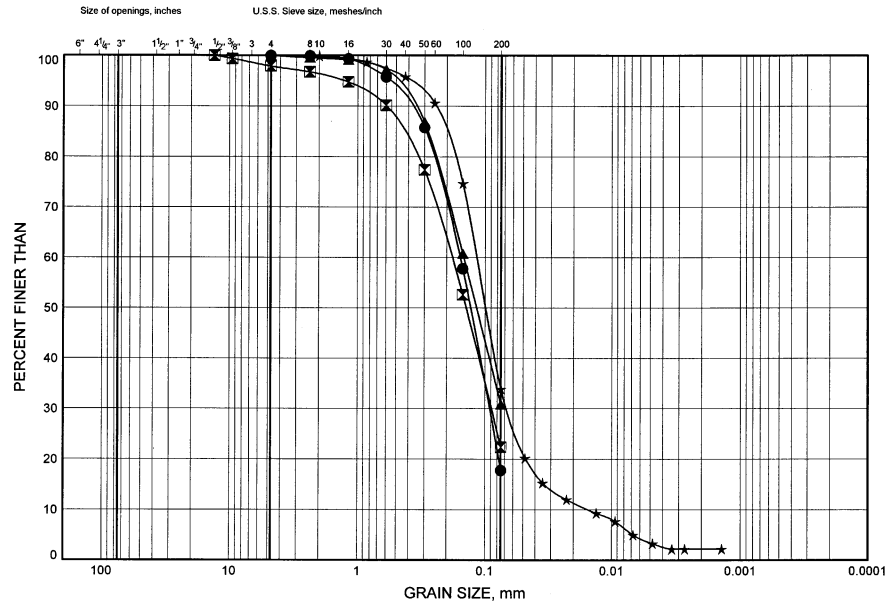
Appendix B

Laboratory Test Results

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B1

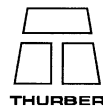
SILTY SAND



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-15	1.83	351.97
⊠	418-20	1.75	353.05
▲	418-5	1.83	349.47
★	418-6A	1.07	349.13

Date April 2004
Project 5039-03-01

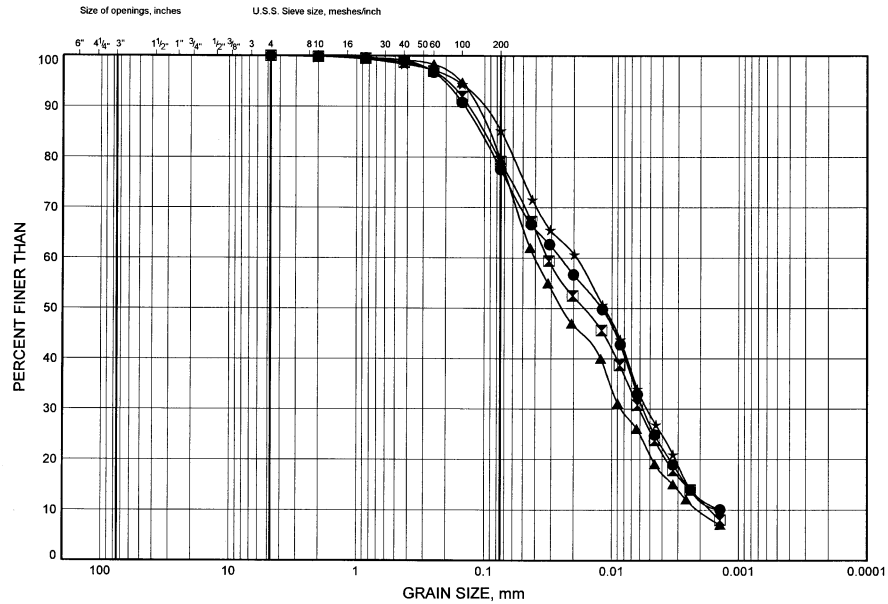


Prep'd SS
Chkd. AEG

Hwy 11 Four Laning
GRAIN SIZE DISTRIBUTION

FIGURE B2

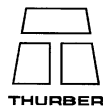
SANDY SILT



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-4	1.83	349.77
☒	418-5	3.35	347.95
▲	418-6A	2.82	347.38
★	418-7	2.59	347.51

Date April 2004
Project 5039-03-01



Prep'd SS
Chkd. AEG

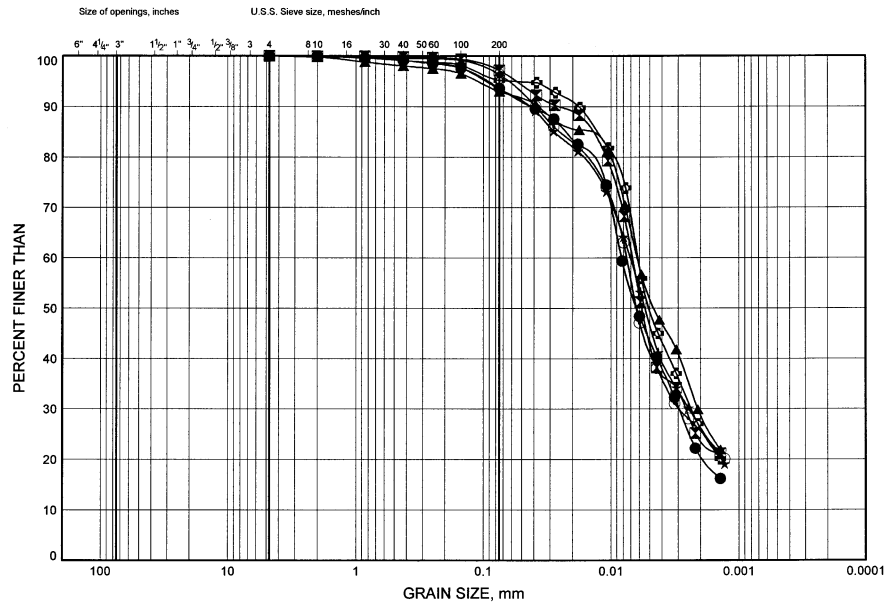
THURBGSD 418HWY 124.GPJ 200404



Hwy 11 Four Laning
GRAIN SIZE DISTRIBUTION

FIGURE B3

CLAYEY SILT

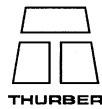


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-15	2.59	351.21
⊠	418-4	4.88	346.72
▲	418-5	4.88	346.42
★	418-6A	4.88	345.32
⊙	418-6A	10.97	339.23
⊛	418-7	6.40	343.70

THURBER 418HWY 124 GPJ 20/04/04

Date April 2004
Project 5039-03-01

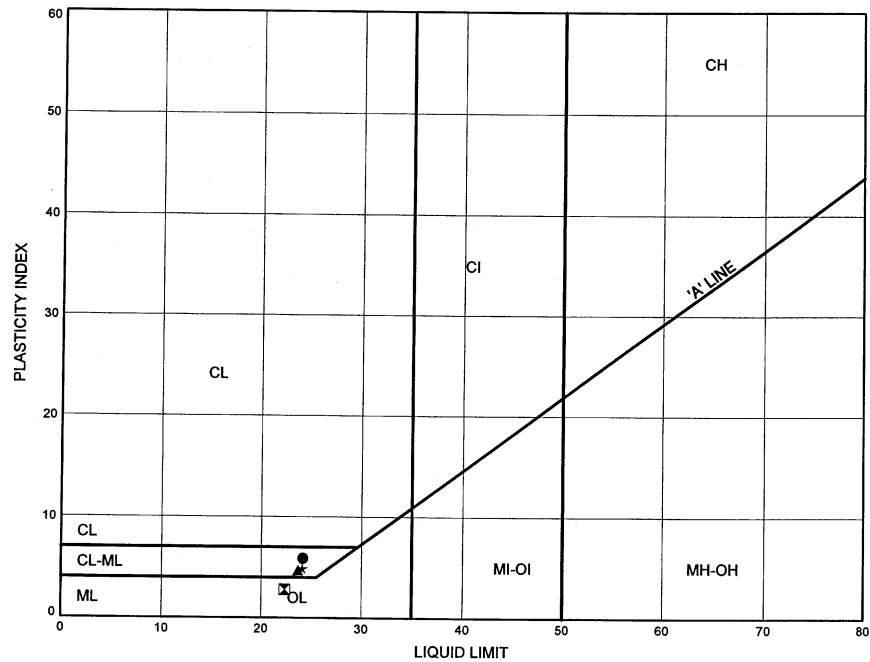


Prep'd SS
Chkd. AEG



Hwy 11 Four Laning
ATTERBERG LIMITS TEST RESULTS

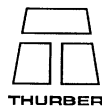
FIGURE B4



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-15	2.59	351.21
⊠	418-18	3.35	350.35
▲	418-4	4.88	346.72
★	418-5	4.88	346.42

THURBALT 418 HWY 124 GPJ 07/01/04

Date January 2004
 Project 5039-03-01

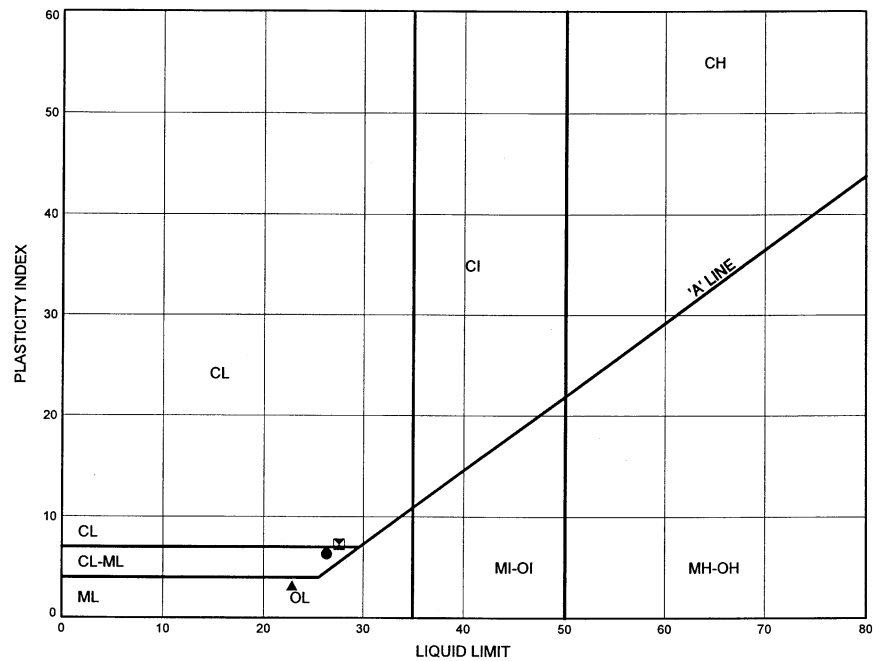


Prep'd WM
 Chkd. PJB



Hwy 11 Four Laning
ATTERBERG LIMITS TEST RESULTS

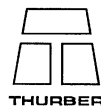
FIGURE B5



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-1A	4.88	345.12
⊠	418-1A	7.92	342.08
▲	418-1A	9.40	340.60

THURBALT 418HWY 124.GPJ 22/01/04

Date January 2004
 Project 5039-03-01

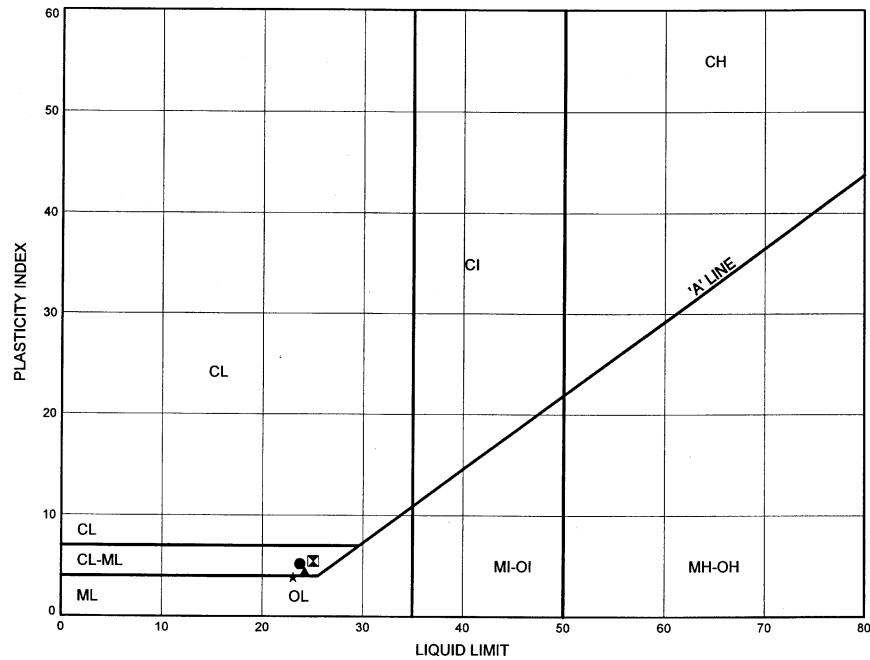


Prep'd SS
 Chkd. PJB



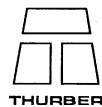
Hwy 11 Four Laning
ATTERBERG LIMITS TEST RESULTS

FIGURE B6



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	418-6A	4.88	345.32
⊠	418-6A	6.55	343.65
▲	418-6A	8.38	341.82
★	418-6A	10.97	339.23

Date January 2004
 Project 5039-03-01



Prep'd SS
 Chkd. PJB

THURBALT 418HWY 124.GPJ 22/01/04



**Site Number 44-418 - Hwy 124 I/C Underpass
Point Load Test Results**

Depth								
feet	Inches	m	Is50	UCS MPa				
BH418-4								
19	6	5.94	6.38	153.12	}			
20	9	6.32	4.68	112.32				
22	9	6.93	6.38	153.12				
24	5	7.44	7.02	168.48				
27	3	8.31	6.38	153.12				
						Average	Minimum	Maximum
						148	112	168 MPa
Depth								
feet	Inches	m	Is50	UCS MPa				
BH418-6A								
43	2	13.16	6.59	158.16	}			
43	11	13.39	6.51	156.24				
44	11.5	13.70	6.8	163.2				
46	0	14.02	6.8	163.2				
47	9	14.55	6.38	153.12				
49	0	14.94	6.38	153.12				
50	8	15.44	6.8	163.2				
52	6	16.00	7.02	168.48				
						Average	Minimum	Maximum
						160	153	168 MPa
Depth								
feet	Inches	m	Is50	UCS MPa				
BH418-7								
30	10	9.40	7.02	168.48	}			
32	2	9.80	8.55	205.2				
33	10	10.31	7.27	174.48				
						Average	Minimum	Maximum
						183	168	205 MPa
Depth								
feet	Inches	m	Is50	UCS MPa				
BH418-15								
14	6	4.42	6.93	166.32	}			
16	0	4.88	6.89	165.36				
17	0	5.18	7.02	168.48				
18	9	5.72	5.95	142.8				
21	0	6.40	6.59	158.16				
22	6	6.86	6.46	155.04				
						Average	Minimum	Maximum
						159	143	168 MPa

Table2

TABLE B1

Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone (905) 567-4444
Fax (905) 567-6561



January 23, 2004

04-1116-004

RECEIVED JAN 26 2004

Thurber Engineering Ltd.
2010 Winston Park Drive
Suite 103
Oakville, Ontario
L6H 5R7

Attention: Mr. Weiss Mehdawi

RE: GEOTECHNICAL LABORATORY TESTING

Dear Sirs:

This letter reports the results of laboratory testing carried out on the samples received at our office in Mississauga. The results of the tests are summarized in the following figures.

We trust that the results are sufficient for your current requirements. If you have any questions, please do not hesitate to call us.

Yours very truly,

GOLDER ASSOCIATES LTD.

Marijana Manojlovic
Laboratory Manager



OFFICES ACROSS NORTH AMERICA, SOUTH AMERICA, EUROPE, AFRICA, ASIA AND AUSTRALIA



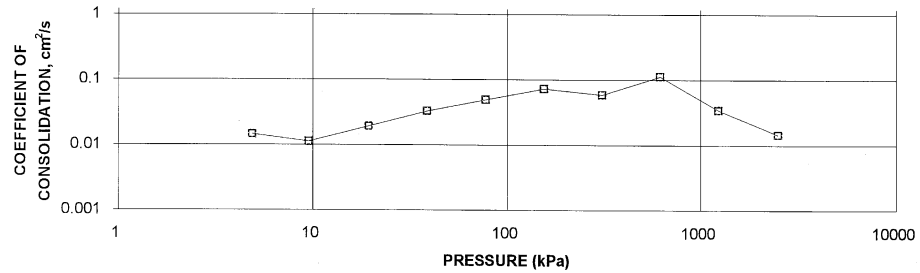
OEDOMETER CONSOLIDATION SUMMARY							
SAMPLE IDENTIFICATION							
Project Number	04-1116-004	Sample Number	ST#3				
Borehole Number	418-1A	Sample Depth, m	7.0-7.5				
TEST CONDITIONS							
Test Type	Standard	Load Duration, hr	24				
Oedometer Number	8						
Date Started	01/07/2004						
Date Completed	01/20/2004						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.92	Unit Weight, kN/m ³	19.21				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.94				
Area, cm ²	31.67	Specific Gravity, measured	2.73				
Volume, cm ³	60.65	Solids Height, cm	1.069				
Water Content, %	28.63	Volume of Solids, cm ³	33.84				
Wet Mass, g	118.83	Volume of Voids, cm ³	26.81				
Dry Mass, g	92.38	Degree of Saturation, %	98.7				
TEST COMPUTATIONS							
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.915	0.792	1.915				
4.85	1.904	0.782	1.910	53	1.46E-02	1.18E-03	1.69E-06
9.50	1.898	0.776	1.901	68	1.13E-02	6.74E-04	7.44E-07
19.40	1.888	0.767	1.893	40	1.90E-02	5.27E-04	9.82E-07
38.64	1.875	0.755	1.882	23	3.26E-02	3.53E-04	1.13E-06
77.43	1.859	0.740	1.867	15	4.93E-02	2.15E-04	1.04E-06
154.66	1.837	0.719	1.848	10	7.24E-02	1.49E-04	1.06E-06
309.21	1.804	0.688	1.821	12	5.86E-02	1.12E-04	6.40E-07
618.39	1.766	0.653	1.785	6	1.13E-01	6.42E-05	7.08E-07
1236.76	1.730	0.619	1.748	19	3.41E-02	3.04E-05	1.02E-07
2480.76	1.691	0.583	1.711	43	1.44E-02	1.64E-05	2.31E-08
1236.76	1.697	0.588	1.694				
618.39	1.704	0.595	1.701				
154.66	1.717	0.607	1.711				
77.43	1.724	0.613	1.721				
19.40	1.737	0.626	1.731				
4.85	1.750	0.638	1.744				
Notes:							
- k calculated using cv based on t ₉₀ values.							
- 77.4kPa and 618.4kPa load increments were held for 48 hours							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.75	Unit Weight, kN/m ³	20.14				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	16.35				
Area, cm ²	31.67	Specific Gravity, measured	2.73				
Volume, cm ³	55.42	Solids Height, cm	1.069				
Water Content, %	23.19	Volume of Solids, cm ³	33.84				
Wet Mass, g	113.80	Volume of Voids, cm ³	21.58				
Dry Mass, g	92.38						

Golder Associates

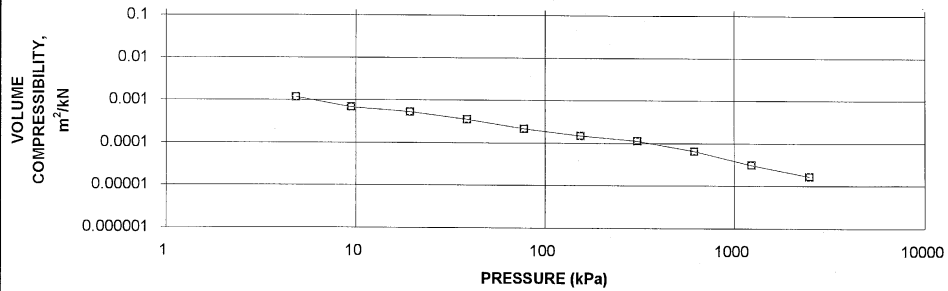


OEDOMETER CONSOLIDATION SUMMARY

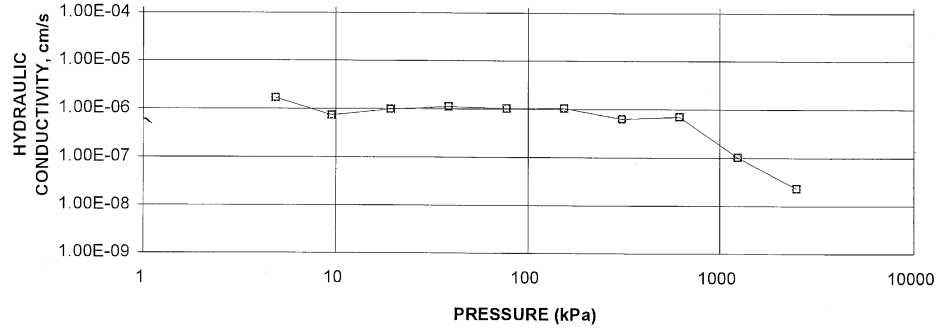
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 418-1A ST#3



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 418-1A ST#3



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 418-1A ST#3



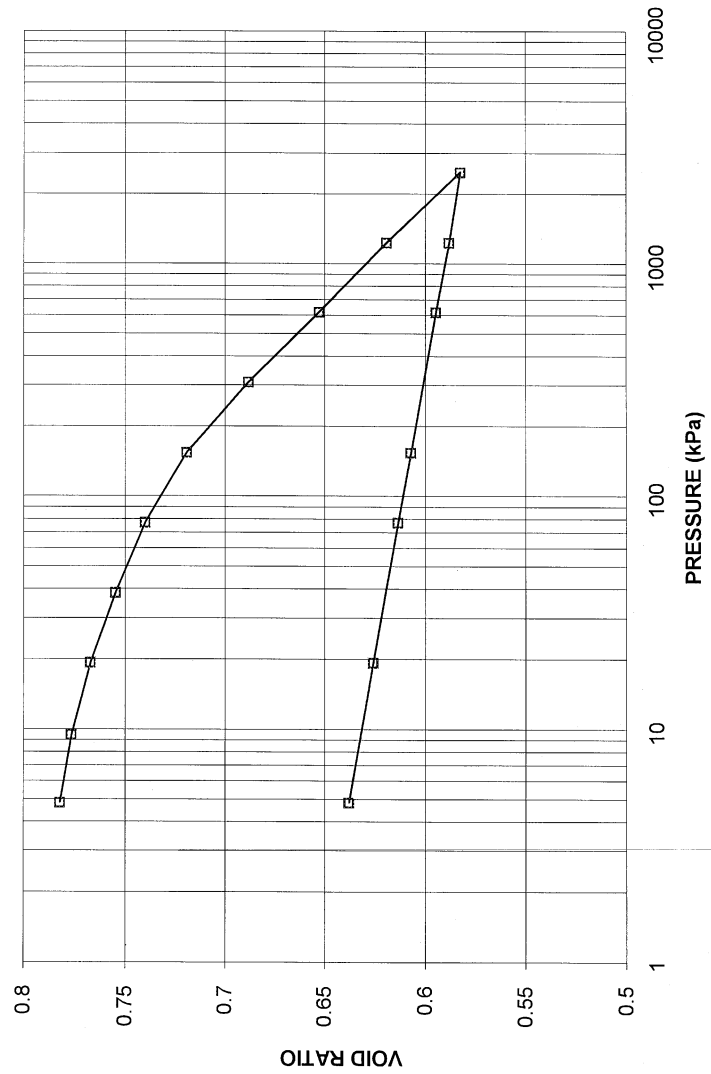
Project No. 04-1116-004

Golder Associates

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 418-1A ST#3

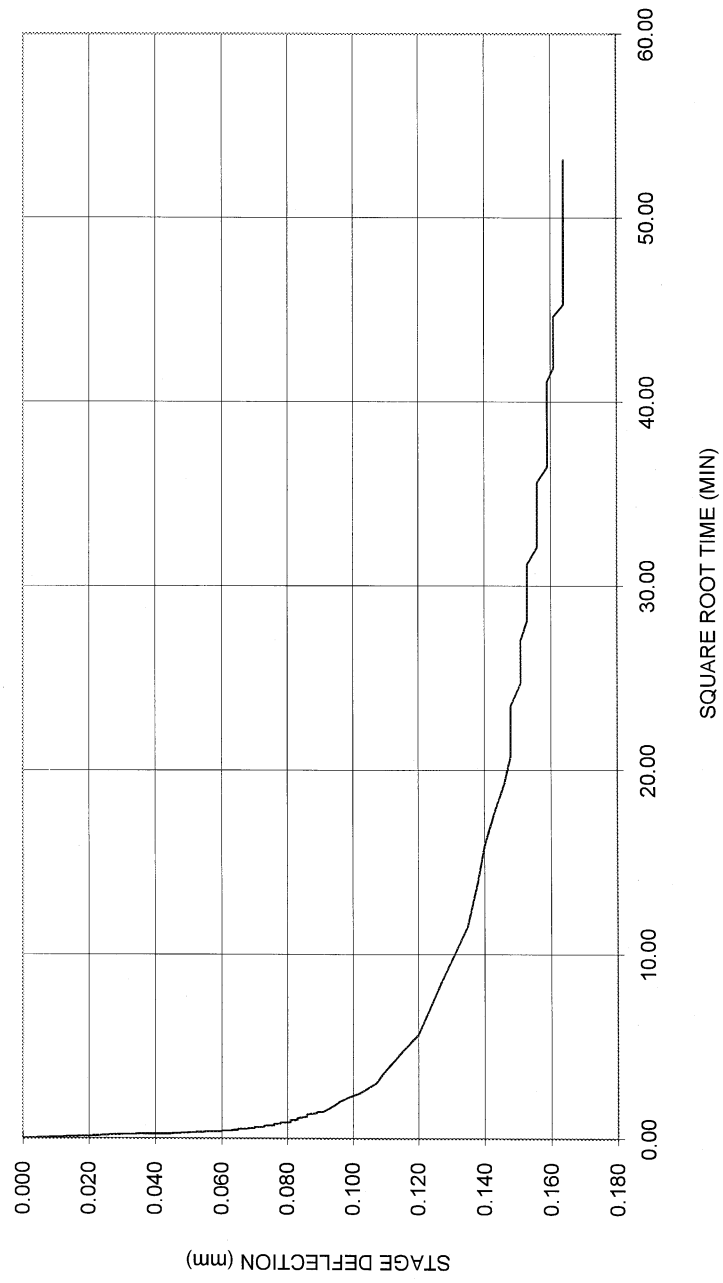


Project No. 04-1116-004

Golder Associates

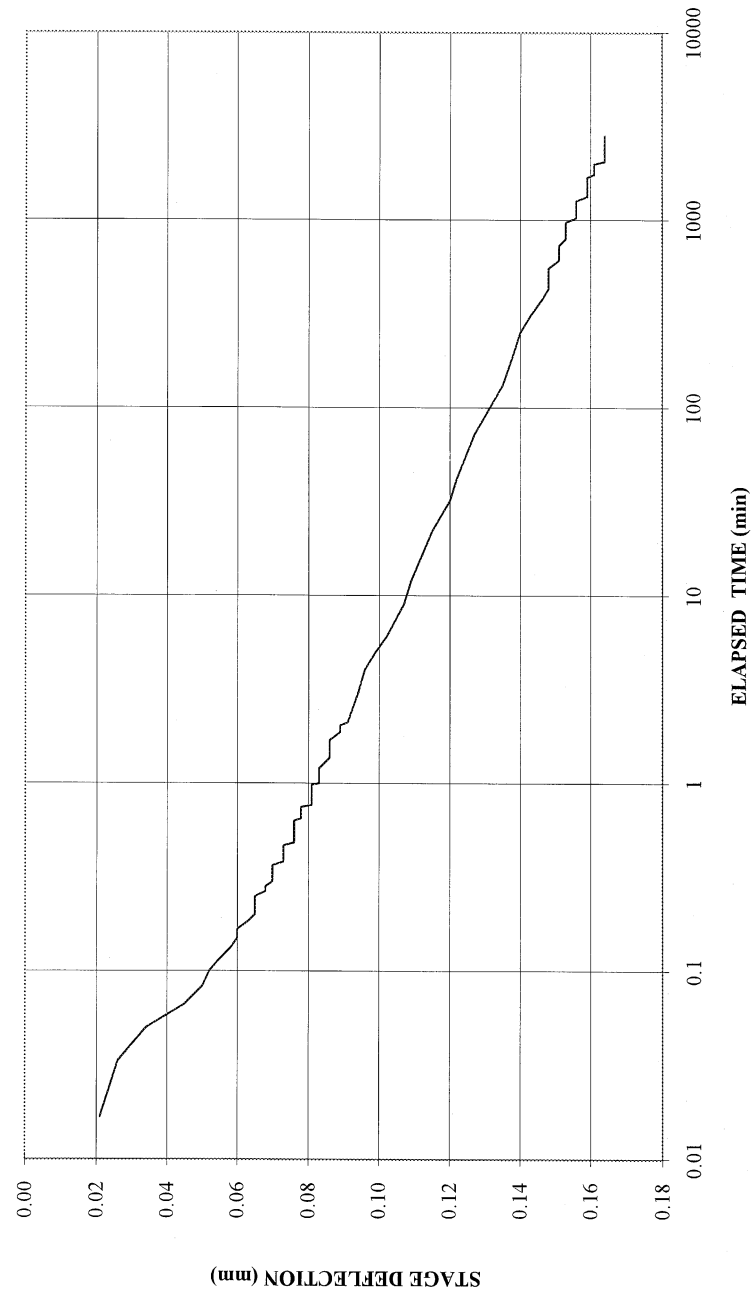
BOREHOLE 418-1A SAMPLE NUMBER ST#3

APPLIED PRESSURE = 77.4 kPa



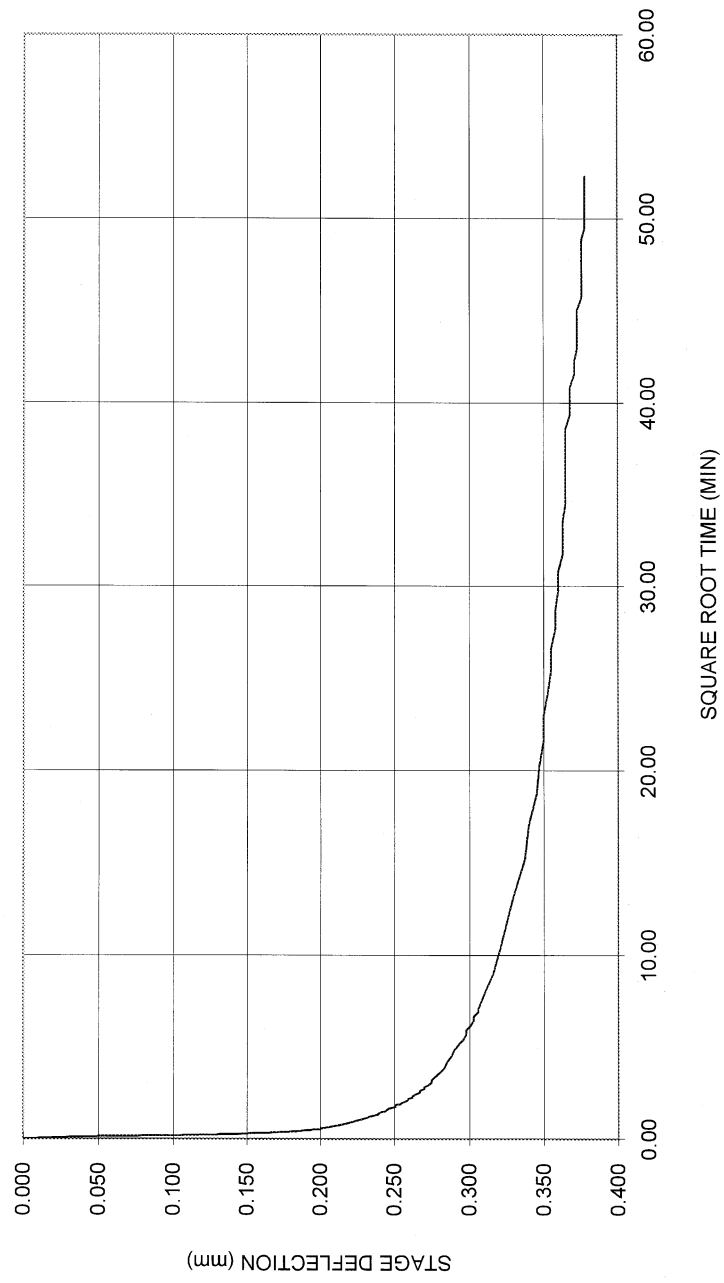
BOREHOLE 418-1A SAMPLE NUMBER ST#3

APPLIED PRESSURE = 77.4 kPa



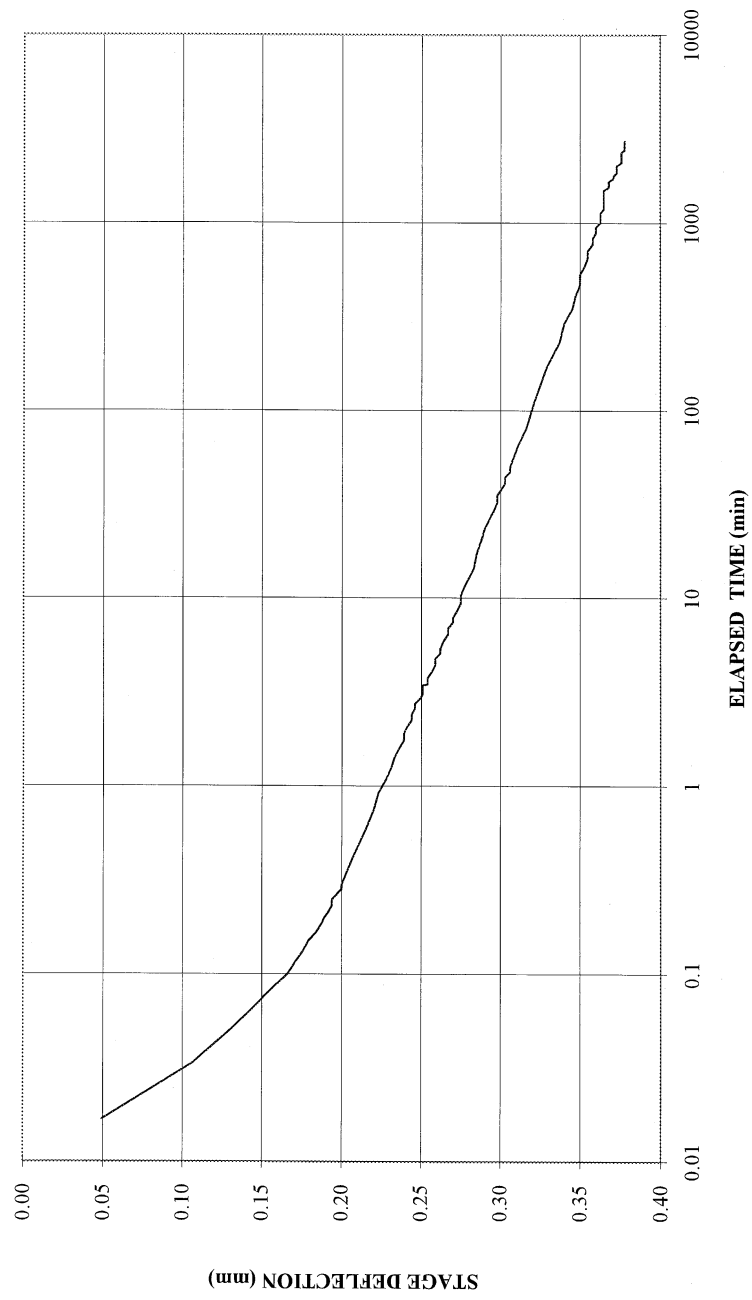
BOREHOLE 418-1A SAMPLE NUMBER ST#3

APPLIED PRESSURE = 618.4 kPa



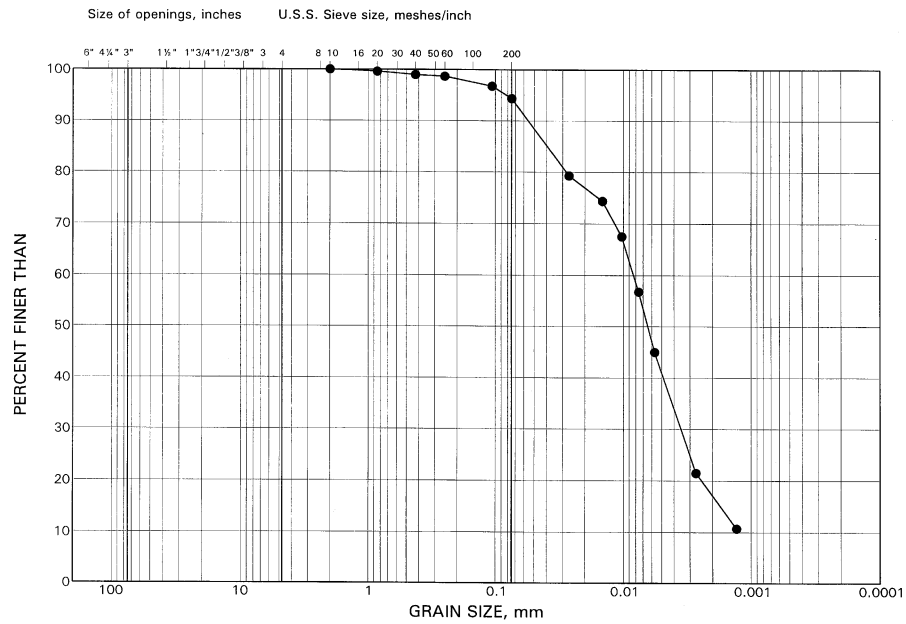
BOREHOLE 418-1A SAMPLE NUMBER ST#3

APPLIED PRESSURE = 618.4 kPa



GRAIN SIZE DISTRIBUTION

FIGURE



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	418-1A	ST#3	7.0-7.5

Project 941-116004

Golder Associates



SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	041-116004	
PROJECT NAME	Thurber / Lab Testing / 19-1423-12	
DATE TESTED	January, 2004	
Borehole	Sample	Specific
No.	No.	Gravity
418-1A	ST#3	2.73

Note: Test carried out on soil particles <4.75mm using distilled water.

Golder Associates



TABLE 1

SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER		04-1116-004			
PROJECT NAME		Thurber / Lab Testing / 19-1423-12			
DATE TESTED		January, 2004			
Borehole	Sample	Depth	Depth	Water	Atterberg Limits
No.	No.	(ft)	(m)	Content	LL, PL, PI
(%)					
418-1A	ST#3	23.0-24.7	7.01-7.53	29.5%	LL=26.1, PL=20.7, PI=5.4

Appendix C

Embankment Stability Analysis

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Hwy124 I/C
 December 2003
 West Approach Embankment - Side Slope
 Long Term - SSM - 21m wide; 6m high Berm - No Reinforcement

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
SSM Fill	22	0	30	0
Sand/Silt	20	0	32	0
Silty Clay Top	18	0	28	0
Silty Clay Mid	18	0	28	0
Silty Clay Bot	18	0	28	0
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

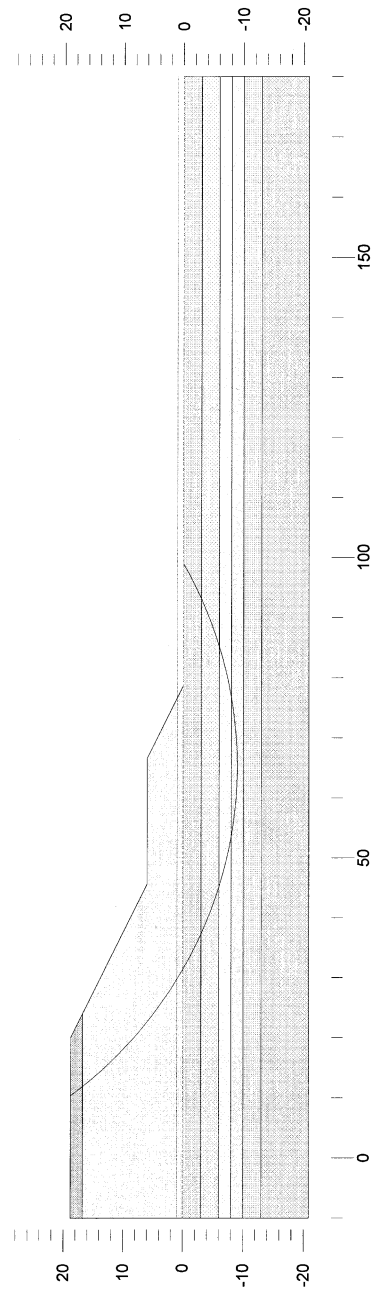
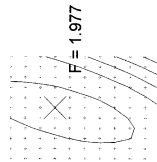


FIGURE 1A

25/02/2004 9:08:25 AM C:\JOBFILE\19\MM\1423-1-31\423-1-7\STABIL-1\124\1D7F2.GSL Thurber Engineering Ltd. - Toronto F = 1.977

Thurber Engineering Ltd. - Toronto				
19-1423-12				
Hwy11 - Burk's Falls - Hwy124 I/C				
December 2003				
West Approach Embankment - Side Slope				
Short Term - SSM - 21m wide; 6m high Berm - No Reinforcement				
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
SSM Fill	22	0	30	0
Sand/Silt	20	0	32	0
Silty Clay Top	18	70	0	.25
Silty Clay Mid	18	35	0	.25
Silty Clay Bot	18	55	0	.25
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

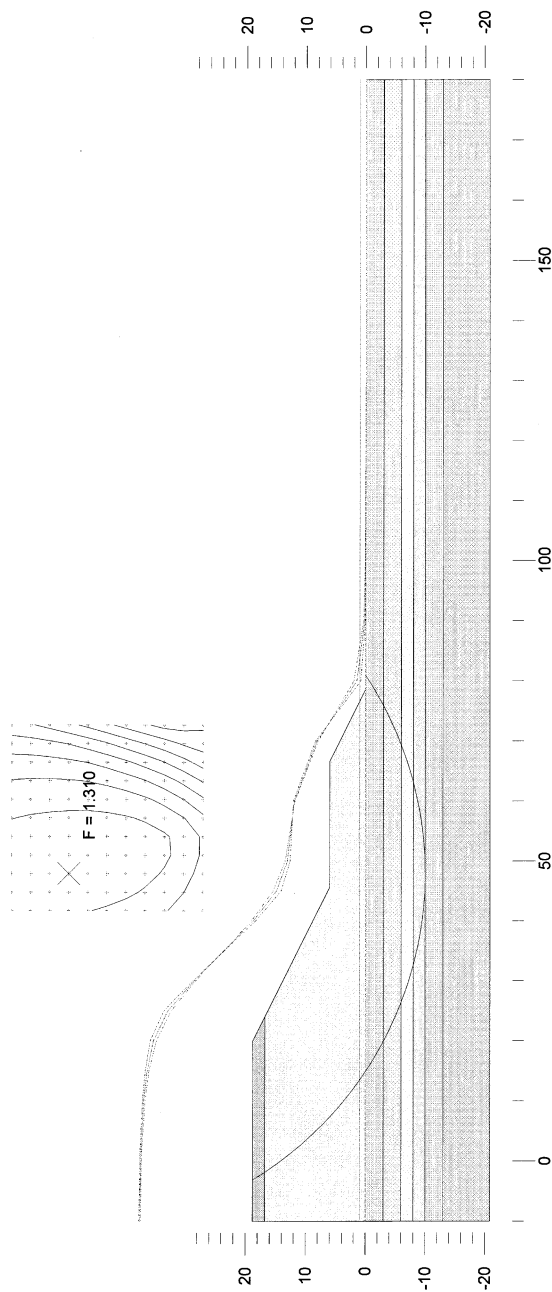


FIGURE 1B

25/02/2004 9:06:22 AM C:\JOB\FILE19\MM1423-1-31423-1-7\STABIL-11\241D7F1.GSL Thurber Engineering Ltd. - Toronto F = 1.310

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Hwy124 I/C
 December 2003
 East Approach Embankment - 4m wide; 3m high Berm - No Reinforcement
 Long Term - SSM

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
SSM Fill	22	0	30	0
Fine Sand	20	0	33	0
Silty Clay	18	0	28	0
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

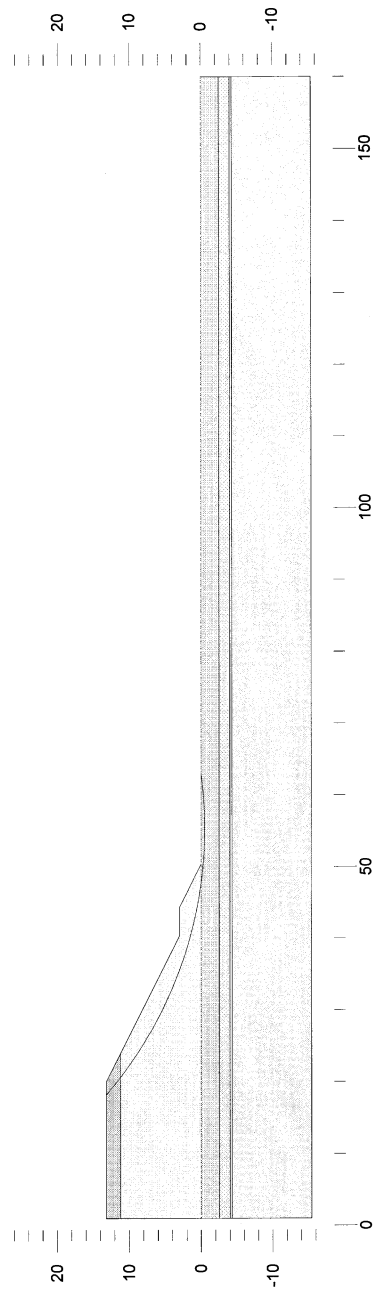
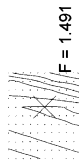
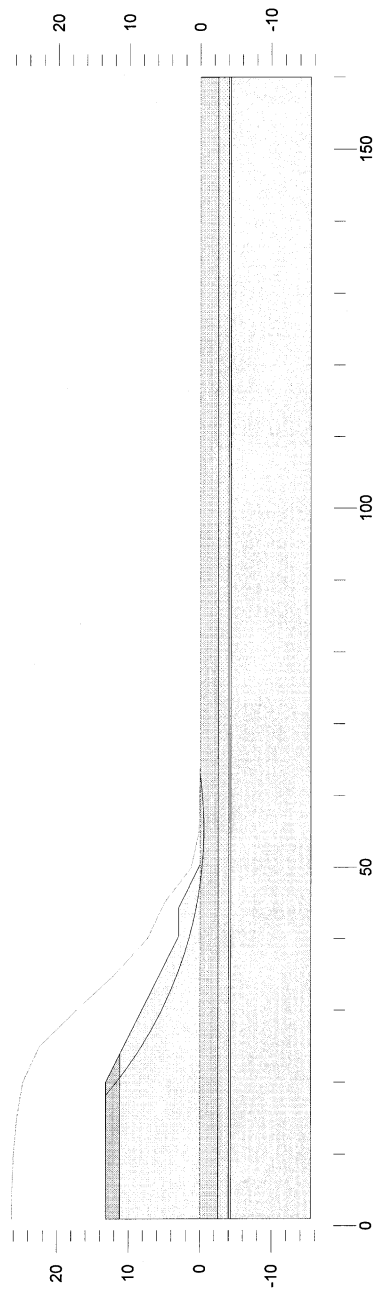
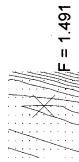


FIGURE 2A

25/02/2004 9:51:52 AM C:\JOBFILE\19\MM\1423-1~311423-1~7\STABIL~11243H2.GSL Thurber Engineering Ltd. - Toronto F = 1.491

	Gamma	C	Phi	Min	Piez
	kN/m ³	kPa	deg	c/p	Surf.
Surcharge	22	0	30	0	1
SSM Fill	22	0	30	0	1
Fine Sand	20	0	33	0	1
Silty Clay	18	80	0	.25	2
Sand	21	0	33	0	1
Hard Bottom	(Infinitely Strong)				



25/02/2004 9:46:22 AM C:\JOB\FILE19\MM1423-1-311423-1-7\NSTABIL-11124-3\42.GSL Thurber Engineering Ltd. - Toronto F = 1.491

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Hwy124 I/C
 December 2003
 West Approach Embankment - Rockfill
 Short Term - Rockfill - 4m high, 21m wide Berm - No Reinforcement

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
Rockfill	20	0	42	0
Sand/Silt	20	0	32	0
Silty Clay Top	18	70	0	.25
Silty Clay Mid	18	35	0	.25
Silty Clay Bot	18	55	0	.25
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

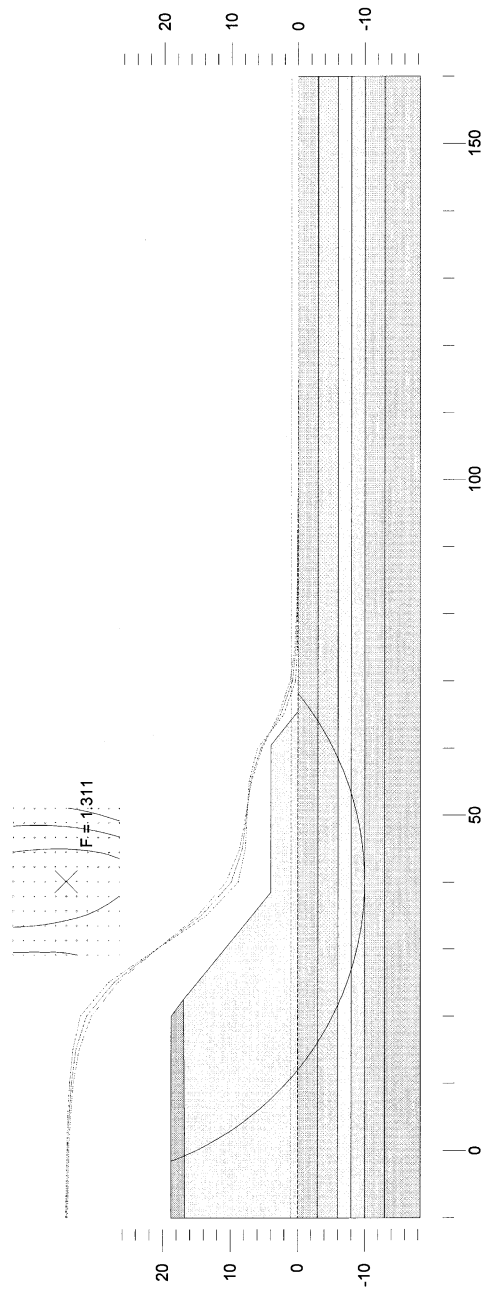


FIGURE 3B

25/02/2004 9:38:39 AM C:\JOB\FILE19\MM1423-1-311423-1-7\STABIL-111242D61.GSL Thurber Engineering Ltd. - Toronto F = 1.311



Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Hwy124 I/C
 December 2003
 West Approach Embankment - Rockfill
 LongTerm - Rockfill - 4m high, 21m wide Berm - No Reinforcement

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
Rockfill	20	0	42	0
Sand/Silt	20	0	32	0
Silty Clay Top	18	0	28	0
Silty Clay Mid	18	0	28	0
Silty Clay Bot	18	0	28	0
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

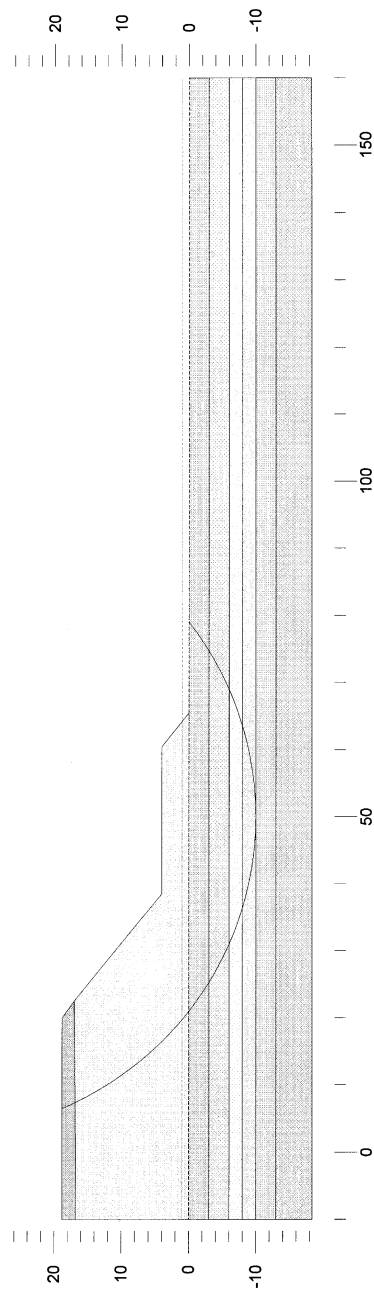
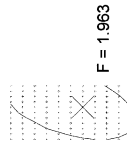


FIGURE 3A

25/02/2004 9:41:00 AM C:\JOB\FILE\19\MM\1423-1-31\1423-1-7\STABIL-1\1242D62.GSL Thurber Engineering Ltd. - Toronto F = 1.963

Thurber Engineering Ltd. - Toronto				
19-1423-12				
Hwy11 - Burk's Falls - Hwy124 I/C				
December 2003				
East Approach Embankment - 2m wide; 3m high berm - No Reinforcement				
Long Term - Rockfill				
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Surcharge	22	0	30	0
Rockfill	22	0	42	0
Fine Sand	20	0	32	0
Silty Clay	18	0	28	0
Sand	21	0	33	0
Hard Bottom	(Infinitely Strong)			

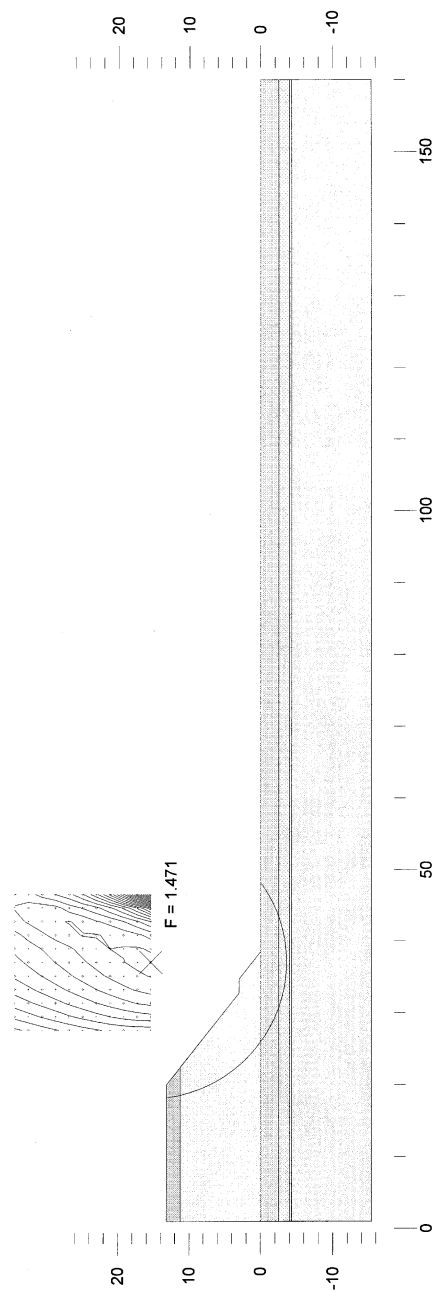


FIGURE 4A

25/02/2004 9:48:45 AM C:\JOBFILE\19\MM\1423-1-31\423-1-7\STABIL-1\124F2.GSL Thurber Engineering Ltd. - Toronto F = 1.471

Thurber Engineering Ltd. - Toronto

19-1423-12

Hwy11 - Burk's Falls - Hwy124 I/C

December 2003

East Approach Embankment - 2m wide; 3m high berm - No Reinforcement

Short Term - Rockfill

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Surcharge	22	0	30	0	1
Rockfill	22	0	42	0	1
Fine Sand	20	0	32	0	1
Silty Clay	18	80	0	.25	2
Sand	21	0	33	0	1
Hard Bottom	(Infinitely Strong)				

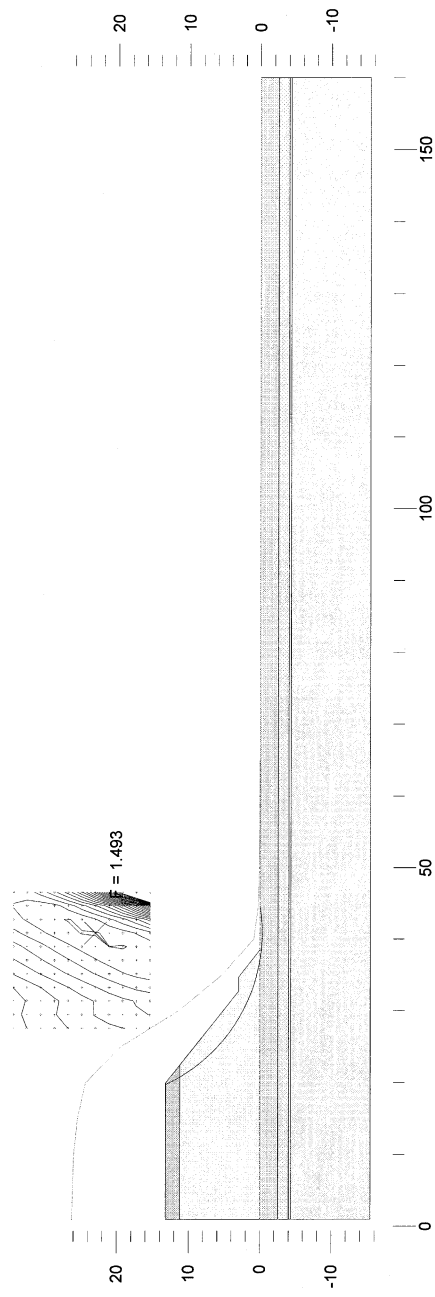


FIGURE 4B

25/02/2004 9:45:52 AM C:\JOBFILE\19\MM\1423-1-31423-1-7\STABIL-1124-4F2.GSL Thurber Engineering Ltd. - Toronto F = 1.493

Appendix D

Foundation Comparison

TABLE D1 - COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Spread Footing	Caissons
West Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles seating on bedrock • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • None identified <p>Disadvantages:</p> <ul style="list-style-type: none"> • Thick compressible soils with variable thickness across the footing • Potential for large absolute and differential long-term settlements 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity on bedrock <p>Disadvantages:</p> <ul style="list-style-type: none"> • Installation through cohesionless soils and high groundwater table: difficult installation and difficult quality control. • Higher cost than footings
Pier	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles seating on bedrock • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Depending on the foundation elevation, piles may have to be socketed into bedrock • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • None identified <p>Disadvantages:</p> <ul style="list-style-type: none"> • Relatively thick compressible soils with variable thickness across the footing • Potential for large absolute and differential long-term settlements 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity on bedrock <p>Disadvantages:</p> <ul style="list-style-type: none"> • Installation through cohesionless soils and high groundwater table: difficult installation and difficult quality control. • Higher cost than footings
East Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles seating on bedrock • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Depending on the foundation elevation, piles may have to be socketed into bedrock • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • Relatively high values of geotechnical resistance are available at on bedrock at 4m depth • Lower cost than piled foundation <p>Disadvantages:</p> <ul style="list-style-type: none"> • High groundwater table: dewatering will be required 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity on bedrock <p>Disadvantages:</p> <ul style="list-style-type: none"> • Installation through cohesionless soils and high groundwater table: difficult installation and difficult quality control. • Higher cost than footings

Appendix E

Special Provisions

AMENDMENT TO OPSS 206, DECEMBER 1993

Special Provision

November 25, 2002

OPSS 206, Construction Specification for Grading (December 1993) is amended as follows:

206.01 SCOPE

Section 206.01 of OPSS 206, December, 1993, is amended by the addition of the following:

Included in this specification are the requirements for the construction and compaction of rock fill embankments to minimize and control settlements within the rock fill.

206.04 SUBMISSION AND DESIGN REQUIREMENTS

OPSS 206, December, 1993, is amended by the addition of the following:

The Contractor shall submit to the Contract Administrator for record purposes a detailed construction procedure outlining the method and sequence of placement of rock fill and method of rock fill compaction. The equipment to be used shall be described in the submission. Where applicable, the submission shall describe modifications to be used during winter construction, to avoid the incorporation of frozen materials into the embankments.

206.06 EQUIPMENT

OPSS 206, December, 1993, is amended by the addition of the following

Equipment for compacting rock fill shall be of the vibratory, smooth, steel drum roller type with a minimum static drum weight of 8000 kg and minimum operating dynamic force of 150 kN.

206.07 CONSTRUCTION

Section 206.07 of OPSS 206, December, 1993, is amended as follows:

206.07.01.03 Compaction

The contents of subsection 206.07.01.03 are deleted and replaced with the following:

206.07.01.03.01 Compaction of Earth Embankments

Compaction of earth materials shall conform to OPSS 501.

206.07.01.03.02**Compaction of Rock Embankments**

Compaction of rock materials shall conform to the method and equipment requirements of this specification. Each rock fill layer shall be compacted with the equipment specified.. The minimum number of passes shall be four. Each roller pass shall overlap the edge of preceding passes by minimum 0.3 m. One hundred per cent roller pass coverages on the surface of each layer shall be provided

206.07.05 Rock Excavation, Grading**206.07.05.01 General**

The first paragraph in subsection 206.07.05.01 is deleted and replaced with the following:

The work to be done under the item Rock Excavation, Grading shall include drilling and blasting to obtain the required rock excavation and shatter, mucking, hauling and placing, handling, compacting and bringing to grade any excavation taken below grade. Whenever a rock face item is not included in the Contract, rock scaling and the removing of all overbreak and scaled materials and their incorporation into embankments shall be included in the rock excavation item.

206.07.08 Rock Embankments

Subsection 206.07.08 Rock Embankments is deleted and replaced with the following:

Construction of embankments using shale shall be carried out conforming to shale embankment requirements as specified in OPSS 206.07.08.01.

Embankments to be constructed of excavated rock other than shale shall be constructed by placing embankment materials full width in successive, uniform layers. Layers shall not exceed 1.5 m thickness prior to compaction. Material in each layer shall be fully compacted before the succeeding layer is placed.

Materials shall be placed in final position by blading. End dumping or depositing of rock over the end of any layer by hauling equipment is not permitted, except as otherwise noted below. Each layer shall be levelled in place and compacted to minimize voids and bridging of large rock fragments within the embankment.

Rocks exceeding 1 metre in size shall be well distributed throughout the embankment. Rock fragments up to a maximum size of 3 metres in size may be incorporated into the embankment provided that the rock fragments are less than two-thirds the remaining embankment height and are sufficiently spaced to allow free access of the specified equipment to compact the intervening fill. The remaining height shall be defined as the distance between the bottom of the oversized rock fragment at point of placement to the top of the rock fill embankment.

Placement in layers and compaction is not required for rock to be placed under water. Rock placed underwater may be placed by end dumping. End dumping shall only be used to an elevation of 1.0 m above the water level after which

rock embankments shall be constructed using the equipment and method specified in this special provision. The rock used for end dumping shall be deposited on the surface of the embankment and pushed forward by blading or dozing over the edge of the embankment.

The materials shall be well distributed to form a solid embankment constructed to full width as the work progresses, or as stage construction allows.

Where rock fill is placed in a wet area (such as swamps with full, partial or no excavation), the direction of the rock fill placement shall be such that mud waves generated by the rock fill placement would move away from the embankment. Mud waves shall be displaced or removed to prevent its entrapment below or within the embankment.

Voids on the top surface of the embankment shall be minimized to prevent migration of the roadway subbase and base into the rock fill embankment by chinking the top surface with rock fragments and spalls to form the subgrade prior to the placement of the roadway subbase. The Contractor shall chink the top surface of the embankment using rock material not exceeding 100 mm in size by conventional blading techniques.

Care shall be taken to avoid large boulders and rock fragments protruding above the average embankment surface within a distance of 3 m beyond the edge of the shoulder for future roadside safety.

Dumping over the sides of embankments is permitted only if the material is surplus to the contract requirements and after the rock embankments have been completed. Dumping over the sides of embankments shall be restricted to standard offset and right of way limits unless otherwise specified in the Contract Documents. The Contractor shall receive written approval from the Contract Administrator before commencing the above operations.

Suggested text to modify OPSS 501 for RSS construction.

501.08.02 Method A shall be replaced by the following:

501.08.02 Method A

Granular materials shall be compacted to 100% of the maximum dry density and all earth materials shall be compacted to 100% of the maximum dry density.

EARTH EXCAVATION FOR STRUCTURE - Item No.
ROCK EXCAVATION FOR STRUCTURE - Item No.
UNWATERING STRUCTURE EXCAVATION - Item No.
CLAY SEAL - Item No.

Special Provision No. 902S01

September 2003

Excavation and Backfilling-Structures

902.02 REFERENCES

Section 902.02 of OPSS 902, December, 1983, is amended by the addition of the following:

OPSS 510

902.03 DEFINITIONS

Section 902.03 of OPSS 902, December, 1983, is amended by the addition of the following:

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to excavation and backfilling of structures, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section 902.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

902.04.02 Working Drawings

Working drawings for protection systems shall be according to OPSS 539.

Where unwatering is required, the Contractor shall be responsible for the design of the unwatering scheme for the intended purpose. The design of temporary structures or protection system for unwatering shall be according to OPSS 539.

902.04.03 Submission of Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- Excavation for Foundation
- Excavation for Backfill and Frost Tapers
- Use of Excavated Material
- Unwatering of Excavation for Structure
- Backfilling

The Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

The Contractor shall be responsible for ensuring the quality of the material used for backfill. The quality of the material shall be verified by test results from a qualified and recognized testing laboratory. The frequency of sampling and testing shall be according to ASTM D75-87 and D3665.

902.05.04 Protection System

Section 902.05 of OPSS 902, December, 1983, is amended by the addition of the following:

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

Subsection 902.07.01 of OPSS 902, December, 1983, is amended by replacing the word "Engineer" in the last paragraph with the words "Contract Administrator".

902.07.02 Excavation

Subsection 902.07.02 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.07.02.01**General**

For excavation, the Contractor shall be responsible for preventing any deterioration of the foundation soil or rock, surface water from entering and eroding the face of the excavation, and build up of hydrostatic pressures which may have harmful effects upon the temporary or permanent structures.

902.07.02.02**Excavation for Foundation**

The excavation for foundation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

The Contractor shall be responsible for maintaining the stability of the excavation if any excavation below stream or channel bed is carried out.

The Contractor shall be responsible for restoring the over excavated area to its original conditions. For over excavation in earth, the backfill material shall be granular material such as Granular A or B compacted according to OPSS 501. For over excavation in rock, concrete shall be placed to achieve the original excavation limits. The concrete shall be of the same class concrete as the element it supports.

The Contractor shall be responsible for all additional costs due to excavation beyond the required tolerance limits, including but not limited to additional structure design, granular materials, concrete, reinforcing steel and retention of the services of a blasting consultant.

902.07.02.03**Excavation for Backfill and Frost Tapers**

Excavation for backfill and frost tapers shall be carried out according to the specifications and details shown on the contract drawings. The Contractor shall be responsible for restoring the over excavated portion with backfill and shall be compacted according to OPSS 501.

The excavation for backfill and frost tapers shall be inspected and approved by the Quality Verification Engineer prior to placement of fill material. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

902.07.02.04 Preservation of Channel

The Contractor shall be responsible for restoring the channel back to its original conditions unless otherwise specified in the contract.

902.07.02.05 Removals

Removal of pavement, curb and gutter, and sidewalks shall be according to OPSS 510.

902.07.03 Unwatering Structure Excavation

Subsection 902.07.03 of OPSS 902, December, 1983, is amended by replacing the first paragraph with the follows:

The Contractor shall carry out all work necessary to prevent disturbance to the founding material. Concrete shall be placed in the dry, unless otherwise specified in the contract.

After the unwatering, the excavation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

902.07.04 Backfilling

Subsection 902.07.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

The Contractor shall ensure that the concrete has reached at least 70 percent of its design strength before placing the backfill against an abutment, wingwall, retaining wall or concrete culvert.

Backfilling shall be according to OPSS 501.

The backfilling operation shall be inspected and approved by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

902.09 Measurement for Payment

902.09.01 Structures

Subsection 902.09.01 of OPSS 902, is amended by deleting the first five paragraphs and replacing them with the following:

"Earth Excavation for Structure" and "Rock Excavation for Structure" applies to the specific structure(s) designated, i.e., Bridge, Retaining Wall or Concrete Culvert, and is measured by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the volume in cubic metres below the designated payment surface.

The above measurement also includes, where applicable, the excavation quantities, below the designated payment surface, for placing granular backfill and for placing the granular frost tapers.

For open footing culverts, the above measurement also includes the excavation quantities below the designated payment surface but between the plan areas of the footings and above the stream bed or the top of the footings, whichever is higher.

Where the structure excavation overlaps excavation required for other work, deductions will not be made to the structure excavation measurement.

902.10 Basis of Payment

902.10.01 Excavation and Backfill

Subsection 902.10.01 of OPSS 902 is amended by deleting the first paragraph and replacing it with the following:

Payment at the contract price(s) for the tender item(s) "Earth Excavation for Structure" and "Rock Excavation for Structure" shall be full compensation for all labour, equipment and material for all excavation required, for removal of pavement, curb and gutter and sidewalk except where there is a separate item for removal of pavement, curb and gutter and sidewalk which overlaps pavement, curb and gutter and sidewalk removal required for structure excavation, protection of adjacent works, unwatering, backfilling and compacting around the footing according to subsection 902.07.04, placing and compacting of suitable material in fill in accordance with OPSS 206 and management of any surplus or unsuitable excavated material, including the cost of disposal areas, all according to the requirements of this specification.

WARRANT: Always with these tender items.

SUPPLY EQUIPMENT FOR DRIVING PILES - Item No.
SUPPLY EQUIPMENT FOR INSTALLING CAISSON PILES - Item No.
SUPPLY EQUIPMENT FOR INSTALLING DISPLACEMENT CAISSON PILES - Item No.
SHEET PILES - Item No.
H-PILES - Item No.
TUBE PILES - Item No.
WOOD PILES - Item No.
PRECAST CONCRETE PILES - Item No.
CAISSON PILES - Item No.
DISPLACEMENT CAISSON PILES - Item No.
DRIVING SHOES - Item No.
ROCK POINTS. - Item No.
RETAPPING PILES – Item No.

Special Provision No. 903S01

October, 2002

13.1.1.1 Piling

OPSS 903, December 1983, is deleted and replaced with the following:

903.01 SCOPE

This specification covers the requirements for the supply and installation of deep foundation units comprised of wood, steel, concrete or a combination of these materials.

903.02 REFERENCES

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Concrete
OPSS 905 Steel Reinforcement
OPSS 909 Prestressed Concrete - Precast
OPSS 911 Coating Structural Steel Construction

Ontario Provincial Standard Specifications, Material:

OPSS 1302 Water
OPSS 1350 Concrete (Materials and Production)
OPSS 1440 Steel Reinforcement for Concrete

Canadian Standards Association Standards:

CAN/CSA 3-G40.20/G40.21-M92 - General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Sheets

CAN3-056-M79 - Round Wood Piles

CSA 080 Series-M97 - Wood Preservation

W47.1-92 - Certification of Companies for Fusion Welding of Steel Structures

W48.1 - M1991 - Carbon Steel Covered Electrodes for Shielded Metal Arc Welding

W59 - M1989 - Welded Steel Construction (Metal Arc Welding)

American Society for Testing and Materials Standards:

ASTM A 252-93 Welded and Seamless Steel Pipe Piles

ASTM A 328/ A 328M-93A Steel Sheet Piling

American Petroleum Institute:

API 13A-86 Oil Well Drilling Fluid Materials

API 13B Standard Procedures for Field Testing Drilling Fluids

903.03 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Anvil: means the component of a diesel hammer that acts as an impact block for the ram

Bedrock: means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic or sedimentary in origin which may or may not be weathered. The actual surface of the bedrock, weathered or unweathered, exists immediately below the overburden.

Casing: means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground with caisson piles that is structurally required and can be used to render a stable excavation hole.

Caisson Pile: means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Cap Block: means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.

Deep Foundation Unit: means a structural member, driven or otherwise installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Displacement Caisson Pile: means a pile formed in the ground by driving a casing or liner by means of a concrete plug or an expendable metal plate and replacing the displaced soil with plain or reinforced concrete.

Driving Shoe: means a reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate into a hard stratum.

Driving to a Set: means driving the pile to a penetration that satisfies pile driving criteria correlated to a required pile resistance

Follower: means a removable extension which transmits the hammer blows to the head of the pile.

Helmet: means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

Jetting: means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

Liner: means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

Pile: means a relatively slender structural element which is installed, wholly or partly in the ground by driving, drilling, auguring, jetting or other means.

Pile Cap: means a footing or some other structural component used to transfer the load to the piles as well as maintaining them in position.

Pile Cushion: means a pad of resilient material placed between the helmet and the top of a reinforced concrete or timber pile to minimize damage to the head during driving.

Pile Group: means the piles supporting a pile cap.

Pumped Concrete: means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Quality Verification Engineer(QVE): means an Engineer who has a minimum of five (5) years experience in the field of installation of piling or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificate(s) of Conformance.

Retapping: means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration.

Rock Points: means a specially designed steel tip, fitted to piles to enable them to be driven into hard, sound sloped bedrock.

Sheet Pile: means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage.

Slurry: means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

Stamped: means drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer.

Tremie: means a hopper with a vertical pipe leading out of the bottom of it, used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete is always above water level.

903.04 SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer experienced in this field. This Engineer, under this section, will not be permitted to carry out the work of the Quality Verification Engineer.

The Contractor shall submit to the Quality Verification Engineer for review and stamping, the equipment and installation procedure and the procedure for monitoring installation.

903.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the CA, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

903.04.02 Materials

903.04.02.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

903.04.02.02 Concrete

Concrete and concrete work shall conform to OPSS 1350 and OPSS 904. The Contractor shall submit a suitable, site specific concrete mix design that meets the requirements of the hardened concrete specified. The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non segregating concrete that does not exhibit rapid slump loss. The concrete mix design shall be submitted to the Contract Administrator for information purposes only, one(1) week prior to construction.

903.04.02.03 Slurry

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

The type, source, physical and chemical properties of the bentonite or polymer.
Slurry mix proportions and procedure.
Quality Control Plan to control properties of slurry mix.
Method of disposal.

903.04.03 Installation

903.04.03.01 Driven Piles

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

Type of equipment and hammer details including Contractors stated potential energy(rated energy) of the hammer, operating efficiency, weight of ram, anvil and helmet.

Procedure including sequence for pile installation.
Procedure for monitoring installation

903.04.03.02 Caisson Piles

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

Shop drawings that describe and illustrate equipment, materials.
Procedure for caisson excavation and construction.
Procedure for monitoring installation and caisson inspection.

903.04.03.03 Displacement Caisson Piles

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

1. Equipment to be used for installation.
2. Procedure for installation
3. Procedure for monitoring installation.

903.04.03.04 Certificate of Conformance

Upon completion of the deep foundation work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The certificate shall state that the work has been carried out in general conformance with the contract documents, specifications and stamped working drawings.

903.05 MATERIAL

903.05.01 Wood Piles

Wood piles shall be according to CSA CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CSA 080.

Wood piles shall not be spliced.

903.05.02 Steel Piles

903.05.02.01 Steel H Piles

Steel H piles shall be according to CSA G40.20/G40.21 and shall be 350 W grade.

903.05.02.02 Steel Tube Piles

Steel tube piles shall be according to ASTM A252 minimum Grade 2.

903.05.02.03 Steel Sheet Piles

Steel sheet piles shall be according to ASTM A328. Steel sheet piles shall not be spliced.

903.05.02.04 Straightness Tolerance for Steel Piles

All steel piles shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.

903.05.03 Driving Shoes and Rock Points

Rock points and driving shoes shall be as specified. Driving shoes shall transfer the driving stresses to the pile over the full cross-sectional area of the pile.

Where the contract shows details of “Splice and Driving Shoe Details for Steel ‘H’ Piles, the Contractor may substitute the Titus “H” Bearing Pile Point, Standard model, in place of the driving shoe details shown.

Where the contract shows details of “Oslo Points for HP310 H-Piles” the Contractor may substitute the Titus “H” Bearing Pile Point, Rock Injector model in place of the pile point details shown.

Welding of Titus Points shall conform to the manufacturer’s specifications.

Where the Contractor elects to use any of the above substitutions, the cost shall be deemed to be included in the contract price for the appropriate item.

903.05.04 Casing for Caissons

Casings shall be according to ASTM A252 Grade 2. If welded they shall be welded by the electric arc method according to CSA W59.

The wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.

903.05.05 Steel Reinforcement

Steel reinforcement shall be according to OPSS 1440.

903.05.06 Concrete

903.05.06.01 General

Concrete shall be according to OPSS 1350.

903.05.06.02 Tube Piles

Concrete shall have a slump of 150 to 180 mm.

903.05.08.03 Caisson Piles

Concrete shall have a slump of 150 to 180 mm. When approved by the Contract Administrator in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal, and to prevent setting of concrete until after the liner is withdrawn.

903.05.07 Slurry

903.05.07.01 Solids

Bentonite and polymers shall be according to API 13A.

903.05.07.02 Slurry Composition

Slurry shall be according to API 13B

903.05.08 Helmets and Striker Plates

The head of piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

903.06 EQUIPMENT

The hammers shall be capable of driving the piles and liners/casings to the prescribed depth or to the specified resistance without damage to portions that are not cut off.

903.07 CONSTRUCTION

903.07.01 Subsurface Conditions

A Foundation Investigation Report that describes the subsurface conditions for the project is available, as specified elsewhere in the Contract. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

Any interpretation of data or opinions expressed in the report are not warranted. Regarding the data presented in the report, although the raw measured data presented is warranted, the Contractor must satisfy itself as to the sufficiency of the information presented for the intended construction purpose and obtain any updating or additional information as required to facilitate the deep foundation works.

903.07.02 Transportation, Handling, Storage

Piles, casings and reinforcing steel cages shall be transported, stored and handled in such a manner that damage and distortion is prevented and that the strength and integrity are maintained.

903.07.03 Driven Piles

903.07.03.01 Pile Driving Requirements and Restrictions

Piles shall be installed at the locations indicated and to the set or depth specified without being damaged.

Damage to adjacent structures, utilities and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 7.5 m of concrete which has been in place for less than 72 hours.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated on the Contract Drawings.

Piles shall not be forced into their proper position by the use of excessive manipulation. Pile damage due to excessive driving shall be avoided.

903.07.03.02 Splicing

903.07.03.02.01 General

Splices within 6 m of the pile cut-off shall be certified by the Quality Verification Engineer as being equal to the full strength of the pile. Any damaged material shall be cut-off prior to splicing. The certificate shall be sealed and signed by the Quality Verification Engineer and shall be submitted to the Contract Administrator.

903.07.03.02.02 H Piles, Tube Piles and Sheet Piles

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H piles and steel tube piles may be spliced providing the pieces being spliced are not less than 3 m long. Splices in marine structures shall be located below the low water level unless otherwise encased in concrete.

Sheet piles shall not be spliced without approval by the Contract Administrator.

903.07.03.02.03 Precast Piles

Precast piles shall only be spliced when specified and the splices shall only be made with approved mechanical splicing devices.

903.07.03.03 Concrete in Steel Tube Piles

Concrete in steel tube piles shall be placed according to the OPSS 904 requirements.

903.07.03.04 Cutting Off Piles

903.07.03.04.01 General

Driven piles shall be cut to the elevation as specified in the contract.

The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.

903.07.03.04.02 Wood Piles

Where wood piles are broomed, splintered or otherwise damaged below the cutoff elevation, the pile shall be considered defective and shall be replaced.

903.07.03.05 Protective Coating for Steel H and Steel Tube Piles

Exposed steel H and steel tube piles shall have a protective coating applied from an elevation 600 mm below the low water level or finished ground surface up to the top of the exposed steel.

The steel surfaces shall be cleaned according to SSPC-SP10 prior to application of a coal tar epoxy system which shall be according to OPSS 911.

903.07.03.06 Reinforcing Steel

Reinforcing Steel shall be installed according to OPSS 905.

The reinforcing steel cage shall be fabricated in one piece.

Welding of reinforcing steel and use of splices shall not be done unless specified in the contract.

903.07.04 Caisson Piles

903.07.04.01 Installation - General

Caissons shall be constructed as specified in the contract.

The final bearing elevation shall be as specified in the contract or shall be an elevation determined by the Contract Administrator. When permanent casings are not specified the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.

903.07.04.02 Excavation

Sidewall stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.

Excavation methods shall be such that the sides and bottoms of the hole are straight and free of loose material.

Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped with each step not greater than 3 the diameter of the bearing area.

903.07.04.03 Unwatering

Where unwatering is required, the Contractor shall effect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material, or prevent subsidence or ground loss that may adversely affect the work of adjacent structures.

903.07.04.04 Backfilling Liners Left in Place

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

903.07.04.06 Concrete

903.07.04.06.01 General

Concrete shall be placed in the caisson according to OPSS 904. Concrete shall be placed immediately following acceptance of the caisson hole by the QVE.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

Arching of concrete during casing withdrawal shall be prevented.

The QVE shall provide inspection throughout the concreting operation.

903.07.04.06.02 Concrete Placed in the Dry

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson, or where columns are cast integral with the caisson, to the elevation of the bottom of the column reinforcing cage. The concrete shall be vibrated for the last 1.5 m of the pour.

903.07.04.06.03 Concrete Placed Under Water or Under Slurry

Tremie or pumped concrete shall be carried out in one continuous operation. The Contractor shall carry out the tremie or pumping operation to ensure a continuous flow of concrete that prevents the inflow of water or slurry.

903.07.04.07 Reinforcing Steel

The reinforcing steel cage shall be checked to ensure conformance to the approved shop drawings prior to installation and during concrete placement.

903.07.05 Displacement Caisson Piles

903.07.05.01 General

Work shall be carried out in accordance with displacement caisson pile suppliers installation procedures. A permanent liner shall be used when specified.

The pile shall not be extended below the specified pile tip elevation without approval in writing from the Contract Administrator.

903.07.06 Tolerances

903.07.06.01 Driven Piles

cut off ∇ 25 mm

deviation from vertical not more than 1 in 50, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1 in 75 in the direction of the span

the deviation from the specified inclination for battered piles shall not exceed 1 in 25

the centre of the pile at the junction with the pile cap shall be within 150 mm of that specified (measured horizontally) except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm(measured horizontally) in the direction of the span.

903.07.06.02 Caissons

Cut off elevation ∇ 25 mm

Horizontal location at cut-off not more than 5% of shaft diameter nor 75 mm

Vertical alignment not more than 2% of the caisson length from vertical for

vertical caissons, nor 2% of the caisson length from the specified inclination for battered caissons

903.08 QUALITY CONTROL

903.08.01 Monitoring Driven Piles

903.08.01.01 General

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile. All driving records shall be certified by the Quality Verification Engineer and submitted to the Contract Administrator.

903.08.01.02 Driving to a Set

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group.

The other piles shall be controlled by the pile penetration rate in blows per mm that correlates to the set.

When new conditions such as change in hammer size, change in pile size or change in soil material occur, new sets shall be determined.

903.08.01.03 Driving to Bedrock

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

903.08.01.04 Hammer Performance

When requested by the Contract Administrator, the Contractor shall verify the hammer performance using the Pile Driving Analyzer or other approved equivalent. The Contractor shall provide all instrumentation, related access and assistance for the testing and monitoring as directed by the Contract Administrator.

Hammer performance shall be verified to ensure that the actual potential energy is not less than 90% of the stated potential energy.

903.08.01.05 Retapping Tests on Piles

In each pile group, 10% of the piles (actual number of piles to be rounded off to higher number) but no fewer than two piles shall be retapped no sooner than 24 hours *after installation of the individual pile* to confirm the bearing resistance has been sustained.

Retapping of piles driven to bedrock is not required.

903.08.01.06 Retapping/Redriving Piles

Where the retapping tests indicate the bearing resistance has not been sustained, all piles in the group shall be retapped.

Where the retapping reveals that the bearing resistance of the piles has not been achieved, the piles shall be redriven to the specified resistance. Where piles have risen, the piles shall be redriven to the original depth.

903.08.02 Inspection of Caisson Holes

The caisson holes shall be inspected and approved by the QVE.

903.09 MEASUREMENT FOR PAYMENT

903.09.01 H Piles, Tube Piles, Wood Piles and Precast Concrete Piles

Measurement is in metres of the piling left in place after cut-off.

903.09.02 Sheet Piles

Measurement is in square metres based on the driving lines specified and the length of piling left in place after cut-off.

903.09.03 Driving Shoes and Rock Points

Measurement is for each driving shoe and rock point specified and used.

903.09.04 Caissons and Displacement Caisson Piles

Measurement is in metres of the depth along the centre line between the approved bearing surface at the bottom and the specified elevation at the top.

903.09.05 Retapping Piles

Measurement is lump sum for retapping the piles above and beyond the minimum 10% but no fewer than two piles requirement for the pile group.

For measurement purposes a count will be made of the number of piles retapped above and beyond the minimum 10% but no fewer than two piles requirement and the number of piles in the pile group and a ratio will be determined.

Where retapping is not required above and beyond the minimum, no measurement for payment will be made for this item.

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item Supply Equipment for Installing Caisson Piles - Item Supply Equipment for Installing Displacement Caisson Piles - Item

Payment at the contract price for the above items shall be full compensation for all labour, testing, equipment and material required to do the work.

It will be assumed, for payment purposes, that 50% of the work under this item has been completed when the satisfactory performance of the equipment has been demonstrated to the Contractor Administrator by the installation of one(1) pile. The remaining 50% will be paid on the satisfactory completion of the installation.

When the hammer performance is requested to be verified, all costs associated with this work will be included in the contract price when the energy delivered is less than 90% of the stated potential energy(rated energy) specified in the submission.

When the energy is greater than 90% of the stated potential energy(rated energy) stated in the required submission, the cost will be paid as extra work.

- 903.10.02 H-Piles – Item**
Tube Piles – Item
Precast Concrete Piles - Item
Wood Piles - Item
Displacement Caisson Pile - Item
Caisson Piles - Item
Driving Shoes - Item
Rock Points - Item
Sheet Piles - Item

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material to do the work

Payment for redriving piles shall be at the contract price for the applicable item(s) above.

903.10.03 Retapping Piles – Item

Payment for retapping the minimum specified number of piles is included in the Pile Item. Where additional retapping is required, payment will be made based on the ratio of the number of piles retapped in a pile group above the minimum requirement, to the total number of piles in that pile group, times the tender price for retapping all piles for that pile group.

WARRANT: Always with these tender items.

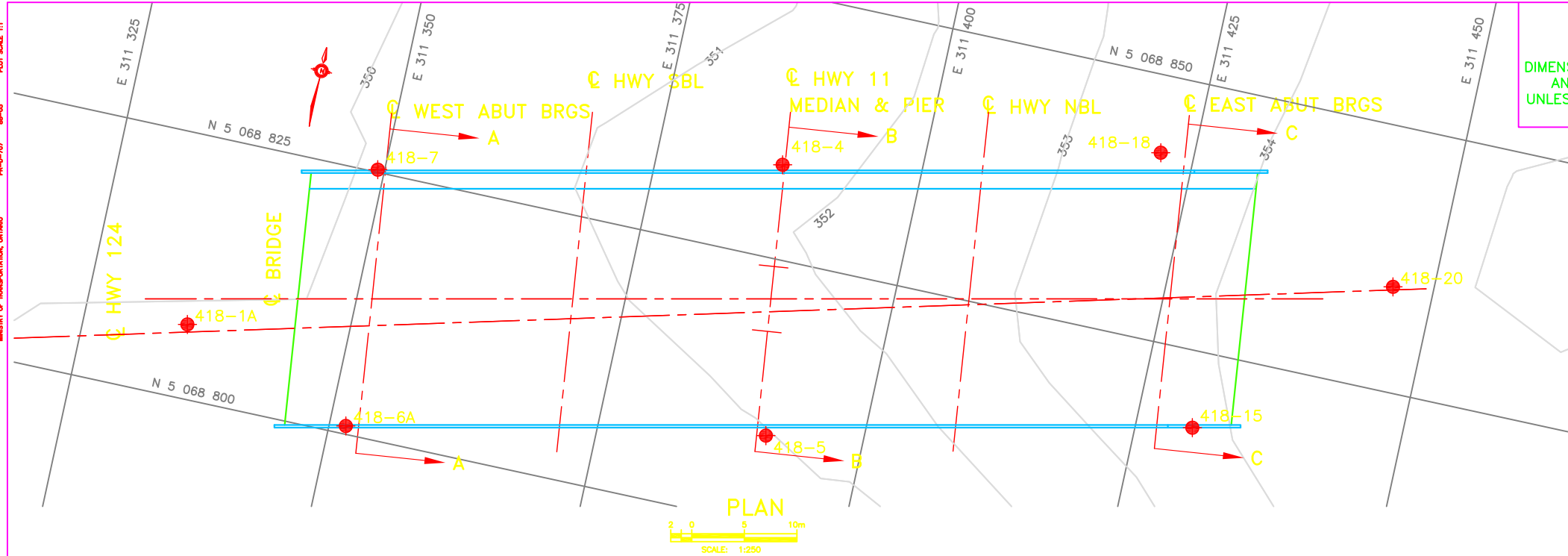
Appendix F

Drawings

MINISTRY OF TRANSPORTATION, ONTARIO
PM-0-707 88-05
PLOT SCALE 1:1

MODIFIED:

DRAWING NAME:
CREATED:



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY 11
CONT No
WP No 5039-03-01



HWY 124 I/C
UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

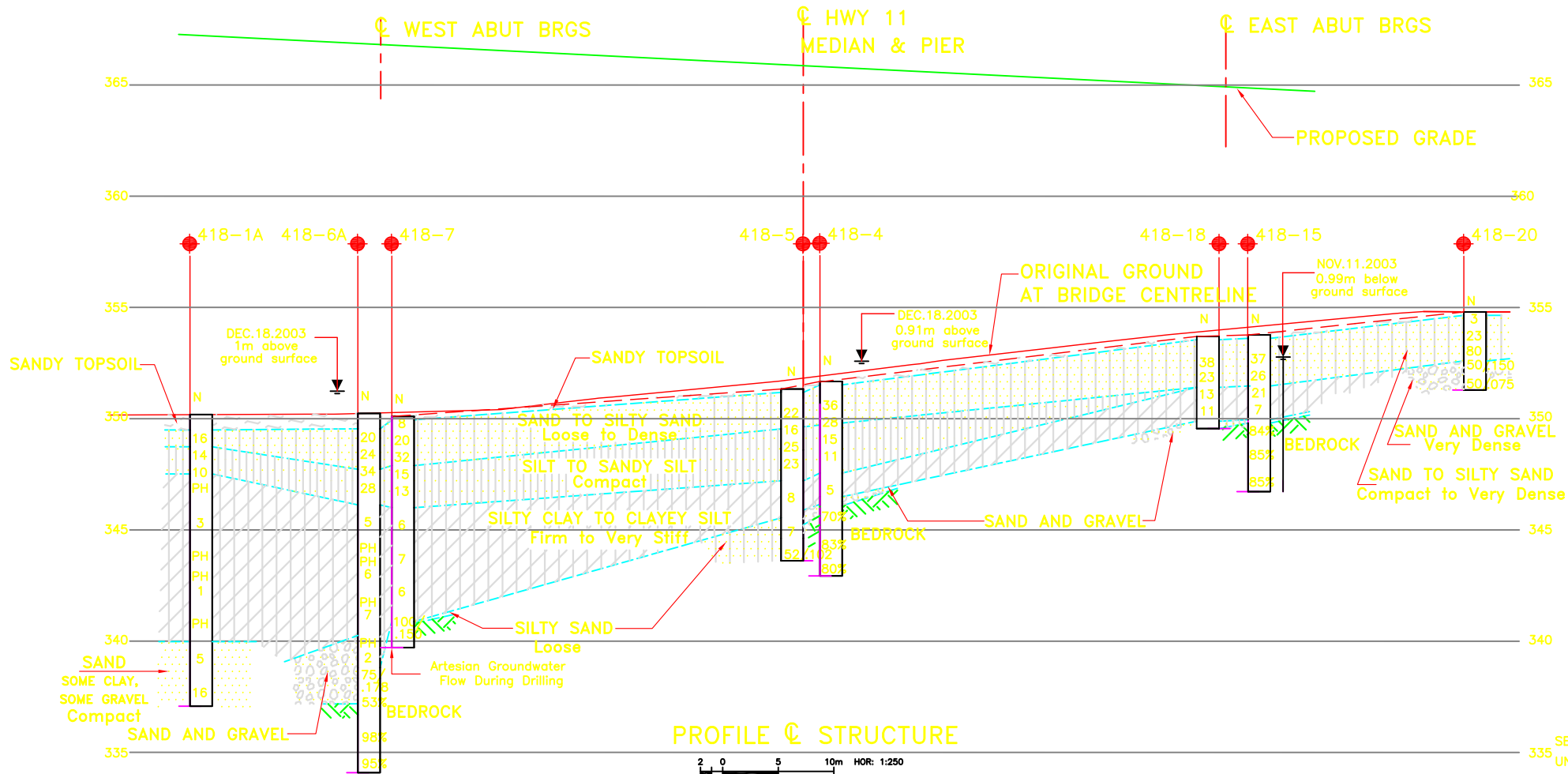
LEGEND

- BoreHole by THURBER
- Dynamic Cone penetration Test (cone)
- BoreHole and Cone
- N Blow/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
418-1A	350.0	5 068 807.0	311 334.8
418-4	351.6	5 068 833.9	311 386.9
418-5	351.3	5 068 808.4	311 390.8
418-6A	350.3	5 068 800.8	311 351.6
418-7	350.1	5 068 825.3	311 349.4
418-15	353.8	5 068 817.8	311 430.4
418-18	353.7	5 068 842.7	311 421.8
418-20	354.8	5 068 834.9	311 446.2

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.



SEE SHEET "HWY 124 I/C
UNDERPASS-SOIL STRATA"
FOR CROSS-SECTIONS
A-A, B-B, C-C
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	PJB	CHK	PK
DRAWN	SS	CHK	AE
DATE	BY	DESCRIPTION	
2004	FEB	2004	
44-418	STRUCT	SCHEME	DWG

BENCH MARK
DHO BM 351-67 Tablet Set
horizontally in SE corner of
church 196.86 RT of 20+202.6
BM Elev. 375.305

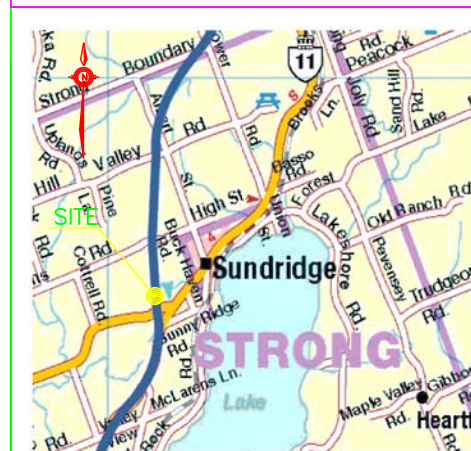
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY 124 I/C
UNDERPASS
SOIL STRATA

SHEET









PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS



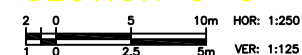
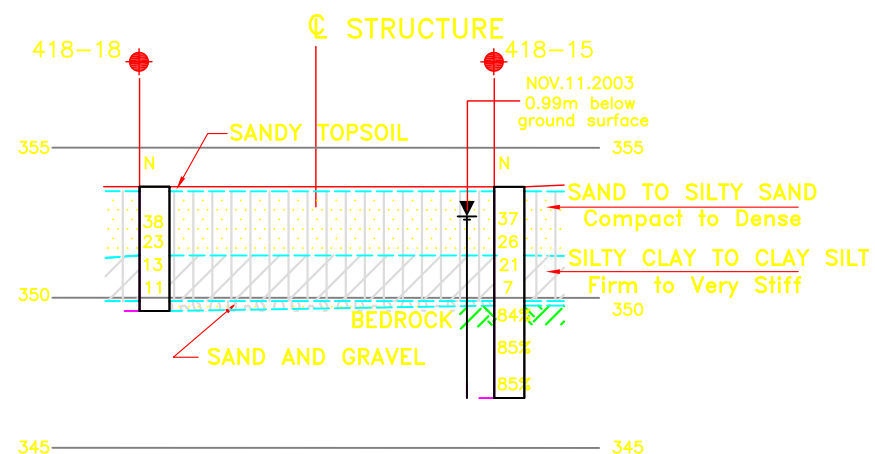
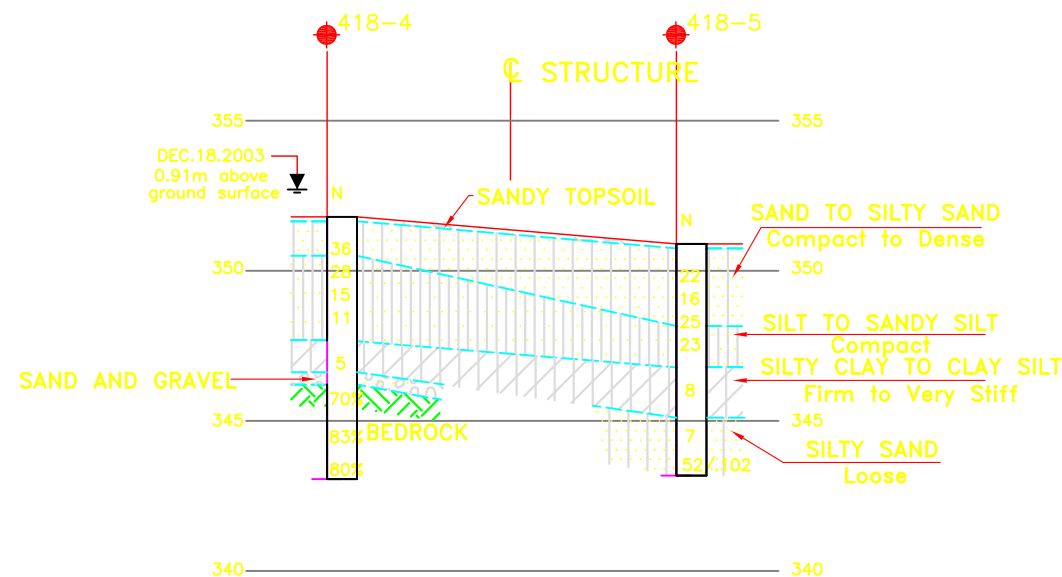
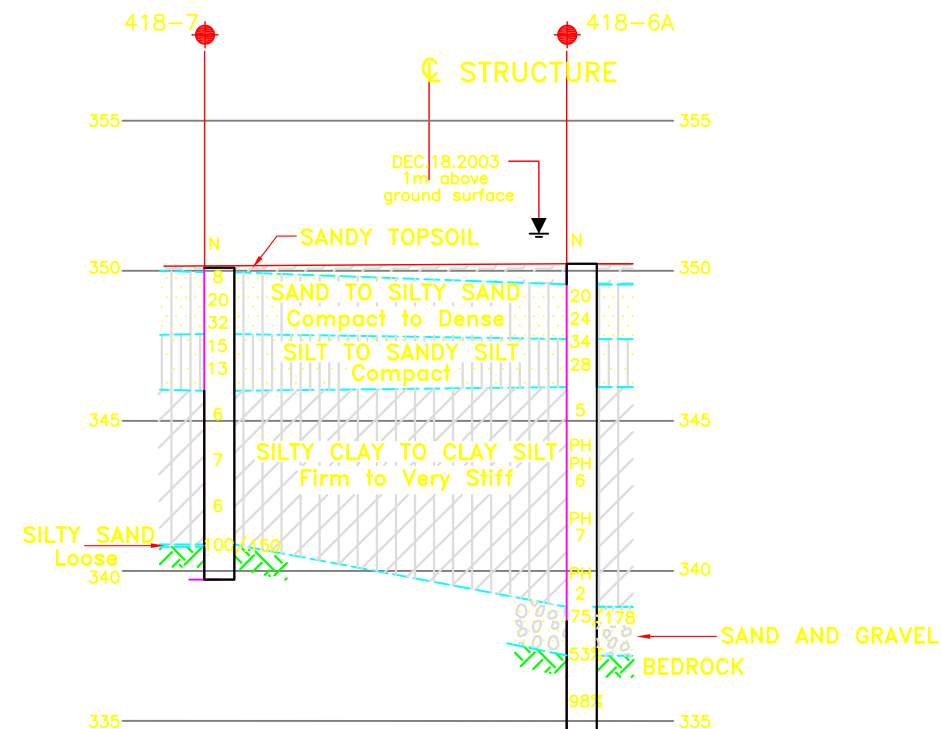
KEYPLAN

LEGEND

- | | |
|---|--------------------------------------|
|  | BoreHole by THURBER |
|  | Dynamic Cone penetration Test (cone) |
|  | BoreHole and Cone |
| N | Blow/0.3m (Std Pen Test, 475 J/blow) |
| CONE | Blows/0.3m (60° Cone, 475 J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation (RQD) |

NO	ELEVATION	NORTHING	EASTING
418-1A	350.0	5 068 807.0	311 334.8
418-4	351.6	5 068 833.9	311 386.9
418-5	351.3	5 068 808.4	311 390.8
418-6A	350.3	5 068 800.8	311 351.6
418-7	350.1	5 068 825.3	311 349.4
418-15	353.8	5 068 817.8	311 430.4
418-18	353.7	5 068 842.7	311 421.8
418-20	354.8	5 068 834.9	311 446.2

— NOTE —



SEE SHEET "HWY 124 I/C
UNDERPASS-BOREHOLE
LOCATIONS AND SOIL STRATA"
FOR CROSS-SECTION LOCATIONS
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

CREATED:

BENCH MARK
DHO BM 351-67 Tablet Set
horizontally in SE corner of
church 196.86 RT of 20+202.6
BM Elev. 375.305

[illegible]