

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CAMPBELL DRIVE UNDERPASS
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-415
GEOCRES Number: 31F-129**

Report to

National Capital Engineering

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

September 20, 2004

File: 19-3745-0

C:\Thurber Projects 2003\19-3645-0\Campbell Dr. Fdn. Inv. & Des. Rep FINAL sep 13.wpd

TABLE OF CONTENTS

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING.....	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Topsoil.....	3
5.2	Fill	4
5.3	Silty Clay.....	4
5.4	Sandy Silt to Silty Sand Till.....	5
5.5	Bedrock	6
5.6	Water Levels	7
6	INTRODUCTION	9
7	STRUCTURE FOUNDATIONS.....	9
7.1	Foundation Alternatives	10
7.2	Spread Footings on Bedrock	11
7.2.1	General.....	11
7.2.2	Bearing Resistance.....	11
7.2.3	Horizontal Resistance of Footings.....	12
7.3	Spread Footings on Engineered Fill.....	12
7.4	Augered Caissons.....	13
7.4.1	Axial Resistance	13
7.4.2	Downdrag.....	13
7.4.3	Lateral Resistance	13
7.4.4	Caisson Installation.....	15
7.5	Driven Piles.....	16
7.5.1	Axial Resistance	16
7.5.2	Downdrag on Abutment Piles.....	17
7.5.3	Lateral Resistance	17
7.5.4	Pile Installation	17
7.6	Frost Cover.....	18
8	EXCAVATION AND BACKFILL	18
8.1	General	18
8.2	Foundations	18
8.3	Earth Excavation	18

8.4	Rock Excavation	19
9	GROUNDWATER CONTROL	19
10	APPROACH EMBANKMENTS	20
10.1	Stability	20
10.2	Settlement.....	22
10.3	Construction	23
11	RETAINED SOIL SYSTEMS.....	23
11.1	Foundation.....	24
11.2	Global Stability	24
11.3	Internal Stability.....	25
11.4	Settlement.....	25
12	BACKFILL TO ABUTMENTS	26
13	EARTH PRESSURE	26
14	SEISMIC CONSIDERATIONS	28
14.1	Seismic Design Parameters	28
14.2	Liquefaction Potential	28
14.3	Retaining Wall Dynamic Earth Pressures	29
15	CONSTRUCTION CONCERNS	29

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Foundation Comparison
Appendix D	Figure and Table
Appendix E	Special Provisions
Appendix F	Borehole Locations and Soil Strata Drawing
Appendix G	Selected Stability Analyses Results

FOUNDATION INVESTIGATION AND DESIGN REPORT

CAMPBELL DRIVE UNDERPASS

HIGHWAY 17 TWINNING

ARNPRIOR TO RENFREW, ONTARIO

G.W.P. 647-92-00, SITE NO. 29-415

GEOCRES Number: 31F-129

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of the Campbell Drive Underpass structure that will carry Campbell Drive over the Highway 17 westbound and eastbound lanes. The Ministry of Transportation (MTO) carried out a preliminary foundation investigation for the existing Highway 17 that included one borehole drilled in the general vicinity of this site. The factual data from that investigation has been used as a reference during the preparation of this report.

The purpose of the investigation was to determine the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan and soil strata drawing with stratigraphic profile and cross-sections, records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed based on the data obtained from this investigation.

Thurber carried out this investigation as a sub-consultant to National Capital Engineering, under the Ministry of Transportation Ontario (MTO) Purchase Order Number 4005-A-000157.

The following document is referenced in the preparation of this report:

- MTO report titled "Preliminary Foundation Report For Structure Crossings of Revised Hwy #17 From Antrim Westerly to Locheil Creek, Regional Municipality of Ottawa, Carleton and Renfrew County", District No. 9 (Ottawa), W.J. 69-F-86 and W.P.'s 5-67 & 190-67, GEOCRES No. 31F-23 dated March 12, 1970 (Reference 1).

2 SITE DESCRIPTION

The site is located just west of the existing at-grade intersection of Highway 17 and Campbell Drive, Township of McNab, County of Renfrew (approximate mainline Station 25+950). This location is to the west of the Town of Arnprior. The Borehole Locations and Soil Strata drawing in Appendix F presents further details on the general site location.

The site terrain is flat with open fields on both sides of Highway 17. The land is lightly vegetated with grass, shrubs and small patches of deciduous and coniferous trees.

The project area is located between the Laurentian upland to the north and west, and the Ottawa lowland to the south and east. This area is situated within a physiographic region known as the Ottawa Valley Clay Plains. Native soil deposits consisting typically of glacio-lacustrine clayey silts and silty clays were deposited in the geologic past when the Champlain Sea inundated the Ottawa – St. Lawrence lowland. The clay deposits are interrupted by ridges of rock and deposits of sands and silts. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechère” graben. To the south and east of Arnprior, the bedrock is predominantly crystalline limestone of the Ordovician Period. To the west of Arnprior, the overburden deposits are underlain by bedrock consisting of metamorphosed greywacke of Precambrian age interbedded with limestone.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of September 25 to 29, 2003, on October 16, 2003 and March 12, 2004. The site investigation consisted of drilling and sampling a total of nine boreholes to depths ranging from 2 m to 7.9 m. The boreholes were numbered CAM-1 to CAM-8 and CAM-8A. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing.

The borehole locations were staked and/or marked in the field by surveyors from J. D. Barnes Limited (Ottawa) who also provided us with the plan coordinates (northings and eastings) and geodetic elevations of the boreholes after drilling was completed. Utility clearances were obtained for the borehole locations by Thurber prior to drilling.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations for all eight boreholes. Auger drilling techniques were used to advance the boreholes through the overburden, and soil samples were obtained using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). In Borehole CAM-8A, an undisturbed cohesive sample was obtained using a 73 mm outside diameter, thin-walled Shelby tube. Field vane shear tests were carried out within the cohesive soils in several boreholes using a standard MTO N-size vane. One borehole at each of the three foundation elements was advanced to a depth of 3.1 m below bedrock surface by NQ size diamond core drilling techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes CAM-3, CAM-4 and CAM-7 to enable monitoring of the groundwater level over a longer period of time. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 19mm diameter Schedule 40 PVC pipes with 2 m long slotted screens were installed near the bottom of the open boreholes. The sand screens surrounding the pipes were in the order of 2.5 m long. Bentonite holeplug seals were placed in the boreholes, one directly on top of the screen and

one immediately below ground surface. The remaining space in the boreholes were backfilled with auger cuttings. Upon completion of drilling and sampling, all remaining boreholes were appropriately backfilled.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, secured the soil and rock samples in labelled containers and core boxes, respectively, which were then transported to Thurber's Oakville laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. On completion of drilling and sampling, all boreholes were appropriately backfilled.

4 LABORATORY TESTING

The 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 soil samples were subjected to gradation analysis and Atterberg Limits tests. A laboratory consolidation (oedometer) test was conducted on an undisturbed sample of the silty clay obtained in Borehole CAM-8A. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point load testing was carried out on selected rock cores retrieved from Boreholes CAM-3, CAM-4 and CAM-7. These results are shown in Table 1 attached immediately following the text.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference should be 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 Soil Strata" drawing in Appendix F. A summary description of the stratigraphy is given in the following paragraphs.

In general, the subsurface conditions at this site consist of topsoil and occasional fill which is underlain by silty clay overlying silty sand to sandy silt till. These soil deposits are underlain by crystalline limestone bedrock.

5.1 Topsoil

A surficial layer of topsoil, ranging between 25 mm and 175 mm in thickness, was encountered at most borehole locations across the site as shown in the table below.

Borehole	Topsoil Thickness (mm)
CAM-1	175
CAM-2	150
CAM-3	150

CAM-4	100
CAM-5	100
CAM-7	25
CAM-8	75

Topsoil thickness may vary between the boreholes and in other areas of the site.

5.2 Fill

A 0.7 m thick layer of gravelly sand fill was encountered in Borehole CAM-6 located on the gravel shoulder of Campbell Drive. An SPT 'N' value of 20 blows for 0.3 m penetration was recorded within this brown fill indicating a compact relative density. The measured moisture content of a sample of this fill is 6%.

5.3 Silty Clay

A deposit of silty clay was encountered below topsoil or fill at all borehole locations across the site, between approximate Elevations 113.6 m and 107.1 m. This deposit extends to depths ranging from 1.5 to 5.9 m below the existing ground surface. The upper 0.2 m of the silty clay in Borehole CAM-6 contains organics, rootlets and occasional peat inclusions.

Within this deposit, the upper, brown coloured zone or "crust" is about 2 m to 2.5 m thick and extends to approximate Elevation 111.0 m. Below this elevation, the lower, grey coloured zone has a softer consistency and a thickness ranging between 2 m and 3 m.

Standard Penetration Tests conducted within the upper zone of the deposit gave 'N' values ranging from 4 to 18 blows per 0.3 m penetration indicating a firm to very stiff consistency, except in Borehole CAM-8 where a soft surficial layer with a 'N' value of 2 blows was recorded. Similar trends of consistency were inferred from pocket penetrometer values. In the lower zone, field vane shear strengths were generally between 24 kPa and 35 kPa. The sensitivity ranged between 4 and 11. Standard Penetration Tests in this lower region of the silty clay deposit gave 'N' values ranging from 1 to 4 blows per 0.3 m penetration. Based on these results, the lower portion of this deposit is considered to have a soft to firm consistency.

Figures B1 and B2 show the grain size distributions of eight selected silty clay samples. These results indicate that the clay content of this soil varied between 15% and 44%. The higher clay contents are generally associated with the lower zone, except for the sample from Borehole CAM-6.

Atterberg Limits tests conducted on selected samples of this silty clay deposit are presented in Figures B4 and B5. The results indicated that the liquid limits vary typically between 39% and 48%, and corresponding plasticity indices between 20% and 24%. These values indicate that the silty clay is typically of medium plasticity (CI). An occasional highly

plastic (CH) zone was present in Borehole CAM-6 where the liquid limit and plasticity index were 56% and 28%, respectively. The moisture contents of the tested samples were generally close to the liquid limit values. For samples from Boreholes CAM-5 and CAM-8, the measured moisture contents were higher than their corresponding liquid limits. Throughout this cohesive deposit, the measured moisture content ranged from 18% to 53%.

One laboratory consolidation (oedometer) test was carried out on an undisturbed specimen prepared from a Shelby tube sample obtained in Borehole CAM-8A. Inferred parameters from the test are summarized in the following table.

Borehole and Sample Number	Existing Overburden Pressure, p'_0 (kPa)	Pre-consolidation Pressure, p'_c (kPa)	Compression Index, C_c	Re-compression Index, C_r	Initial Void Ratio	Over-consolidation Ratio OCR
CAM-8A TW 1	45	110	0.26	0.025	0.92	2.5

The average coefficient of consolidation, C_v , value is $0.035 \text{ cm}^2/\text{s}$ within the range of stresses anticipated to be acting on the foundation soils.

The parameters obtained from this test are considered representative of the over-consolidated nature of the clay deposit.

A specific gravity value of 2.79 was measured for the tested specimen. This value corresponds to a unit weight of approximately 17.6 kN/m^3 .

Detailed results of this oedometer test are included as Figure B6 in Appendix B.

5.4 Sandy Silt to Silty Sand Till

A deposit of sandy silt to silty sand till was encountered at several borehole locations (CAM-1, CAM-2, CAM-3, CAM-6 and CAM-7) across the site between approximate Elevations 112.0 m and 106.7 m. This till was encountered from a depth of 1.5 m to 5.6 m and extended to a depth of 2 m to 7.6 m.

Standard Penetration Tests conducted within this stratum yielded 'N' values ranging from 10 blows to more than 50 blows for 0.3 m penetration. This till is considered to have a generally compact relative density with occasional "refusal" values attributed to the presence of bedrock pieces, cobbles or boulders.

Figure B3 shows the grain size distributions of two selected till samples. The clay content of these till samples were 15% and 19%. The measured natural moisture contents of the till samples lied within 9% and 11%.

5.5 Bedrock

The soils described above are underlain by crystalline limestone bedrock of the Ordovician period. Bedrock was proven by coring in Boreholes CAM-3, CAM-4, and CAM-7. The bedrock surface was inferred from refusal to auger penetration in the remaining five boreholes drilled at this site. Bedrock surface depths and elevations at the borehole locations are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Bedrock Surface	
		Depth (m)	Elevation (m)
CAM-1	113.7	2.0*	111.7*
CAM-2	113.3	2.2*	111.1*
CAM-3	113.2	4.8	108.4
CAM-4	113.3	4.5	108.8
CAM-5	112.7	4.4*	108.3*
CAM-6	114.3	7.6	106.7*
CAM-7	113.2	4.1	109.1
CAM-8	113.0	5.9*	107.1*

Note : * Inferred from auger refusal.

The crystalline limestone is generally described as fresh to slightly weathered at the joints, very thinly to thinly bedded, grey with dark grey and white horizontal and sub-vertical bandings. In Borehole CAM-4, occasional red subvertical banding was also observed.

The measured Total Core Recovery (TCR) in the bedrock ranged between 67% and 100%. Rock Quality Designation (RQD) values ranged typically from approximately 50% to 100%, indicating fair to excellent rock quality. RQD values of 0% and 28% were recorded in the Run #1 of Boreholes CAM-7 and CAM-3, respectively, and a value of 38% was obtained in Run #2 in Borehole CAM-4.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low, ranging from 0 to 4. Higher values of 5 and greater were recorded in Run #1 of all three boreholes. The joints were generally sub-vertical to vertical, and the conditions were uneven and rough. Occasional iron oxide staining of joints and calcite infilling of vugs were also observed.

The inferred Unconfined Compressive Strengths (UCS) of the rock cores from point load tests, expressed as average value per run, ranged from 80 MPa to greater than 100 MPa, indicating that the intact rock is strong to very strong. However, in Borehole CAM-3, the strength of the rock was assessed to be in the order of 16 MPa between 4.8 m (Elevation 108.4 m) and 5.3 m (Elevation 107.9 m) inferring moderately strong rock within this zone. A summary of the Point Load Test Results is presented in Table 1 attached immediately following the text.

5.6 Water Levels

Caving was observed in Borehole CAM-6 at a depth of 7 m below ground surface. No free groundwater was recorded in the remaining boreholes on completion of drilling. Standpipe piezometers were installed in Boreholes CAM-3, CAM-4 and CAM-7 and their water levels were measured on three separate visits made after completion of drilling.

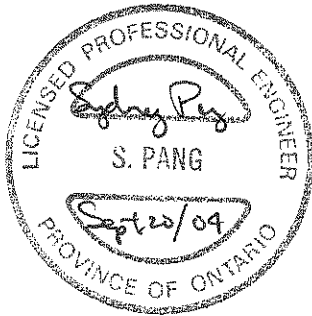
These readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
CAM-3	October 22, 2003	0.1	113.1
	December 18, 2003	0.1	113.1
	February 04, 2004	0.3	112.9
CAM-4	October 22, 2003	0.6	112.7
	December 18, 2003	0.4	112.9
	February 05, 2004	0.8	112.5
CAM-7	October 22, 2003	0.3	112.9
	December 18, 2003	0.3*	112.9*
	February 05, 2004	0.3*	112.9*

* Water frozen in piezometer.

Based on these observations, it is considered that the local groundwater levels exist at elevations varying between Elevations 112.5 m and 113 m.

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



Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.
Senior Project Engineer



Report Reviewed by:
P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

FOUNDATION INVESTIGATION AND DESIGN REPORT
CAMPBELL DRIVE UNDERPASS
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-415
GEOCRES Number: 31F-129

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 INTRODUCTION

This report presents 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 for the proposed structure.

It is understood that the preliminary design plan calls for the construction of a new structure to carry the realigned Campbell Drive over the twinned Highway 17. The existing Highway 17 will become the eastbound lanes of the twinned Highway 17, and a new roadway will be constructed on the north side to become the westbound lanes.

The proposed structure will be a two-span post tensioned concrete box girder bridge. Each span will be approximately 55 m long and the foundation elements will be skewed at 30°.

At the bridge site, the proposed grade of Campbell Drive will be at about Elevation 122 m at the south abutment and at about Elevation 123 m at the north abutment. Given the existing ground surface at approximate Elevation 114 m, the south and north approach fills are estimated to be in the order of 8 m and 9 m, respectively, above the existing ground surface.

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.

7 STRUCTURE FOUNDATIONS

The proposed bridge for this site will consist of a two-span underpass structure with a total of three foundation elements: two abutments and one centre pier.

The stratigraphy encountered at the locations of the proposed abutments and pier consists of predominantly silty clay overlying relatively thin layers of sandy silt to silty sand till underlain by bedrock.

The elevations at which bedrock was encountered or inferred at the three foundation elements are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
North Abutment			
East Side	CAM-2	113.3	111.1±*
West Side	CAM-3	113.2	108.4
Centre Pier			
East Side	CAM-4	113.3	108.8
West Side	CAM-5	112.7	108.3 ±*
South Abutment			
East Side	CAM-6	114.3	106.7±*
West Side	CAM-7	113.2	109.1

* Bedrock inferred from auger refusal.

7.1 Foundation Alternatives

This section presents discussions on available foundation alternatives, provides recommendations and foundation design parameters for feasible and/or preferred foundation option(s) for this site.

During initial assessment, consideration was given to the following foundation types:

- Spread footings
- Driven piles
- Caissons (drilled shaft piles)

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix C.

An integral abutment design is not a practical foundation alternative due to the skew angle of the proposed structure and the presence of bedrock at relatively shallow depth. Should the structure be designed to include integral abutments, piles may have to be socketted at some locations into bedrock in order to provide adequate lateral flexibility. A semi-integral abutment design utilizing footings is possible.

Spread footings founded on the compressible silty clay or on an engineered fill pad resting on the silty clay are not feasible due to the anticipated large magnitude of post construction settlement. Spread footings founded on engineered fill resting on bedrock are not considered feasible at the abutments as the required height of the abutment walls would be impractically high, unless the engineered fill pads are greater than 6 m to 7 m in height in which case the bearing resistance at SLS would be limited (much less than that on bedrock). At the pier, this option is technically possible but unnecessary as it would be more practical to place the footing directly on the shallow bedrock (see below) to take advantage of the much higher bearing resistance.

Short augered caissons socketed into bedrock is a feasible foundation option at this site. It is considered impractical to use driven piles at the centre pier as the required pile length would only be 3 m to 4 m below the anticipated base of the pile cap. Footing resting directly on bedrock is a practical alternative at the pier. At the abutment locations where the proposed grade raise is in the order of 8 m to 9 m, a perched abutment design should be considered where piles are driven to, or socketed into, bedrock, or augered caissons are socketed into bedrock.

The preferred scheme of foundation support for this bridge would be to use augered caissons or driven piles to bedrock at both abutments, and spread footings on bedrock at the pier.

7.2 Spread Footings on Bedrock

7.2.1 General

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of the highway, this option is only feasible for the pier foundation. At the centre pier, footing construction will require a 4 m to 5 m deep excavation with temporary support and unwatering.

The top surface of the bedrock should be stripped of all overburden and be cleaned. Inspection should be carried out to confirm that the bedrock conditions, as exposed at the founding level, are consistent with the design assumptions. All shattered and loosened rock fragments should be removed from the footprint of the footing and replaced with mass concrete fill, where necessary. Where bedrock is lower than anticipated, mass concrete may be placed to raise the subgrade to the design footing level.

Rock excavation is expensive and unnecessary excavation of bedrock should be avoided where practicable.

Where practicable, 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. This approach will reduce the risk of having to excavate bedrock under a footing. Where necessary, the footing may be stepped down across the width of the structure to accommodate changes in the elevation of the top of bedrock. The recommended design top of rock is as follows:

Centre Pier

The top of rock varies between approximate Elevations 108.8 m and 108.3 m across this foundation.

7.2.2 Bearing Resistance

Footings bearing on sound crystalline limestone bedrock encountered at this site may be designed for a factored geotechnical resistance of 5,000 kPa at Ultimate Limit States (ULS)

for vertical, concentric loads. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4. The design of the footing may be governed by other considerations such as sliding resistance or overturning moment.

The SLS condition will not govern design for footings founded on bedrock.

The same value of resistance may be used where mass concrete of suitable strength is poured in neat contact with a clean, sound bedrock surface.

7.2.3 Horizontal Resistance of Footings

Resistance to lateral forces / sliding resistance between the concrete footing and the bedrock surface at the centre pier location should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.85.

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide shear resistance.

The dowel may be considered as acting as a fully embedded short pile in the rock. Using a lower bound value of 30 MPa for the unconfined compressive strength of the rock (reduced to allow for fracturing near the surface) or the strength of the grout, an ultimate horizontal resistance of 1.3 MN was calculated 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.

7.3 Spread Footings on Engineered Fill

Spread footings founded on the compressible silty clay or on engineered fill pads resting on the silty clay is anticipated to result in post construction settlement up to the order of 50mm. At the approaches, this settlement is in addition to the settlement of the foundation soils due to embankment loading.

Spread footings founded on engineered fill resting on bedrock are not considered feasible at the abutments as the required height of the abutment walls would be impractically high, unless the engineered fill pads are greater than 6 m to 7 m in height in which case the bearing resistance at SLS would be limited (much less than that on bedrock). At the pier, this option is technically possible but unnecessary as it would be more practical to place the footing directly on the shallow bedrock to take advantage of the much higher bearing resistance.

Based on the above, it is therefore recommended that spread footings on engineered fill not be considered at this site.

7.4 Augered Caissons

The abutments may be supported by augered caissons (drilled shafts) founded on bedrock. In order to found the caissons below the surficial, more fractured zone of the bedrock and to enhance caisson base contact with sound bedrock, it is recommended that the caissons be designed to be nominally socketted at least 500 mm into bedrock. The sockets should be formed below the low side of a sloping bedrock surface. The recommended design top of bedrock is the same as those presented for spread footings in Section 7.2.1.

7.4.1 Axial Resistance

For a caisson nominally socketted 500 mm into bedrock, the axial capacity is assumed to be derived from end bearing only. It is recommended that a factored geotechnical resistance at ULS of 10,000 kPa be used for design.

The SLS condition will not govern for caissons founded on bedrock.

7.4.2 Downdrag

Downdrag forces could be induced on the caissons as a result of consolidation of the cohesive foundation soils under the loading of the 10 m high approach fills. The magnitude of the downdrag force depends on the contact area between the caisson surface and the surrounding soil. Reference should be made to the CHBDC (2000) Clauses 6.8.4 and C6.8.4 for downdrag calculations. Given the high end bearing resistance provided by the bedrock, the neutral plane may be assumed to be located at the bedrock surface.

Due to the relatively small thickness of the lower, compressible zone of the silty clay (in the order of 2 m to 3 m) present above bedrock at this site, it is estimated that the magnitude of downdrag force acting on a caisson is relatively insignificant compared with its end bearing resistance, and will not affect the geotechnical resistance recommended in this report. For example, the factored downdrag force for a 900 mm diameter caisson at the south abutment (deepest clay deposit) is estimated to be in the order of 270 kN. Downdrag is, therefore, not considered to be a design issue at this site.

7.4.3 Lateral Resistance

At this site, the caissons that may be used at the abutments would be relatively short and be nominally socketted into bedrock only to enhance base contact with sound rock. If fixity is required at the rock contact, the caissons should be drilled to a depth of at least twice its diameter into the rock.

Given the shallow depth to bedrock and the proposed final grade of the highway, much of the soil lateral resistance would be derived from the approach fill. For the soil conditions at this location, the lateral resistance of the caissons may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

Engineered Sand and Gravel Fill (compact)

$$k_s = n_h \cdot z / D \quad (\text{kPa/m})$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa}) \quad (\text{at and above Elevation 113.5 m})$$

Silty Clay (stiff to soft)

$$k_s = 250 \cdot S_u / D \quad (\text{kPa/m})$$

$$p_{ult} = 9 \cdot S_u \quad (\text{kPa})$$

where

z = depth below abutment base in metres

D = caisson diameter in metres

n_h = 6,000 kPa/m (engineered fill compacted to at least 95% Standard Proctor density)

γ = 20 kN/m³ (engineered fill)

= 18 kN/m³ (silty clay)

K_p = 3.0 (passive earth pressure coefficient)

S_u = undrained shear strength of silty clay

= 30 kPa (below Elevation 110.5 m)

= 60 kPa (at and above Elevation 110.5 m)

The above equations and recommended parameters may be used for numerical analysis of the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the numerical analysis should not exceed the ultimate lateral resistance, p_{ult} .

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times L \times D$ (MN/m), where k_s is the coefficient of horizontal subgrade reaction (MPa/m), D is the caisson width (m), L is the length (m) of the pile segment or element used in the analysis.

Since the caissons are end bearing on rock, the vertical resistance will not be significantly affected by the caisson spacing. Caisson interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil / pile group interaction analysis, the equation for k_s quoted above may be used in conjunction with appropriate reduction factors.

Where a caisson group is oriented **perpendicular** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows :

Caisson Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D	1.00
1 D	0.50

where D is the diameter of the caisson, and spacing is measured centre to centre

Where a caisson group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_h by a reduction factor R as follows :

Caisson Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

Where the lateral resistance derived from the soils is insufficient to withstand the design lateral loads, consideration may be given to extending the rock sockets further into bedrock. The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by :

$$P_p = 6 \cdot c \cdot D \cdot L$$

where

$$c = 2,000 \text{ kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)}$$

$$L = \text{depth of socket in rock, m}$$

7.4.4 Caisson Installation

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

Caisson installation at the abutment locations would be carried out predominantly through silty clay and/or sands and silts with cobbles and boulders and socketted into bedrock. It is recommended that the caissons be constructed by using temporary steel liners to support the sidewalls and to allow hand cleaning and inspection of the rock bearing surface. A minimum caisson diameter of 900 mm, and as governed by applicable regulations, is required to allow down-the-hole hand cleaning and inspection.

The base of the caisson should be drilled at least 500 mm into the bedrock to remove weathered and highly fractured rock, and to mitigate the impact of a sloping rock surface.

For moderately sloping bedrock surface anticipated at this site, it is recommended that the base of the caisson be drilled to 500 mm below the low side of the rock surface. Stepping of the caisson base is allowed in SP 903S01, but is likely not required for this site.

It should be expected that caisson installation would encounter cobbles and boulders within the sand and silt till. The caisson installation equipment should be capable of dislodging, handling and removing cobbles and boulders.

It is anticipated that a liner advanced into the bedrock will provide some seepage cut-off. Should water seepage be encountered, the caisson hole should be pumped dry prior to allowing personnel into the hole. The concrete should be placed using good tremie techniques.

It is recommended that two NSSPs be included in the contract documents, one to alert the contractor of the potential presence of boulders and cobbles, and one to address the dewatering requirements.

7.5 Driven Piles

Steel piles driven to bedrock may be used to provide foundation support at the abutments. It is considered that the required pile length would be too short for the centre pier foundation. Based on the borehole information, bedrock is present at between 2.5 m and 5 m depths below existing ground surface at the north abutment, and at between 4 m and 8 m depths at the south abutment. The following pile tip elevations are recommended for design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
North Abutment	CAM-2, CAM-3	111± to 108± (east to west)
South Abutment	CAM-6, CAM-7	106.5± to 109± (east to west)

For integral abutment design, the piles will have to be socketted into bedrock in order to allow adequate pile flexibility to develop within the upper 3 m of the pile.

7.5.1 Axial Resistance

For designing HP 310 x 110 piles driven to bedrock, the following recommended pile capacities may be used:

- Factored geotechnical resistance at ULS of 2,000 kN per pile.

The SLS condition does not apply to piles founded on bedrock.

The structural resistance of the pile should be reviewed by the structural designer to confirm that the value given above is not exceeded.

7.5.2 Downdrag on Abutment Piles

Downdrag forces could be induced on the piles as a result of consolidation of the cohesive foundation soils under the loading of the new approach fills. Due to the small thickness of the lower, compressible zone of the silty clay (in the order of 2 m to 3 m) present above bedrock at this site, it is estimated that the magnitude of downdrag force acting on the piles is relatively insignificant and will not affect the geotechnical resistance above. For example, the factored downdrag force for an HP 310 x 110 pile at the south abutment (deepest clay deposit) is estimated to be in the order of 120 kN. Downdrag is, therefore, not considered to be a design issue at this site.

7.5.3 Lateral Resistance

For design of conventional pile groups at the piers, it is recommended that the unbalanced horizontal forces be resisted by battered piles.

For lateral soil-pile interaction analysis, the recommendations, expressions and parameter values for coefficient of horizontal subgrade reaction (k_s), ultimate lateral resistance (p_{ult}) and group action reduction factors in Section 7.4.3 are applicable.

For integral abutments, the flexibility of the pile can be increased by providing a double or single corrugated steel pipe (CSP) system. Considering that the flexible zone is situated within compacted fill, the double concentric pipe configuration is likely more suitable.

The sand for filling the CSPs should be uniformly graded and meet the gradation requirements presented in Table D1 in Appendix D.

Where the piles are socketted into bedrock, the recommended expression and parameter values in Section 7.4.3 may be used.

7.5.4 Pile Installation

All piles shall be installed in accordance with Special Provision SP No. 903S01.

Glacial tills inherently contain cobbles and boulders. Hard driving conditions through the typically compact glacial tills should be expected. Sloping bedrock surface is also anticipated at the site. In order to be able to penetrate boulders, cobbles and harder/denser zones in the foundation soils and to enhance adequate seating into bedrock, it is recommended that the pile tips be reinforced with rock points such as the Titus "H" Bearing Pile Point, Rock Injector design, or equivalent.

The appropriate pile driving note to be shown on the contract drawing is "Piles to be fitted with rock points and driven into bedrock in accordance with 903S01 (Note 6 in Clause 3.3.3 of Section 3 Piles, the Ministry of Transportation, Ontario "Structural Manual").

7.6 Frost Cover

The provision of frost cover for footings founded on sound bedrock is not required.

Frost protection should be provided to pile caps and footings founded on engineered fill, or native earth material. This may take the form of 1.9 m of earth cover, or equivalent insulation, over the underside of the pile cap or footing base (founding elevation).

It may be possible to eliminate the depth of frost cover if:

- The foundation is underlain by at least 2.5 m of free-draining, non-frost susceptible granular fill or by rock fill, and
- The water table is more than 2.5 m below the underside of the foundation.

8 EXCAVATION AND BACKFILL

8.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the fill, and the silty clay and till deposits below the groundwater table can be classified as Type 3 soils.

8.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01. A copy of this Special Provision is included in Appendix E.

8.3 Earth Excavation

In addition to SP 902S01, a NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles and boulders in the overburden especially in the glacial till.

At the centre pier location, excavation for footing construction will extend through the existing silty clay deposit to bedrock. Where open cutting with inclined slopes (according to OHSA) is not feasible, a braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring at this site. The soldier piles will need to be socketted into bedrock through pre-augered holes. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

An item titled "Roadway Protection" as per SP 539S01 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 be specified for this site.

For a temporary braced soldier pile and lagging wall, the lateral pressure diagram as shown on Figure D1 may be used for design in conjunction with the following parameter values.

$$\gamma = 20 \text{ kN/m}^3$$

$$\begin{aligned}\gamma_w &= 10 \text{ kN/m}^3 \\ K_a &= 0.4 \text{ (silty clay)} \\ h_w &= 0 \\ &\text{(assuming that there is no hydrostatic pressure build-up} \\ &\text{behind a presumably permeable wall)} \\ H &= \text{depth to base of excavation (rock surface), m}\end{aligned}$$

Below the excavation base, lateral earth pressures are applied over a width of $3B$, where B is the diameter of the socket, to take into account three dimensional effects. The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by :

$$\begin{aligned}P_p &= 6 c B L \\ \text{Where } c &= 2,000 \text{ kPa (equivalent Mohr-Coulomb cohesion} \\ &\text{based on Hoek and Brown rock mass classification)} \\ L &= \text{depth of socket in rock, m}\end{aligned}$$

It should be pointed out that the actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the shoring wall. These factors should also be considered when designing the shoring system.

8.4 Rock Excavation

It is anticipated that requirements for rock excavation will be minimal at this site. The strength of intact rock is strong to very strong. Relatively shallow excavation into rock for footing construction may require appropriate excavators equipped with rock teeth and rock splitting equipment. Blasting is not likely required at this site.

Any rock excavation should be carried out in accordance with the Special Provision, Amendment to OPSS 120, 1994 included in Appendix E.

Should blasting be proposed, the Contractor's blasting and monitoring plan should take into account nearby structures (residential dwellings etc.). The contract documents should alert the contractor to these structures. The Contract Administrator should retain a blasting expert for review of the Contractor's blasting procedures prior to approving them.

9 GROUNDWATER CONTROL

The relatively impervious deposits of silty clay should not yield significant amounts of seepage water in the short term. At locations where the more pervious zones such as the underlying sand and silt till and water-bearing seams within the silty clay are exposed in an excavation or cut, water seepage will occur into the excavations.

The design of foundations bearing on bedrock will not be influenced by the groundwater but the Contractor must make provision to control the groundwater seepage and use sump pumps to

remove any accumulated water from the footing base prior to placing concrete or compacting granular fill. All footings must be constructed in the dry.

10 APPROACH EMBANKMENTS

For the purpose of embankment stability analyses in this report, a commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed for short term (total stress, undrained) and long term (effective stress, drained) conditions, where applicable.

Immediate (elastic) settlements due to compression of cohesionless soils have been estimated based on elastic methods. Anticipated settlements due to primary consolidation of the foundation silty clay have been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry, foundation conditions and also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rockfill, it may be assumed that the side slopes will be stable at inclinations not steeper than 1.25H : 1V. Embankments constructed using granular material and select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V.

The approach embankments for this structure will be constructed on 3 m to 6 m of native silty clay overlying bedrock. After the topsoil, organics and other deleterious materials are removed to expose the subgrade, the required embankment will be in the order of 8 m to 9 m in height at the south and north abutments, respectively. With the exception of the compacted Granular A or other engineered fill core, the remainder of the embankment may consist of rockfill. The slope of the core may be formed not steeper than 1 H : 1 V. Provided that the core is constructed as recommended in this report, rockfill embankments formed with a slope inclination of 1.25H : 1V will be stable. Some settlement will occur within the rockfill and the compacted granular fill. This settlement should be complete by the end of construction and no post construction settlement of the fill itself is anticipated.

10.1 Stability

Stability analyses were carried out for the proposed slope configurations at the south and north approaches. At the south approach, it is assumed that foundation conditions consist of 3 m of silty clay crust overlying 3 m of firm silty clay, which is underlain by 1 m of sand and silt till on bedrock. At the north approach, it is assumed that 2 m of silty clay crust overlies 2 m of firm silty clay, which is underlain by 1 m of sand and silt till on bedrock.

Side Slopes

The Factors of Safety (F.S.) for the side slopes were typically greater than 1.3 for both north and south approaches. Figures G1 to G4 present selected stability analyses results for the north approach.

It is noted that rock fill embankments will have higher F.S. values than earth fill embankments.

In order to achieve higher F.S. and the likely situation where blast rock will be available from sites in this Highway 17 Twinning project (and perhaps also from other sites in the vicinity of this project), it is recommended that rock fill be used to construct the approach embankments at this site.

Forward Slopes

At the south approach, the calculated F.S. for rotational type failure for forward slopes were in the order of 1.2 and 1.3 during construction (undrained) and in the long term (drained), respectively.

At the north approach, the calculated F.S. for the forward slopes were in the order of 1.1 and 1.3 during construction and in the long term, respectively. Figures G5 and G6 present selected stability analyses results illustrating cases where the F.S. could be less than 1.3.

A F.S. of less than 1.3 (construction stage) is considered too low at the forward slopes. Figures G7 and G8 illustrate selected cases for the north approach where a minimum F.S. of 1.3 was achieved by relocating the abutment away from the slope toe and the use of a stabilizing berm.

It is noted that the use of berms at the forward slope location is impractical, or even impossible, due to space restrictions. Relocation of the abutment away from the slope toe would result in a longer and, therefore, more expensive superstructure. Slope flattening is not applicable to a forward slope configuration.

In view of the above, it is considered that the most cost-effective means of achieving the required minimum F.S. of 1.3 at all times is to place all required embankment fill in advance of the bridge construction. The advance placement of fill will induce excess pore pressures within the lower portion of the foundation silty clay deposit. By allowing the pore pressures to dissipate within the waiting period, it is anticipated that the resulting gain in strength within the silty clay would assist in maintaining a minimum F.S. of 1.3. Figures G9 to G11 illustrate the various phases of constructing the north approach involving advance fill placement, pore pressure dissipation, strength gain and fill removal. Similar results can be achieved for the south approach.

Earth fill is not recommended for constructing the approach embankments as it would result in lower F.S. than if rock fill is used, and embankment construction would likely have to be carried out in two stages.

The recommended procedures for carrying out the above scheme is outlined as follows:

Rock Fill Embankment

- Place rock fill up to the design pavement subgrade level at both approaches. The toe of the rock slope (1.25H : 1V) should coincide with the heel of the proposed abutment footing.
- Place Granular B, Type II fill (2H : 1V) in front of the rock fill slope. The toe of the granular fill should be at least 20 m in front of the abutment wall face. Place granular fill to top of pavement level (Figure G9 : F.S. = 1.31).
- Leave all fill in place for not less than six months (waiting period) after the completion of fill placement. This will allow dissipation of a majority of the excess pore pressures generated within the foundation clay by placement of the new fill. As a result, the foundation clay will proportionally gain strength and post construction settlement will also be reduced (Figure G10 : F.S. = 1.4). Survey monitoring (such as using settlement plates) should be carried out to confirm that all foundation settlements have stabilized prior to proceeding with abutment construction.
- Construct the abutment.
- After the abutment is constructed and all fill removed from the front of the abutment face, all excess pore pressures is anticipated to be fully dissipated (Figure G11 : F.S. = 1.32).

Seismic Stability

In general, the approach embankments may consist of rock fill with a granular core or inorganic earth fill founded on native silty clay overlying bedrock. The foundation sand and silt till only exists as thin discontinuous pockets on bedrock. The groundwater level is at or below the base of the embankment. These materials have negligible to no potential for liquefaction. Consequently, the approach embankments will be stable against seismic activities at this site.

10.2 Settlement

Settlement in the order to 40 to 50 mm will occur within the rock fill or well compacted non-cohesive earth fill. This settlement should be complete by the end of construction and negligible post construction settlement is anticipated in the fill.

The new approach fills will induce settlement within the foundation soils. Without preloading and surcharging, results of calculations indicate that the settlement due to elastic recompression (heavily over-consolidated zone above Elevation 111 m, and lightly over-consolidated zone below Elevation 111 m) under 9 m of fill could be in the order of 50 mm to 75 mm (for 5 m of clay) that is anticipated to occur immediately upon

completion of fill placement. Settlement due to primary consolidation could be up to 50 mm (for 5 m of clay) that is anticipated to occur within 3 to 4 months after completion of fill placement. Settlement should be less than those quoted above in areas where clay thickness is less than 5 m.

Provided that the approach embankments are constructed in advance of bridge construction as recommended in this report, it is estimated that post construction settlement at both approaches should be negligible.

10.3 Construction

Embankment construction should be carried out in accordance with OPSS 206, as amended by Special Provision “Amendment to OPSS 206, December 1993”, dated November 2002 and included in Appendix E. Earth fill should consist of granular materials or Select Subgrade Material (SSM) in compliance with Special Provision 110F13, “Amendment to OPSS 1010, March 1993”. Clean, inorganic earth fill (in accordance with OPSS 212) may consist of clayey materials that could itself settle in the order of 50 mm. The use of cohesive earth fill is, therefore, not recommended for use within a 20 m zone immediately behind the abutments, but may be considered for use beyond the 20 m zone. SSM should be used within the 20 m zone immediately behind the abutment wall.

Where rock fill embankments are higher than 10 m, mid-height berms should be incorporated. The berms should be 2 m wide and extend for the length through which the embankment height exceeds 10 m. Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated. The berms should be 2 m wide and extend for the length through which the embankment height exceeds 8 m, and the berms should maintain a 2% positive drainage grade to shed surface run-off. The requirement for these berms is in place to address surficial stability and to provide access for post construction maintenance.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572 and related special provision(s).

11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used at this site provided preloading is carried out as recommended in this report. A conventional concrete abutment will be required for the contemplated design but RSS could be used for wing walls and other retaining structures. The risk of using RSS walls at this site is low provided that the construction of all RSS components, including the levelling pad, reinforcement strips, soil mass and concrete panels, commence only after completion of advance fill placement, waiting period, and stabilization of foundation settlements.

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.

11.1 Foundation

At this site, it is recommended that the levelling pad for an RSS wall be centred on top of a mat of engineered fill that is itself resting on native stiff silty clays or compacted, approved embankment fill. Where applicable, the native soil under the RSS foundation should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at $\pm 2\%$ of its optimum moisture content. The engineered fill should consist of OPSS Granular A compacted in accordance with the OPSS 501, Method A. The engineered fill mat should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

The following parameters may be used for the design of the levelling pad:

- Factored geotechnical resistance at ULS of 300 kPa, and geotechnical resistance of 200 kPa at SLS on an engineered Granular A pad.
- Coefficient of friction of between cast in-situ concrete levelling pad on Granular A is 0.7.

In addition to the requirements for the levelling pad, the RSS should be founded on native stiff silty clay or on compact embankment fill. The following parameters may be used for foundation design of an RSS wall:

- Factored geotechnical resistance at ULS of 225 kPa, and geotechnical resistance at SLS of 150 kPa, on the compact embankment fill
- Factored geotechnical resistance at ULS of 225 kPa, and geotechnical resistance of 150 kPa at SLS, on native, firm to soft silty clay.
- Ultimate coefficient of friction of between RSS mass and earth/rock fill is 0.55.
- Ultimate coefficient of friction of between RSS mass and native silty clay is 0.45.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

11.2 Global Stability

The global stability of the RSS is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment.

RSS walls, if used at this site, are likely to be for wing walls at the abutments. It is envisaged that the RSS will be founded on compacted granular fill overlying native stiff

silty clay. The main body of the approach embankments should consist of rock fill as recommended in this report.

For the purpose of stability analyses in this report, a commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed for short term (total stress, undrained) and long term (effective stress, drained) conditions, where applicable.

Stability analyses on selected configurations were carried out considering the following variables:

- Compact granular fill – Granular B, Type II compacted to 100% SPMDD at $\pm 2\%$ optimum moisture content (angle of internal friction, ϕ , of 32° , cohesion of 0, and unit weight, γ , of 21 kN/m^3).
- Rock fill embankment – outer slope of 1.25H : 1V (angle of internal friction, ϕ , of 42° , cohesion of 0, and unit weight, γ , of 19 kN/m^3).
- Groundwater level at existing ground surface.
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 50% of the height, founded on compacted earth fill.

Results of the analyses yield minimum Factors of Safety of 1.3, indicating that global stability can be maintained for the assumed RSS configuration. Figures G12 and G13 present selected stability analyses results for the assumed RSS configuration.

The actual design configuration must be checked for global stability prior to finalization.

11.3 Internal Stability

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

11.4 Settlement

The settlement of a RSS wall founded on compact embankment fill or native clays will depend on the thickness of the pad, the material used, the conditions of the subgrade and the quality of construction. At this site, settlements of RSS walls founded on well compacted engineered fill, or native, stiff silty clay subgrade prepared as recommended in this report, are expected to be not greater than 25 mm provided that the subgrade is preloaded as recommended in this report.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material. In cases where the approach embankment consists of rock fill, the backfill to the abutment wall should consist of OPSS Granular "B" Type II.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. 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 granular backfill to the abutment wall is used, 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.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993".

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.06.

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

13 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

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:

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

where P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see below)

γ = unit weight of retained soil (see below)

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 2.0 m for Granular B Type I or at a depth of 1.7 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 unfactored values are shown in the following table.

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I (modified) $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	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	0.40*	0.31	0.48*	0.2	.28*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	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. However, the use of Granular "B" Type I may be restricted if the approach embankment consist of rock fill.

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 CHBDC, 2000.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters are provided in Table A3.1.7 of the CHBDC for the Arnprior/Renfrew area:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.10
- Acceleration Related Seismic Zone 4
- Zonal Acceleration Ratio 0.20

The soils at this site consist of stiff to soft silty clays overlying compact sands and silts at some locations with a total overburden thickness ranging from 3 m to 6 m. The Soil Profile Type at these locations 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 1.0.

Site specific seismic hazard calculations for this area were obtained from Earthquakes Canada. For the design level earthquake (1 in 475 year event), the Peak Horizontal Ground Acceleration (PHA) is 0.184g, and the Peak Horizontal Ground velocity (PHV) is 0.091m/sec.

Clause C4.6.4 of the CHBDC suggests that the value of k_h used in calculating the earth pressure coefficients for yielding structures is equivalent to $0.5 \times$ Zonal Acceleration Ratio, A , (including the effects of site amplification), or 0.1g at this site, provided that allowance is made for an outward displacement of the abutment of up to $250A$, or 50 mm. The vertical acceleration factor, k_v , has been taken as 0.6 times k_h . This design assumption is based on conventional practice in seismic design. For non-yielding walls, the recommended k_h design value according to CHBDC is equivalent to $1.5 \times A$, or 0.3g at this site. The Woods method adopted in this report is based on elastic theory resulting in seismic earth pressure coefficients comparable to those outlined in the CHBDC.

14.2 Liquefaction Potential

Since the abutments are to be founded on native silty clay or on compacted embankment fill resting on silty clay, there is negligible potential for soil liquefaction under the foundations. The pier may be founded on bedrock which is no potential for liquefaction.

The approach embankments will be founded on silty clay above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur in the approach embankments but this is expected to be minor in nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC 2000, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that include the effects of earthquake loading. The following geotechnical parameters were used to calculate the seismic earth pressures :

ϕ = angle of internal friction of backfill

δ = angle of friction between the wall and the backfill

The seismic earth pressure coefficients to be used in design at this site are shown in table below.

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as Percentage of Wall Height	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17.5^\circ$		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})	40%	0.33	0.70	0.37	0.90*	0.26	0.40
Passive (K_{PE})	33%	3.5	-	3.0	-	4.8	-
At Rest (K_{OE})**	45%	0.67	-	0.72	-	0.58	

* Slope may undergo movement for short durations during seismic activities

** After Woods

15 CONSTRUCTION CONCERNS

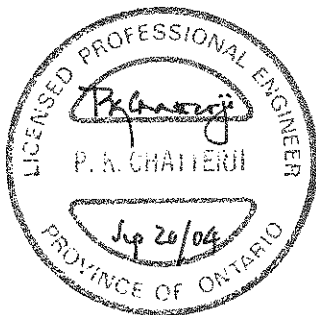
During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- post construction settlement at the approach embankments due to loading from new fill
- potential for encountering boulders and cobbles during piling or caisson installation procedures
- disturbance of the bedrock under the foundations due to excavation and other procedures
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation.



Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.
Senior Project Engineer



Report Reviewed by:
P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

TABLE 1
Campbell Drive Underpass
Point Load Test Results

Depth		m	Is50	UCS (MPa)				
feet	Inches							
CAM-3								
16	7	5.05	0.70	16.85	}	Total Rock Core		
17	5	5.31	3.99	95.86		Average	Minimum	Maximum
17	11	5.46	4.87	116.92		97	17	145 MPa
19	8	5.99	5.35	128.51				
20	10	6.35	4.17	100.07		Run #	Average	
21	9	6.63	6.06	145.36		1	16.85	
23	5	7.14	3.47	83.22		2	104.55	
24	6	7.47	3.73	89.54		3	96.91	
25	3	7.70	4.04	96.91				

Depth		Is50	UCS (MPa)					
feet	Inches							m
CAM-4								
15	6	4.72	4.52	108.50	}	Total Rock Core		
17	1	5.21	1.05	25.28		Average	Minimum	Maximum
18	3	5.56	3.51	84.27		104	25	138 MPa
19	6	5.94	5.62	134.83				
20	4	6.20	4.87	116.92		Run #	Average	
21	11	6.68	4.78	114.82		1	88.22	
22	9	6.93	4.61	110.60		2	120.08	
23	11	7.29	5.75	137.99				

Note: Point load test at 5.21 m was performed at hidden joint

Depth			Is50	UCS (MPa)	}				
feet	Inches	m				Average	Minimum	Maximum	
CAM-7						154	40	218	MPa
13	7	4.14	13.17	316.01		Run #	Average		
15	4	4.67	6.58	158.00					
16	2	4.93	6.80	163.27					
17	6	5.33	9.00	215.94		1	188.82		
18	5	5.61	9.09	218.05					
18	11	5.77	8.51	204.35					
19	9	6.02	2.85	68.47		2	115.61		
21	1	6.43	1.67	40.03					
22	4	6.81	6.23	149.58					
23	0	7.01	7.15	171.70	3	171.70			

Note: Point load test at 4.14 m was performed in Granitic cobble
Point load test at 6.43 m was performed at hidden joint

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

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

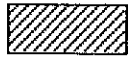




 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>		
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250 Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m			
Medium bedded	0.2 to 0.6m	Very Strong	100-250 15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m			
Very thinly bedded	20 to 60mm	Strong	50-100 7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm			
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0 3,500 to 7,500	Breaks under single blow of geological hammer.
<u>TERMS</u>		Weak	5.0 to 25.0 750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0 150 to 750	Can be peeled by a pocket knife, 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.	Extremely Weak (Rock)	0.25 to 1.0 35 to 150	Indented by thumbnail
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.			
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen			
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.			



THURBER

RECORD OF BOREHOLE No CAM-1

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 886.5 E 312 127.0 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29.09.03 - 29.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
113.7														
113.9	TOPSOIL (175mm)													
0.2	Silty CLAY, with thin sand seams, trace rootlets, occasional silt pockets Firm to Stiff Brown Moist (Cl)		1	SS	8		113							
			2	SS	12									0 6 79 15
112.0	Occasional sand lenses, trace gravel													
1.7	Silty SAND, trace gravel		3	SS	10		112							
111.7	Compact													
2.0	Brown Moist (TILL) END OF BOREHOLE AT 2.03m. AUGER REFUSAL AT 2.03m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM ON COMPLETION.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CAM-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 875.9 E 312 147.9 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29.09.03 - 29.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
113.3								20 40 60 80 100							
110.9	TOPSOIL (150mm)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
0.2	Silty CLAY, with thin sand seams, trace gravel, trace rootlets Stiff to Firm Brown Moist		1	SS	10		113								
			2	SS	8		112								
111.8															
1.5	Silty SAND, some clay, trace gravel Compact Brown Moist		3	SS	10										10 41 34 15
111.1	(TILL)														
2.2	END OF BOREHOLE AT 2.18m. AUGER REFUSAL AT 2.18m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM ON COMPLETION.														

+³ × 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CAM-3

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 867.6 E 312 123.4 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 29.09.03 - 29.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
113.2												
113.0	TOPSOIL (150mm)											
0.2	Silty CLAY, trace sand Stiff to Firm Brown Moist (Cl)		1	SS	9		113					
			2	SS	8		112					
			3	SS	6		111					
	Soft		4	SS	3		110					
			5	SS	2		109					
	Grey/ Brown						108					
108.9							107					
4.3	Sandy SILT, some clay, trace gravel Very Dense Grey		6	SS	50/0.076		106					
108.4	Wet (TILL)		1	RUN	>5		105					
4.8	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with dark grey and white subvertical banding, occasional iron oxide staining at joints and calcite infilling in vugs, moderately strong to very strong Subvertical joint from 4.8m to 4.95m, 5.11m to 5.18m, 5.56m to 5.71m, 5.84m to 5.92m, 6.4m to 6.45m, 6.76m to 6.88m Broken rock core from 4.95m to 5m, 5.18m to 5.25m		2	RUN	0		104					
			3	RUN	4		103					
			4	RUN	2		102					
105.3							101					
7.9	END OF BOREHOLE AT 7.87m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 2.13m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 113.1 18/12/03 113.1 04/02/03 112.9				FI		100					

ONTMT4 7450CAM.GPJ 16/03/04

RECORD OF BOREHOLE No CAM-4

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 831.3 E 312 181.1 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 26.09.03 - 26.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
113.3								20 40 60 80 100						
113.8	TOPSOIL (100mm)							○ UNCONFINED + FIELD VANE						
0.1	Silty CLAY, with thin sand seams Stiff to Firm Brown Moist (CI)		1	SS	12		113	● QUICK TRIAXIAL × LAB VANE						
			2	SS	12		112							
			3	SS	7		111							
	Soft		4	SS	4		110							
	occasional silt pockets Grey		5	SS	1		109							0 3 62 35
108.8					FI		109							
4.5	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, thinly bedded, grey with dark grey and white and occasional iron oxide staining at joints, strong to very strong Subvertical joint at 5.18m, from 5.26m to 5.31m, 5.82m to 5.87m Multiple subvertical and vertical joints from 4.9m to 5.16m, 6.04m to 6.15m, 6.32m to 6.53m		1	RUN	2 >5 2 2 2		108							RUN 1# TCR=100%, SCR=100%, RQD=72%, UCS=38MPa
			2	RUN	>5 5 2 4 2		107							RUN 2# TCR=92%, SCR=92%, RQD=39%, UCS=120MPa
105.7							106							
7.6	END OF BOREHOLE AT 7.57m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 2.13m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 112.7 18/12/03 112.9 04/02/04 112.5													

ONTMT4 7450CAM.GPJ 16/03/04

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CAM-5

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 824.3 E 312 160.9 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 26.09.03 - 26.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
112.7								SHEAR STRENGTH kPa						
112.6	TOPSOIL (100mm)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
0.1	Silty CLAY		1	SS	11									
	Stiff						112							
	Brown		2	SS	10									
	Moist						111							
	(Cl)		3	SS	6									
	becoming soft and grey													
	occasional silt pockets		4	SS	3		110							0 4 63 33
			5	SS	2		109							
108.3														
4.4	END OF BOREHOLE AT 4.42m. AUGER REFUSAL AT 4.42m ON PROBABLE BEDROCK OR BOULDERS.													

+³ ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CAM-6

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 786.5 E 312 221.7 (Campbell Drive) ORIGINATED BY JL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 16.10.03 - 16.10.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20	40	60		
114.3												
0.0	Gravelly SAND Compact Brown Moist (FILL)		1	SS	20							
113.6												
0.7	Silty CLAY, with organics, trace rootlets, occasional peat inclusions		2	SS	10							
113.4	Black Silty CLAY, occasional oxide lenses											
0.9	Stiff to Firm Grey Moist (CL)		3	SS	7							
			4	SS	6							
			5	SS	5							
	trace gravel, trace sand		6	SS	4							
	Soft		7	SS	2							

ONTMT4 7450CAM.GPJ 16/03/04

RECORD OF BOREHOLE No CAM-7

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 779.7 E 312 194.3 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 25.09.03 - 25.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
113.2												
112.8												
0.0	TOPSOIL (25mm)		1	SS	18							
	Silty CLAY		2	SS	9							
	Stiff to Firm											
	Brown											
	Moist											
			3	SS	5							
			4	SS	3							
	Soft, Grey		5	SS	1							
109.6												
3.6	Sandy SILT, some clay, trace gravel											
	Inferred Loose											
109.1	Grey				FI							
4.1	Wet		1	RUN	>5							
	(TILL)				>5							
	CRYSTALLINE LIMESTONE (BEDROCK)		2	RUN	5							
	Fresh, slightly to moderately weathered at joints, very thinly to thinly bedded, grey with dark grey and white subvertical banding, very strong				2							
	Broken zone from 4.19m to 4.57m				2							
	Subvertical joint at 4.7m, 4.83m, from 5.26m to 5.28m, 6.07m to 6.14m, 6.63m to 6.68m, 7.01m				0							
	Multiple subvertical joints from 4.78m to 4.9m with some calcite infilling		3	RUN	3							
					4							
					2							
					3							
106.0												
7.2	END OF BOREHOLE AT 7.16m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 2.13m slotted screen.											
	WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 112.9 18/12/03 Frozen at 112.9											

ONTMT4 7450CAM.GPJ 16/03/04

RECORD OF BOREHOLE No CAM-8

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 767.1 E 312 216.7 (Campbell Drive) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 25.09.03 - 25.09.03 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE	20	40
113.0							20	40	60	80	100	20	40	60				
112.8	TOPSOIL (75mm)		1	SS	2													
0.1	Silty CLAY Soft to Stiff Brown Moist some silt pockets/ seams		2	SS	14													
			3	SS	8													
			4	SS	4													
	trace gravel Soft, grey, wet		5	SS	3										1 8 70 22			
			6	SS	1													
107.1																		
5.9	END OF BOREHOLE AT 5.94m. AUGER REFUSAL AT 5.94m ON PROBABLE BEDROCK OR BOULDERS.																	

+ 3, × 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CAM-8A

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 032 766.8 E 312 216.7 (Campbell Drive) ORIGINATED BY JL
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 12.03.04 - 12.03.04 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
113.0 0.0	Augered without sampling to 4.0m depth						113					
							112					
							111					
							110					
			1	TW	WH		109					
108.0 5.0	AUGER REFUSAL AT 5.03m. BOREHOLE OPEN AND WET AT 3.66m UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS AND SEALED WITH BENTONITE AT SURFACE.						108					

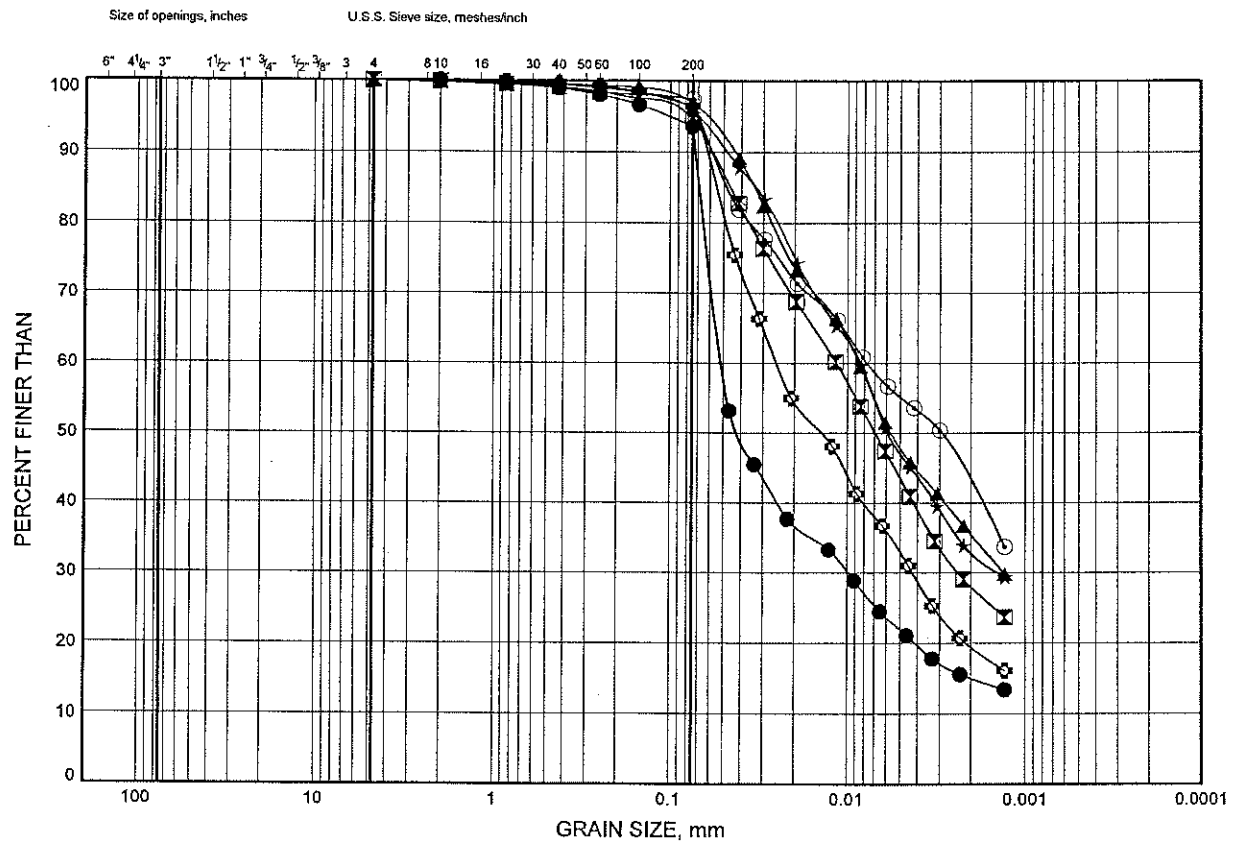
Appendix B

Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

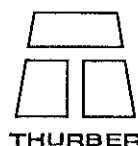
SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CAM-1	1.07	112.63
⊠	CAM-3	2.59	110.58
▲	CAM-4	3.35	109.92
★	CAM-5	2.59	110.08
⊙	CAM-6	1.83	112.47
⊛	CAM-7	1.83	111.37

Date March 2004
Project 647-92-00



Prep'd SS
Chkd. SP

FIGURE B2

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

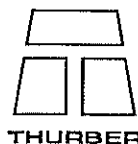
GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (Solid Circles)	Percent Finer Than (Open Squares)
100	100	100
10	100	100
1	98	98
0.75	95	95
0.6	92	92
0.425	90	88
0.3	85	75
0.25	80	65
0.2	75	55
0.15	68	45
0.125	62	38
0.106	58	32
0.09	52	28
0.075	48	22
0.06	40	18

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CAM-6	4.88	109.42
☒	CAM-8	3.35	109.65

Date March 2004
Project 647-92-00

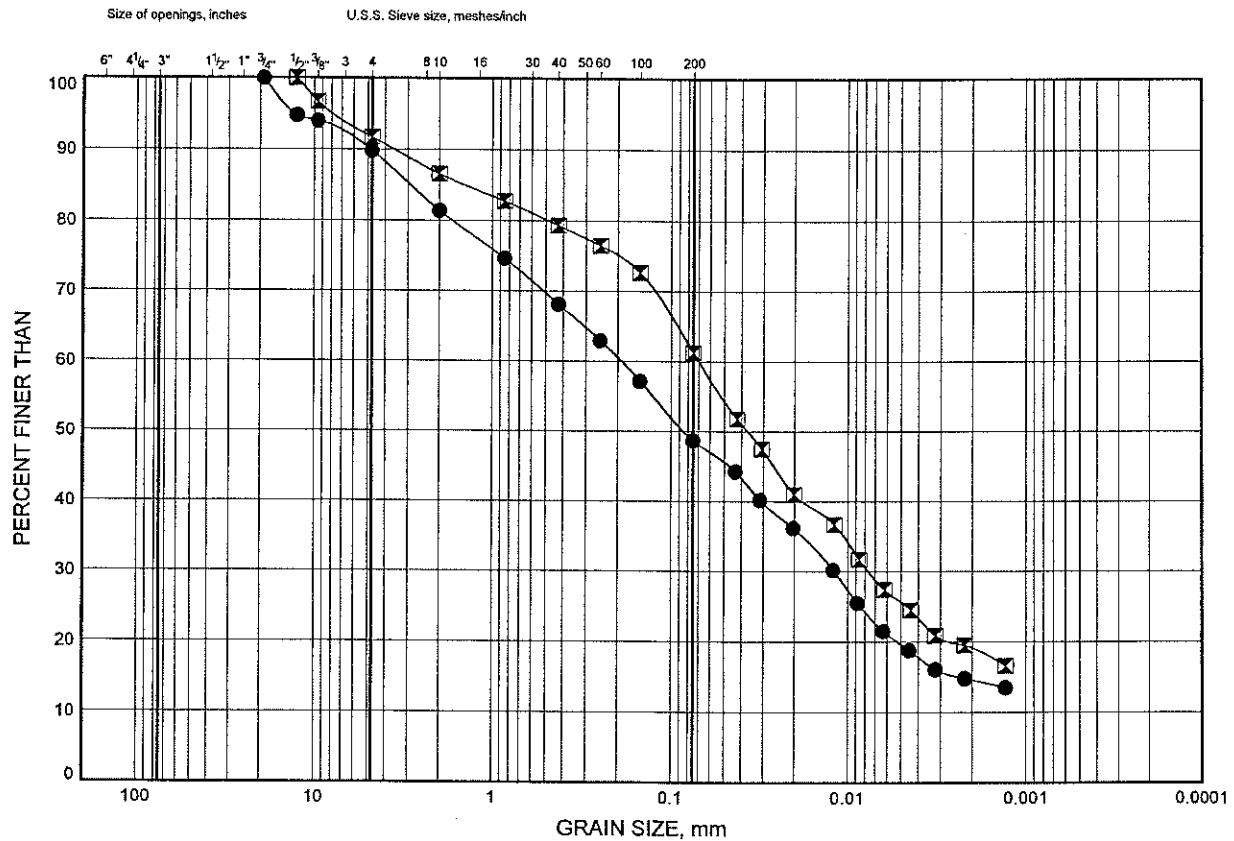


Prep'dSS.....
Chkd.SP.....

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B3

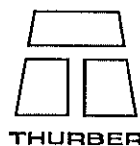
SANDY SILT TO SILTY SAND (TILL)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CAM-2	1.83	111.45
⊠	CAM-3	4.72	108.45

Date March 2004
Project 647-92-00

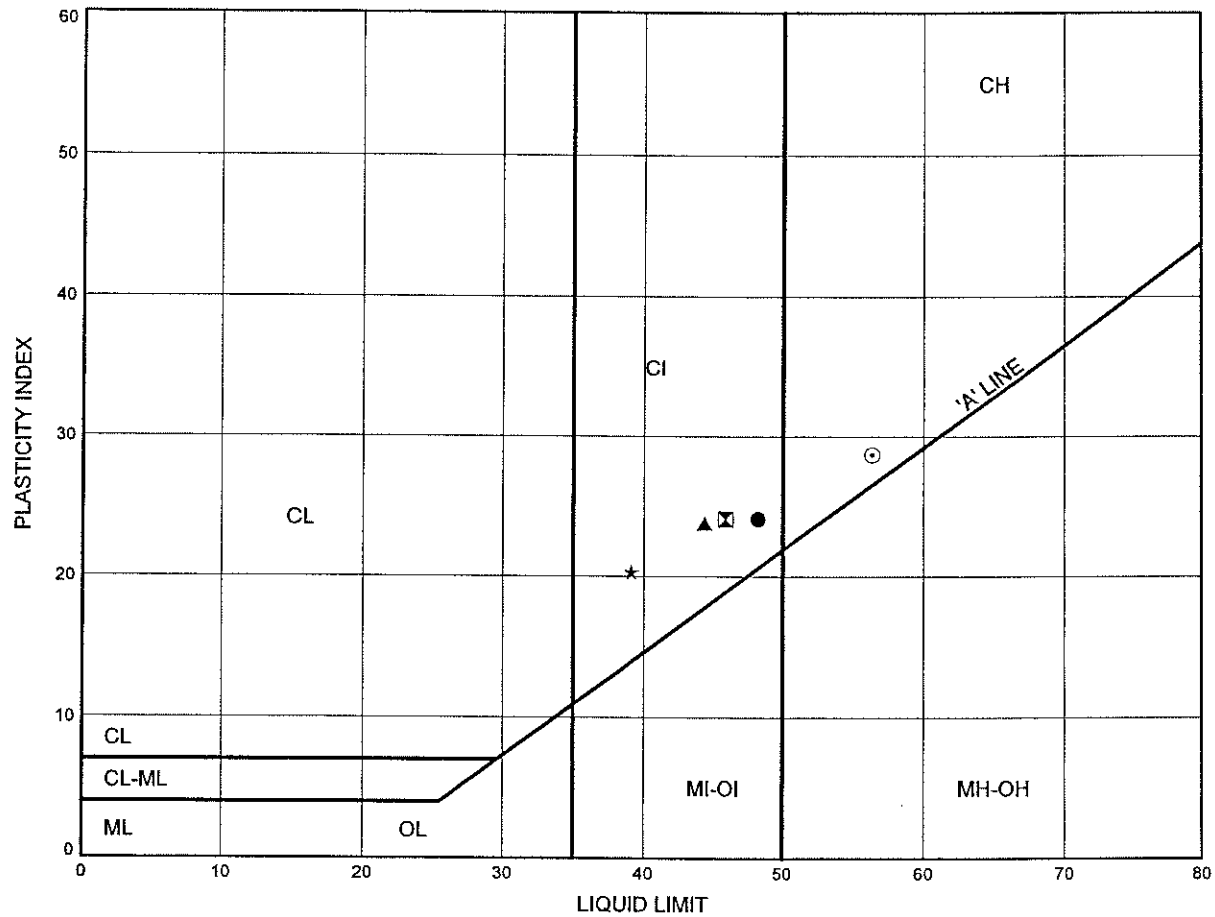


Prep'd SS
Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

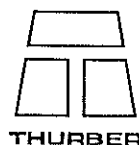
FIGURE B4

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CAM-1	1.07	112.63
⊠	CAM-3	2.59	110.58
▲	CAM-5	2.59	110.08
★	CAM-5	3.35	109.32
⊙	CAM-6	1.83	112.47

Date March 2004
 Project 647-92-00

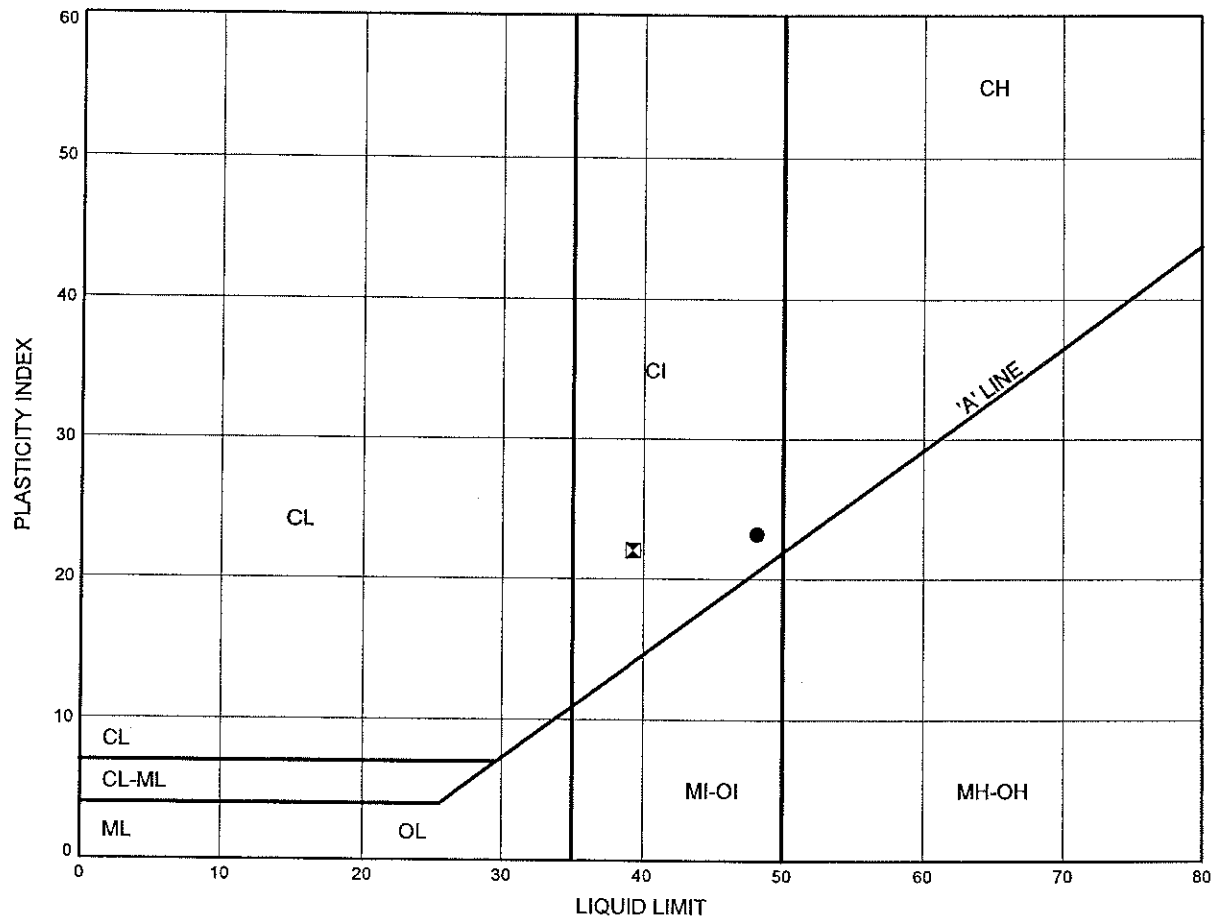


Prep'd SS
 Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

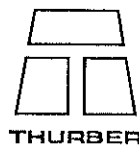
FIGURE B5

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CAM-7	1.83	111.37
⊗	CAM-8	3.35	109.65

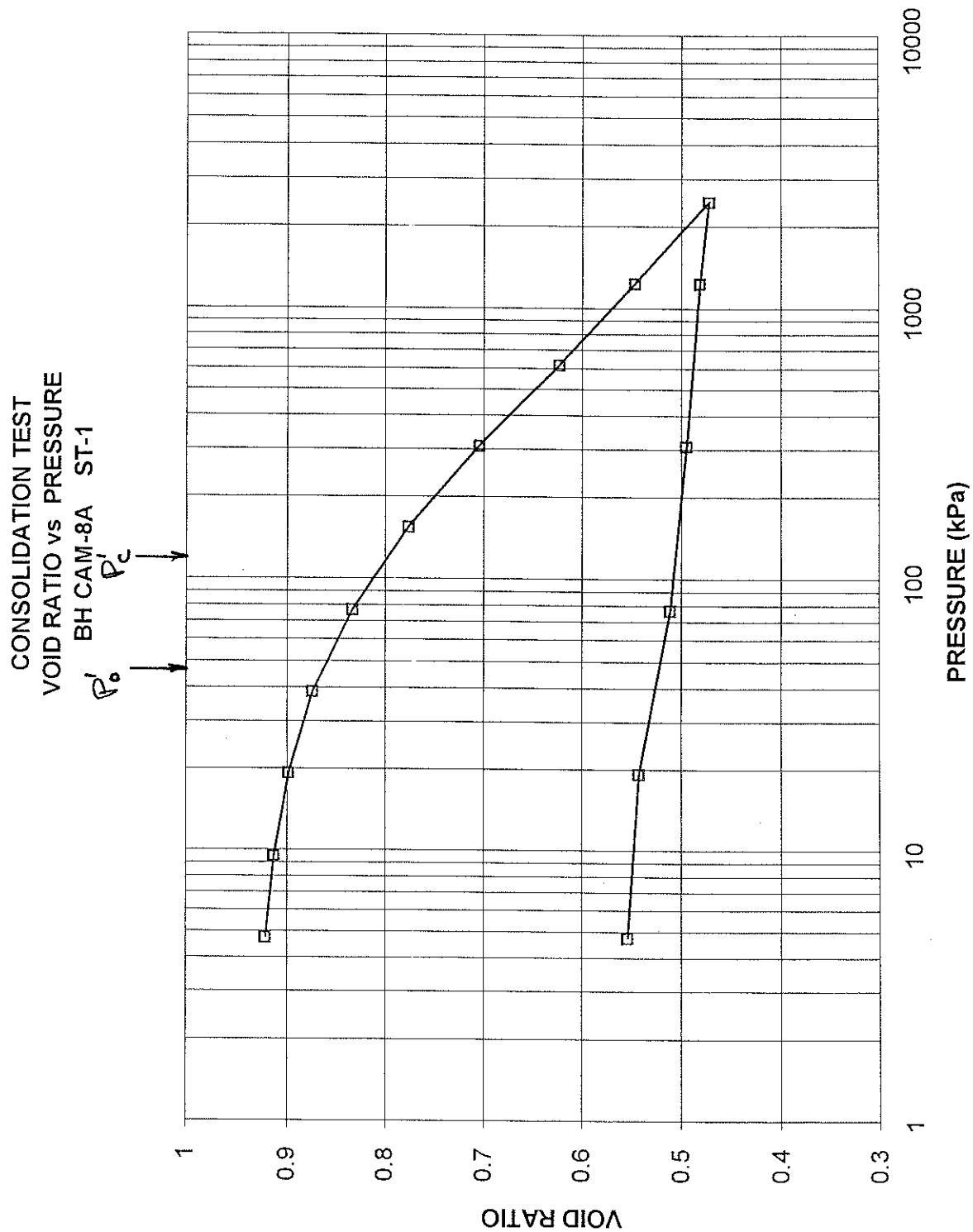
Date March 2004
 Project 647-92-00



Prep'd SS
 Chkd. SP

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE B6



OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1116-026	Sample Number	ST-1
Borehole Number	CAM-8A	Sample Depth, m	4.1-4.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	03/18/2004		
Date Completed	03/28/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	18.72
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.17
Area, cm ²	31.67	Specific Gravity, measured	2.79
Volume, cm ³	60.17	Solids Height, cm	0.984
Water Content, %	32.09	Volume of Solids, cm ³	31.16
Wet Mass, g	114.85	Volume of Voids, cm ³	29.01
Dry Mass, g	86.95	Degree of Saturation, %	96.2

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	1.900	0.931	1.900				
4.75	1.891	0.922	1.896	13	5.86E-02	9.97E-04	5.73E-06
9.54	1.883	0.913	1.887	21	3.59E-02	8.79E-04	3.10E-06
19.25	1.868	0.898	1.876	34	2.19E-02	8.13E-04	1.75E-06
38.68	1.844	0.874	1.856	17	4.30E-02	6.50E-04	2.74E-06
77.38	1.804	0.833	1.824	17	4.15E-02	5.44E-04	2.21E-06
154.68	1.748	0.776	1.776	28	2.39E-02	3.81E-04	8.92E-07
309.04	1.677	0.704	1.713	53	1.17E-02	2.42E-04	2.78E-07
618.33	1.597	0.623	1.637	113	5.03E-03	1.36E-04	6.71E-08
1237.24	1.522	0.547	1.560	103	5.01E-03	6.38E-05	3.13E-08
2472.00	1.449	0.472	1.486	103	4.54E-03	3.11E-05	1.39E-08
1237.24	1.458	0.482	1.454				
309.04	1.471	0.495	1.465				
77.38	1.487	0.511	1.479				
19.25	1.518	0.543	1.503				
4.75	1.529	0.554	1.524				

Notes:

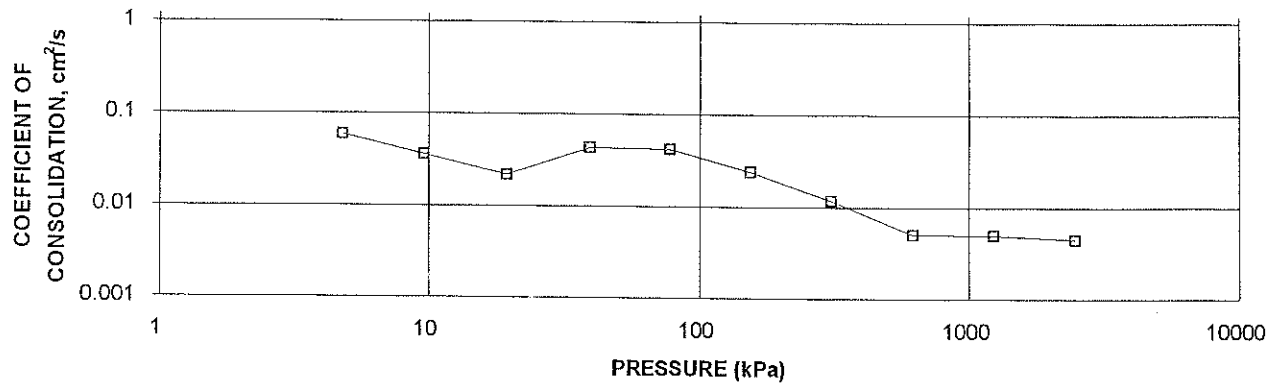
k calculated using cv based on \log values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

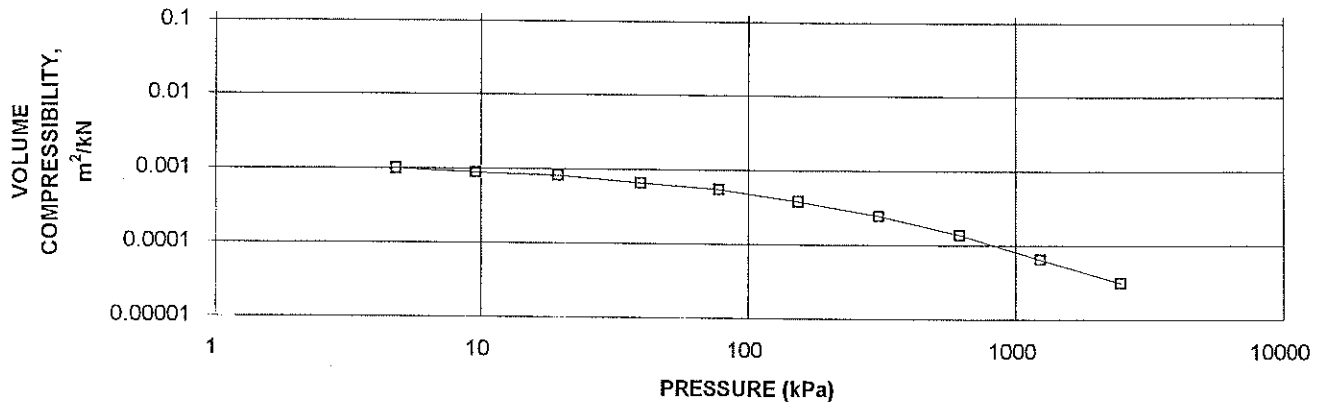
Sample Height, cm	1.53	Unit Weight, kN/m ³	21.19
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.61
Area, cm ²	31.67	Specific Gravity, measured	2.79
Volume, cm ³	48.42	Solids Height, cm	0.984
Water Content, %	20.36	Volume of Solids, cm ³	31.16
Wet Mass, g	104.65	Volume of Voids, cm ³	17.26
Dry Mass, g	86.95		

OEDOMETER CONSOLIDATION SUMMARY

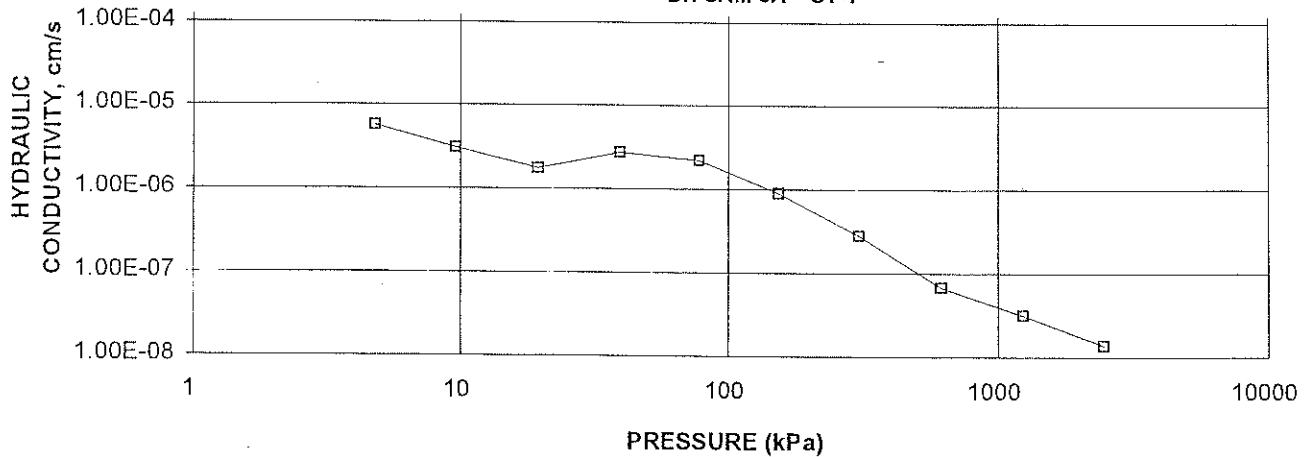
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH CAM-8A ST-1



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH CAM-8A ST-1



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH CAM-8A ST-1



Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

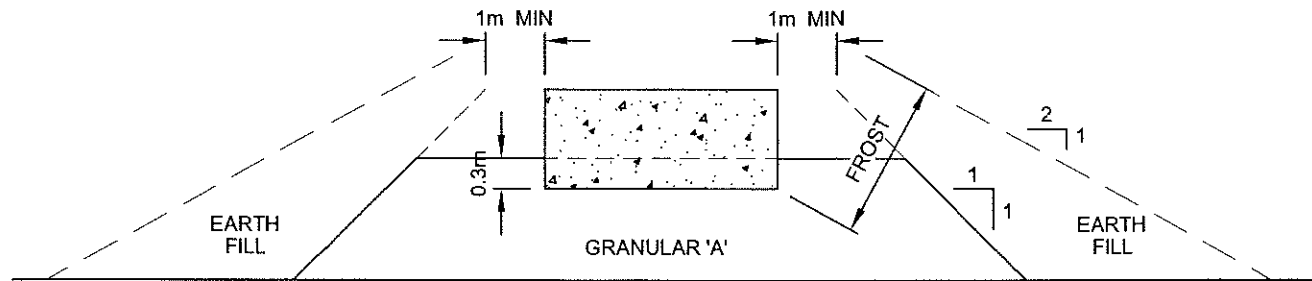
Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Caisson
North Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> i. Required if an integral abutment design is pursued. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface rendering the use of driven piles impractical, although technically possible. ii. Piles may have to be socketted into rock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Due to the proposed height of approach fills (in the order of 10 m), the required dimensions of the abutment wall would become impractical and uneconomical. ii. Subexcavation of native soils required. iii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more practical footing dimensions. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Significant long term settlements as the foundation clay is compressed under the embankment and footing loads. ii. Lower geotechnical resistance than bedrock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. ii. No excavation required for foundation construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Engineering fill pad is required as a platform to accommodate caisson installation equipment. ii. Nominal rock socketting is required to enhance seating on bedrock.
Centre Pier	<p>Advantages:</p> <ul style="list-style-type: none"> None identified. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface rendering the use of driven piles unnecessary and impractical. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface below existing ground surface. ii. High values of geotechnical resistance are available on the bedrock <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Stepped footing may be required. 	<p>Advantages:</p> <ul style="list-style-type: none"> None identified. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface making the use of engineered fill unnecessary. ii. Lower geotechnical resistance than bedrock. 	<p>Advantages:</p> <ul style="list-style-type: none"> None identified <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface rendering the use of caisson unnecessary, although technically possible.

Campbell Drive Underpass
Highway 17 Twinning, Arnprior to Renfrew

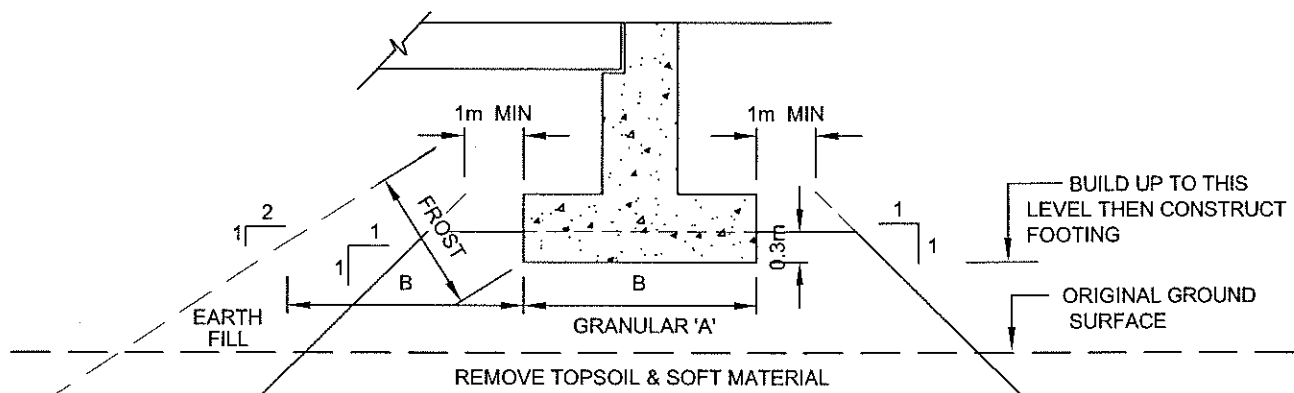
		<p>ii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface.</p>		
<p>South Abutment</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Required if an integral abutment design is pursued. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface rendering the use of driven piles unnecessary, although technically possible. ii. Piles may have to be socketted into rock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface below existing ground surface. ii. High values of geotechnical resistance are available on the bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Due to the proposed height of approach fills (in the order of 10 m), the required dimensions of the abutment wall would become impractical and uneconomical. ii. Subexcavation of native soils required. iii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more practical footing dimensions. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Significant long term settlements as the foundation clay is compressed under the embankment and footing loads. ii. Lower geotechnical resistance than bedrock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. iii. No excavation required for foundation construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Engineering fill pad is required as a platform to accommodate caisson installation equipment. ii. Nominal rock socketting is required to enhance seating on bedrock.

Appendix D

Figure and Table



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.

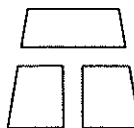
ENGINEER	SP	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	 THURBER
DRAWN	SS		
DATE	March , 2004		
APPROVED	PKC		
SCALE	NTS		
			DWG. NO. FIGURE D1

TABLE D1

The annular space surrounding the piles in an integral abutment design should be filled with sand meeting the following gradation requirements :

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

Appendix E

Special Provisions

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.

AMENDMENT TO OPSS 120, AUGUST, 1994

Special Provision

OPSS 120, August 1994, General Specification For The Use of Explosives is deleted in its entirety and replaced with the following:

Construction Special Provision for Rock Excavation Utilizing Blasting

120.01 SCOPE

This special provision describes the conditions under which explosives are to be used on the Contract.

120.02 REFERENCES

This special provision refers to the following standards, special provisions or publications:

Canadian Standards Association:

CAN3-Z107.54-M85(R 1999) – Procedure for Measurement of Sound and Vibration Due to Blasting Operations

Ministry of Transportation Publications:

Ontario Traffic Manual Book 7

Federal Government Publication:

Explosives Act (Canada)

Department of Fisheries and Oceans Publication:

Guidelines for the Use of Explosives in or near Canadian Fisheries Waters, 1998

120.03 DEFINITIONS

For the purposes of this special provision, the following definitions apply:

Blaster: means a competent person knowledgeable and experienced in the handling, use and storage of explosives and their effect on adjacent property and persons

Blasting Consultant: means an Engineer, with a minimum of five (5) years experience related to blasting design, blasting operations and monitoring of rock excavation blasting. The Blasting Consultant 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.

Fugitive Flyrock: means broken rock that is dislodged from the parent bedrock formation during the blasting operation and escapes the blasting rock control measures

Peak Particle Velocity(PPV): means the maximum speed that ground particles move as a result of energy released from explosive detonations. PPV is measured in millimeters per second.

Pre-Blast Survey: means a detailed record, in written form accompanied by film or video, of the condition of private or public property, prior to the commencement of blasting operations.

Rock Excavation: means the removal of material from solid masses of igneous, sedimentary or metamorphic rock which prior to removal was integral with the parent mass and the removal of boulders and rock fragments larger than 1.0 cubic metre in volume.

120.04 SUBMISSION AND DESIGN REQUIREMENTS

120.04.01 General

The Contractor shall submit the following to the Contract Administrator a minimum of five (5) working days prior to the use of explosives:

- a) The name and qualifications of the blasting contractor.
- b) The names of the blasting supervisor and blasters in charge of the blasting including a record of their experience and safety training.
- c) A detailed plan of the blasting operation and schedule of excavation.
- d) Blasting Permits, Approvals and/or Agreements
- e) Name and qualifications of the Blasting Consultant.
- f) A blasting design and blasting vibration and noise monitoring program.
- g) A pre-blast survey.
- h) A trial blasting program.

120.04.02 Blasting Design and Monitoring

120.04.02.01 Blasting Consultant

The Contractor shall retain the services of a Blasting Consultant to produce a blasting design and blasting vibration and noise monitoring program.

120.04.02.02 Blasting Design

A blasting design shall be submitted to the Contract Administrator and shall include but not be limited to the following:

- a) Design peak particle velocity (PPV) and design peak sound pressure level.
- b) Number, pattern, orientation, size and depth of drill holes

- c) Collar and toe load; number and time of delays, distribution of explosives per hole, per delay and per blast and the sequence and pattern of the delays.
- d) Setback distances to affected waterbodies, substrate type and the explosive products to be used.

The following shall be submitted to the Contract Administrator 48 hrs prior to the use of explosives:

- a) A letter signed by the Blast Consultant indicating the areas for which a blast design has been completed.
- b) A letter signed by the Blaster indicating receipt of the blast design and agreement that the blasting shall be according to the design.

120.04.02.03 Blasting Monitoring

A detailed blasting monitoring program to monitor ground vibrations, water pressure and noise for each blast shall be produced and submitted to the Contract Administrator. The blasting monitoring shall be carried out by the Blasting Consultant

The blasting monitoring equipment shall meet the following requirements:

- a) Be capable of measuring and recording ground vibrations up to 200 mm/s of peak particle velocity (PPV) in each of three (3) mutually perpendicular directions.
- b) Be capable of measuring and recording frequency in all three (3) mutually perpendicular directions.
- c) Be capable of monitoring impact noise levels from the blasting up to a minimum 142 dbf.
- d) Instrumentation shall have been calibrated within six (6) months prior to commencement of blasting operations. Proof of calibration shall be submitted to the Contract Administrator five(5) working days prior to the commencement of blasting operation.

120.04.03 Blasting Permits, Approvals and/or Agreements

Any blasting permits, approvals and/or agreements shall be obtained by the Contractor and submitted to the Contract Administrator at no additional cost to the Owner.

120.04.04 Pre-Blast Survey

Prior to any blasting operation, the Contractor shall conduct a pre-construction inspection of all buildings, structures, utilities, residences and facilities within a minimum 100 m radius of the location where explosives are to be used and submit a pre-blast survey certificate. The pre-blast survey certificate shall be signed by the Blasting Consultant and shall be submitted to the Contract Administrator 48 hours prior to the use of explosives.

A formal request for permission to carry out an inspection with an explanatory letter to the owner shall be produced. In the event that a property owner refuses to permit inspection, the property owner shall be asked to sign a refusal to inspect release form and the Contract Administrator shall be informed prior to the use of explosives.

The pre-blast survey report shall include as a minimum the following information:

- a) Type of structure or utility, including type of construction and the date, if possible, when built.
- b) Any differential settlements: visible cracks in walls, floors and ceiling shall be identified and described, including a diagram, if applicable.
- c) Any other apparent structural or cosmetic damage or defect.
- d) As a minimum, clear quality photographs, as deemed necessary for proper recording of significant concerns.

120.04.05 Trial Blasting

The Contractor shall undertake trial blasting based on the proposed blasting design as produced by the Blasting Consultant to demonstrate that the blast design satisfies the vibration and noise level limits specified in the contract.

The Contractor shall carry out a series of a minimum of three (3) trial blasts based on the Contractor's blasting design. These trial blasts shall be carried out at a location selected by the Contractor and approved by the Contract Administrator.

The trial blasting shall be carried out with the appropriate monitoring of blast vibrations and noise levels in the presence of the Contract Administrator. A report summarizing the trial blast monitoring results shall be submitted to the Contract Administrator signed and sealed by the Blasting Consultant.

The Contractor shall review the trial blast results and revise the blasting design as required to ensure satisfactory levels of blast vibrations, noise, shatter depth and to prevent flyrock.

120.04.06 Post Blast Survey and Reporting

Following the blasting operation, the Contractor shall conduct a post-construction inspection of all buildings, structures, utilities, residences and facilities within a minimum 100 m radius of the location where explosives were used to determine if any damage has resulted from the blast. A post-construction survey certificate signed by the Blasting Consultant shall be submitted to the Contract Administrator ten(10) working days following the blasting operation.

A blast report summarizing the results of the vibration and air blast levels signed and sealed by the Blasting Consultant shall be submitted to the Contract Administrator at the end of each work day in which blasting is carried out.

120.04.07 Certificate of Conformance

Upon completion of the blasting work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Blasting Consultant. The certificate shall state that the work has been carried out in general conformance with the requirements of the contract .

120.05 MATERIAL

120.05.01 Explosives

Only explosive products that are approved for use in Canada shall be used.

120.05.02 Handling and Storage

Explosives and all detonating apparatus shall be handled and stored according to the Explosives Act (Canada) and any applicable Municipal By-laws.

120.06 EQUIPMENT

Only firing devices approved for use by the manufacturer of the selected detonator shall be used. All apparatus shall be thoroughly inspected before and after each blasting operation.

All wiring connected to electrical firing devices shall be properly insulated.

120.07 CONSTRUCTION

120.07.01 General

Blasting shall be carried out only during daylight hours and at any time when atmospheric conditions provide clear observation of the blasting site from a distance of 1,000m. Blasting shall not be conducted on Sundays, statutory holidays or during electrical storms

120.07.02 Safety Precautions

All safety precautions shall be observed prior to blasting, including the sounding of audible warning device before and after blasting as required. The Contractor shall be responsible for all claims arising from the transportation, handling, storage and use of explosives.

The Contractor shall comply with the requirements and limitations for protecting fish and fish habitat according to the Department of Fisheries and Oceans (DFO) Guidelines and as detailed elsewhere in the Contract.

Prior to blasting, investigations shall be done to determine if radio frequency hazards exists. Where they exist, necessary precautions shall be taken.

120.07.03 Notice

The Contractor Administrator shall be provided in writing with a blasting schedule a minimum of 48 hr prior to the start of blasting.

All departments or agencies of government, persons, partnerships and corporations affected shall be notified a minimum of 48 hrs prior to blasting

The Contractor shall provide notice to property owners within a 200 metre radius from the actual blasting site, a minimum of four (4) hours prior to the commencement of blasting.

120.07.04 Vibration Monitoring

Vibration monitoring shall be carried out, unless otherwise specified in the Contract Documents. Ground vibration levels shall be monitored 200 m from the blast site or at the closest structure within the radius during

each blast. Water pressure shall be monitored adjacent to the shore, closest to the blast site.

The Contractor shall limit PPV ground vibrations to the following maximum levels:

Structures

50 mm/s measured in one of three planes where the predominating frequency exceeds 40 Hz.

20 mm/s measured in one of three planes where the predominating frequency is below 40 Hz.

Fresh Concrete

For concrete or grout in place less than 60 hours, the maximum PPV due to blasting operations shall not exceed 10 mm/sec.

No blasting shall be carried within 30 m of freshly placed concrete for the following periods:

For concrete curing in ambient temperatures less than 20°C, 72 hours from completion of placement.

For concrete curing in ambient temperatures greater than 20°C, 24 hours from completion of placement.

120.07.05 Utilities

The Contractor shall provide written notification to all utility authorities within vicinity of the blasting operation a minimum of five (5) working days prior to the use of explosives.

120.07.06 Excessive Vibration Readings – Work Stoppage

Should any two (2) consecutive readings exceed the maximum PPV values specified in sub-section 120.07.04, all blasting operations shall cease. A new blast design shall be submitted to the Contract Administrator to reduce PPV values within the allowable limits prior to resuming the blasting operation.

WARRANT: Always when the use of explosives is permitted in the contract.

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.

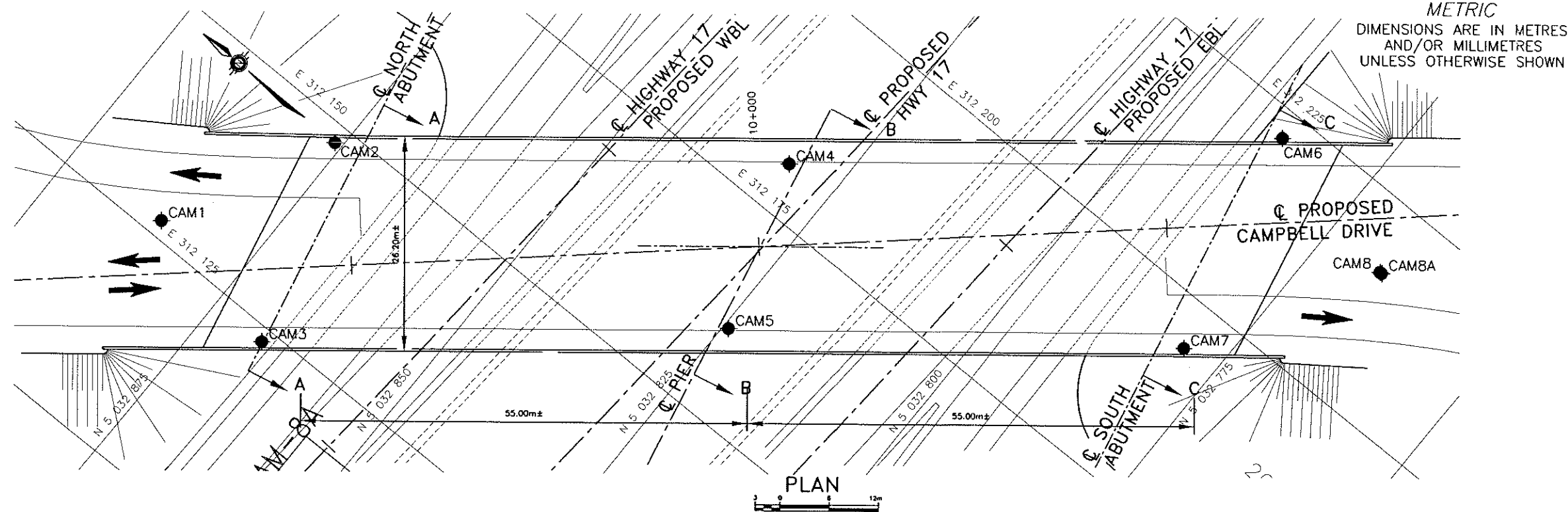
Voids on the top surface of the embankment shall be minimized to prevent migration of the roadway subbase and base into the rock fill embankment by chinking the top surface with rock fragments and spalls to form the subgrade prior to the placement of the roadway subbase. The Contractor shall chink the 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.

Appendix F

Drawing



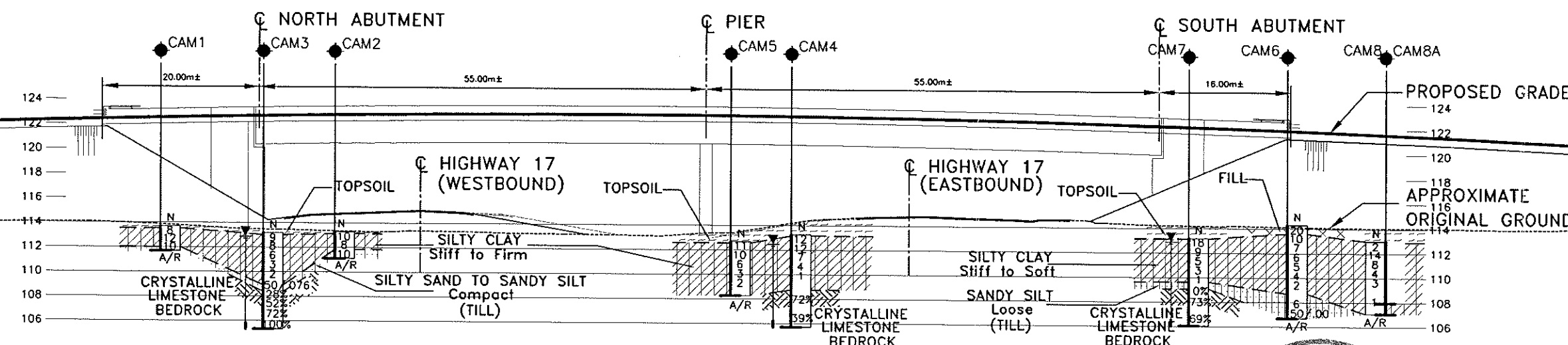
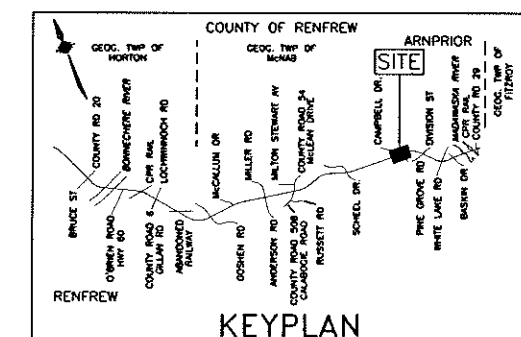
HWY.17
WP NO. 647-92-00

HIGHWAY 17 TWINNING
CAMPBELL DRIVE UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

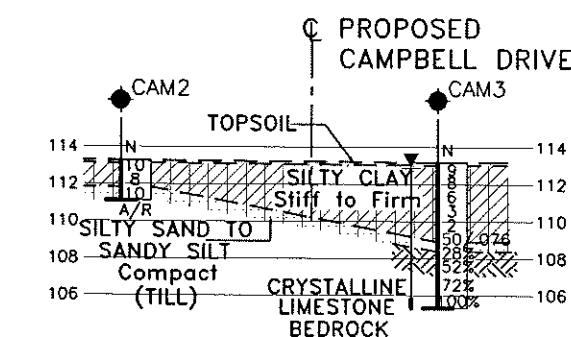
SHEET



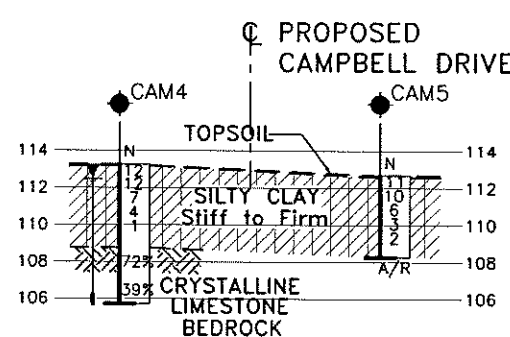
THURBER ENGINEERING LTD.



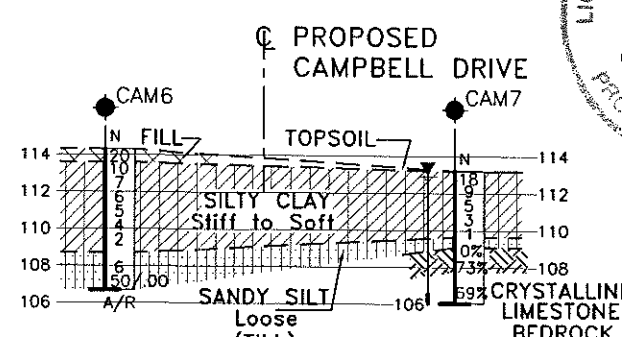
PROFILE @ CAMPBELL DRIVE



SECTION A-A



SECTION B-B

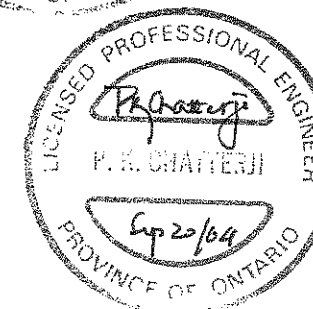
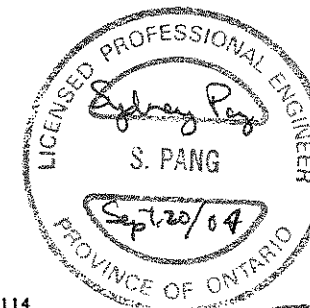


SECTION C-C

LEGEND			
●	Bore Hole		
⊕	Dynamic Cone Penetration Test (cone)		
⊗	Bore Hole & Cone		
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)		
CONE	Blows/ 0.3m (60' Cone, 475 J/blow)		
PH	Pressure, Hydraulic		
⬇	WL at Time of Investigation		
⬆	Head Artesian Water		
⬆	Piezometer		
90%	Rock Quality Designation (RQD)		
A/R	Auger Refusal		
NO	ELEVATION	NORTHING	EASTING
CAM1	113.7	5 032 886.5	312 127.0
CAM2	113.3	5 032 875.9	312 147.9
CAM3	113.2	5 032 867.6	312 123.4
CAM4	113.3	5 032 831.3	312 181.1
CAM5	112.7	5 032 824.3	312 160.9
CAM6	114.3	5 032 786.5	312 221.7
CAM7	113.2	5 032 779.7	312 194.3
CAM8	113.0	5 032 767.1	312 216.7

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

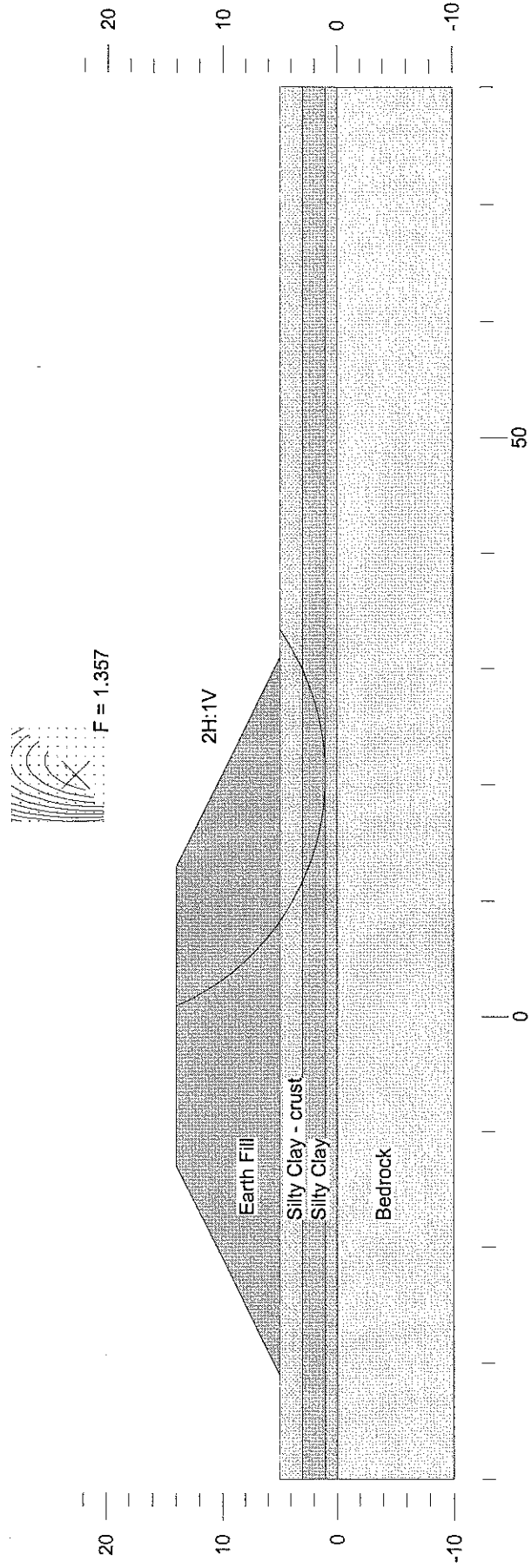
REVISIONS	DATE	BY	DESCRIPTION
SEP. 04	SP	FINAL	
MAR. 04	SP	ISSUED AS DRAFT FOR REVIEW	
DESIGN	SP	CHK PKC	CHBDC 2000
DRAWN	SS	CHK SP	SITE
			LOAD
			DATE SEPT.2004
			STRUCT
			DWG.




Appendix G

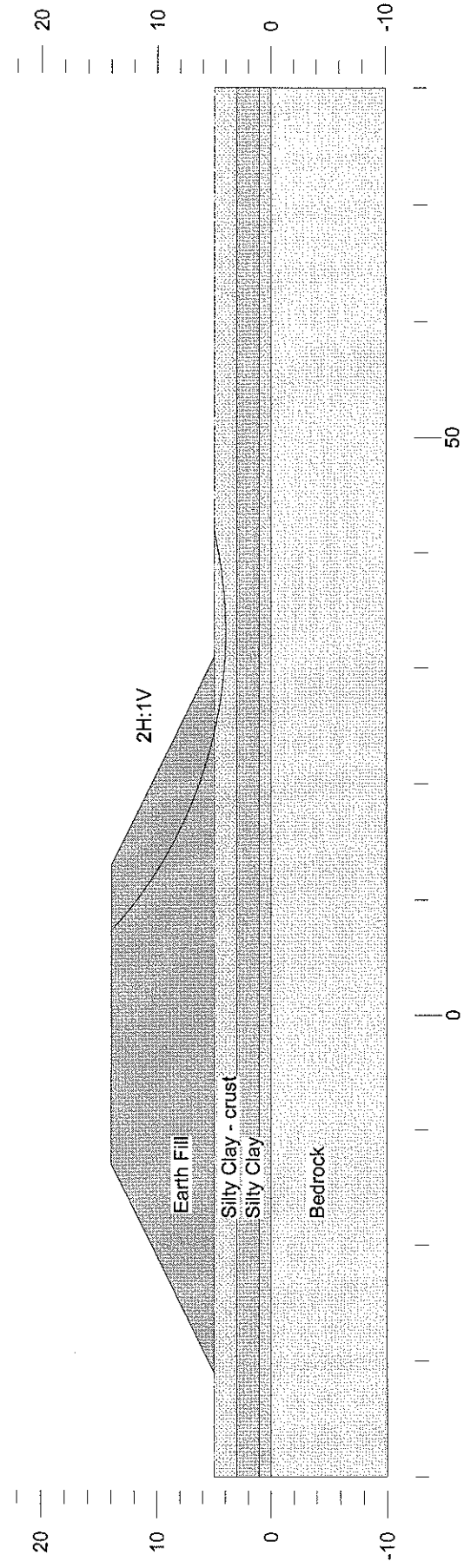
Selected Stability Analyses Results

	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Earth Fill	21	0	32	0	0
Silty Clay-crust	18	60	0	0	1
Silty Clay-firm	18	30	0	0	1
Sand-Silt Till	20	0	30	0	1
Bedrock	(Infinitely Strong)				

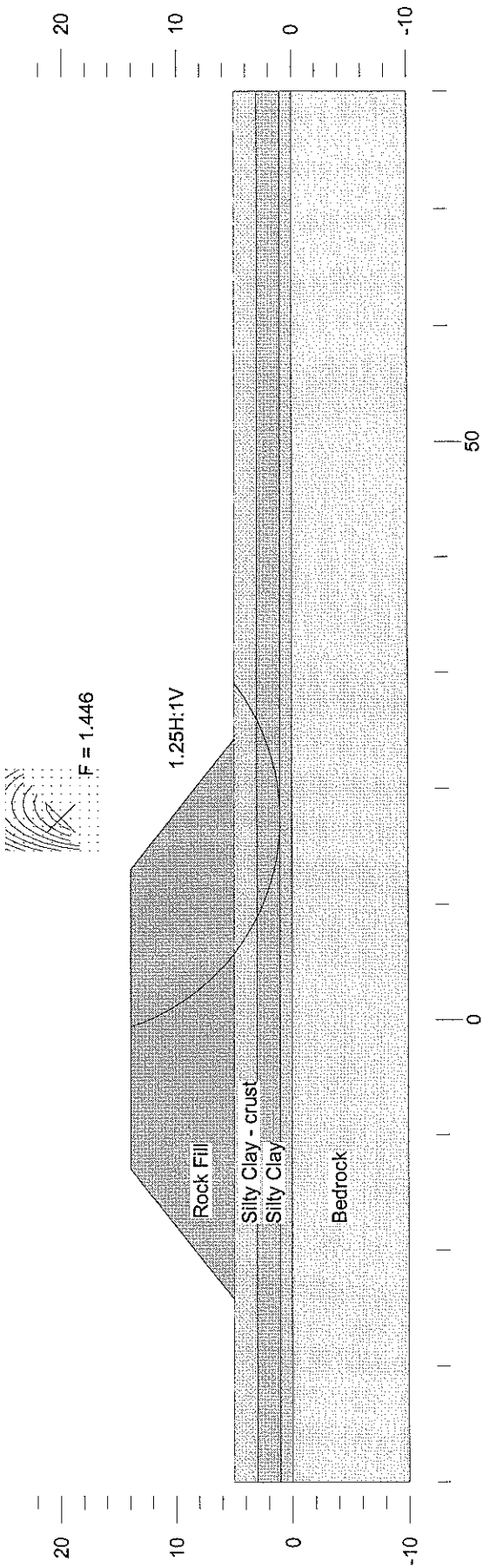


	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Earth Fill	21	0	32	0	0
Silty Clay-crust	18	0	28	0	1
Silty Clay-firm	18	0	27	0	1
Sand-Silt Till	20	0	30	0	1
Bedrock	(Infinitely Strong)				

 F = 1.580

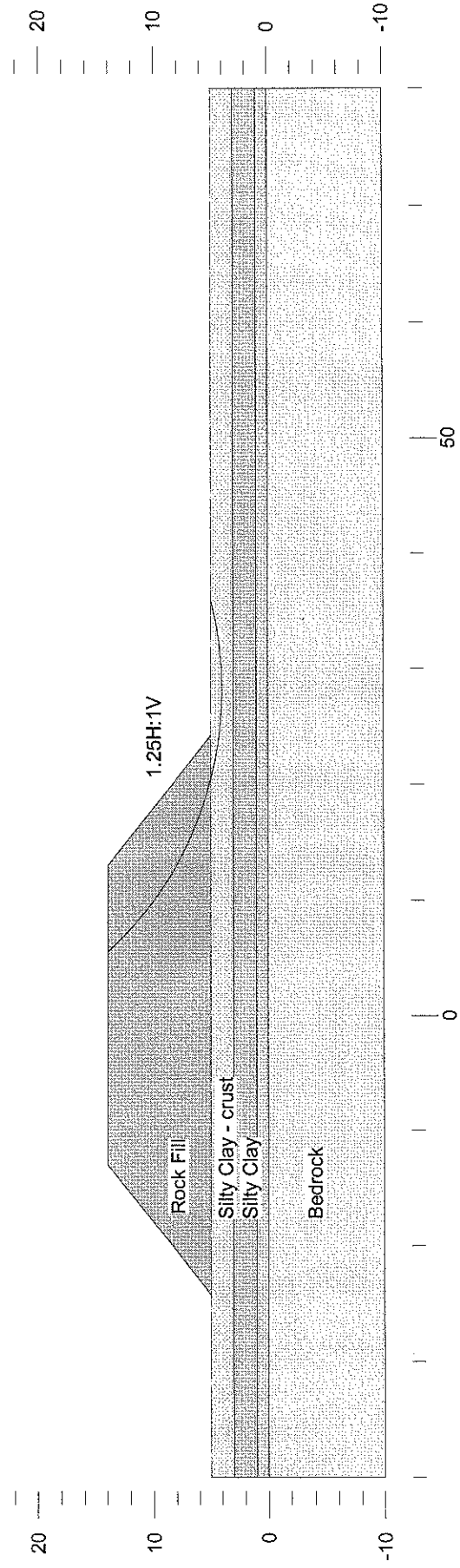


	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	1
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

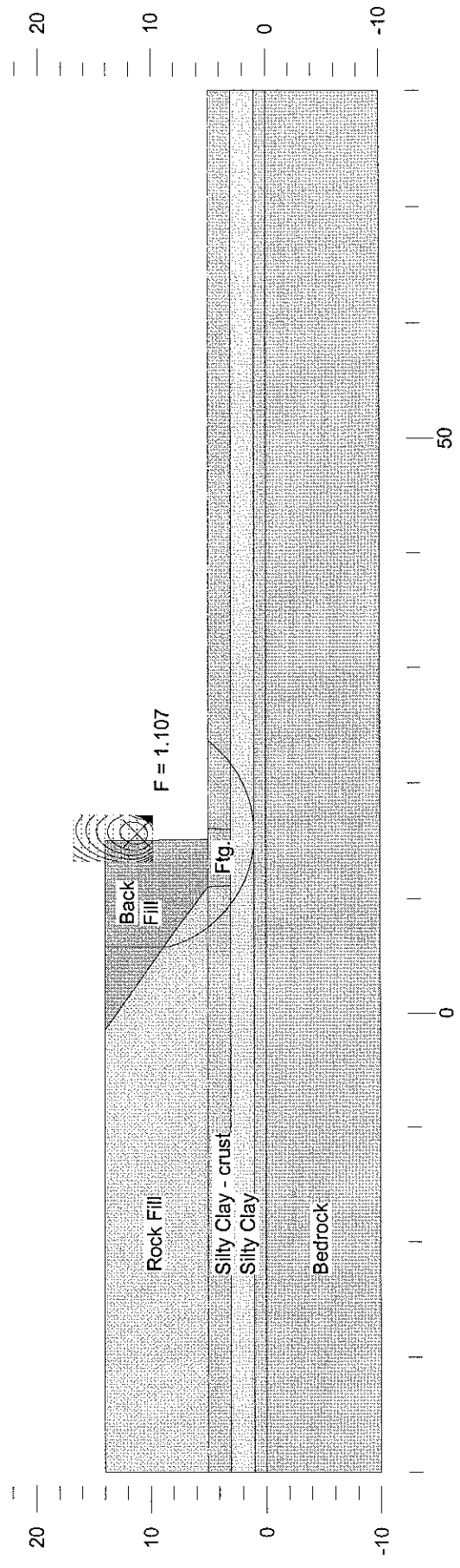


	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	19	0	42	0
Silty Clay-crust	18	0	28	1
Silty Clay-firm	18	0	27	1
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

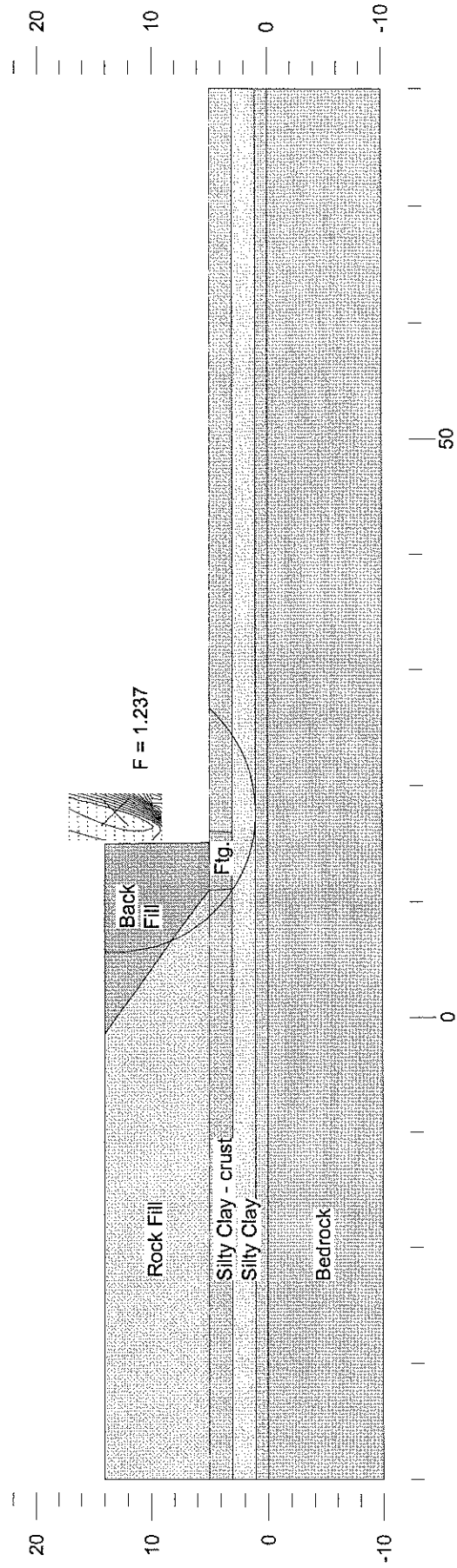
$F = 1.865$



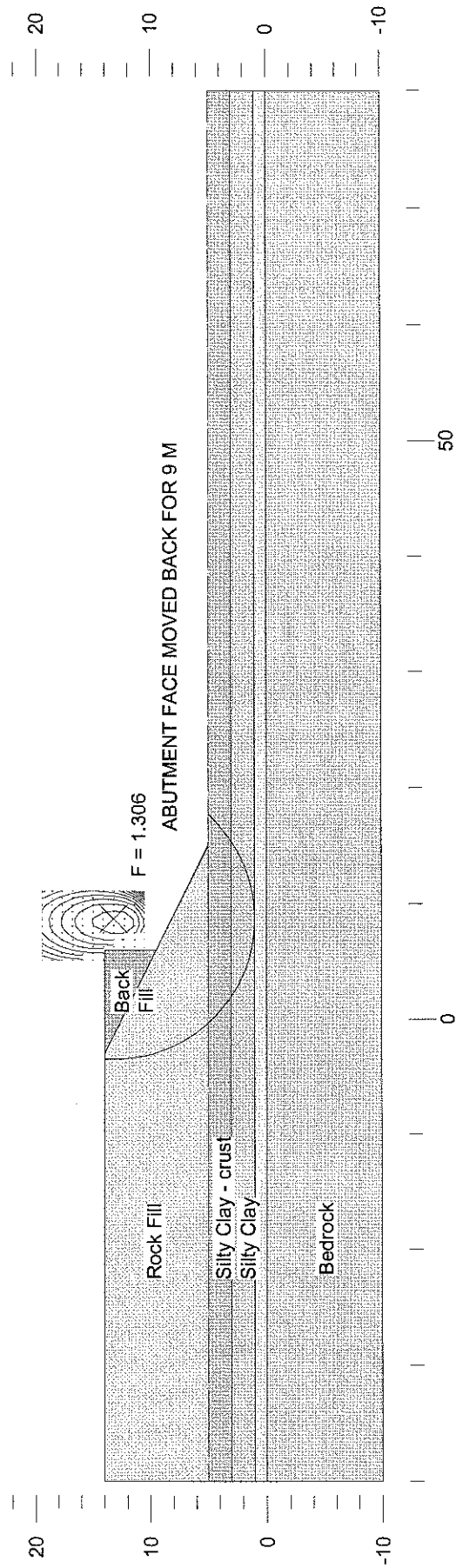
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Wall Backfill	21	0	32	0
Rock Fill	19	0	42	0
Footing (Fig.)	25	500	0	1
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	1
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			



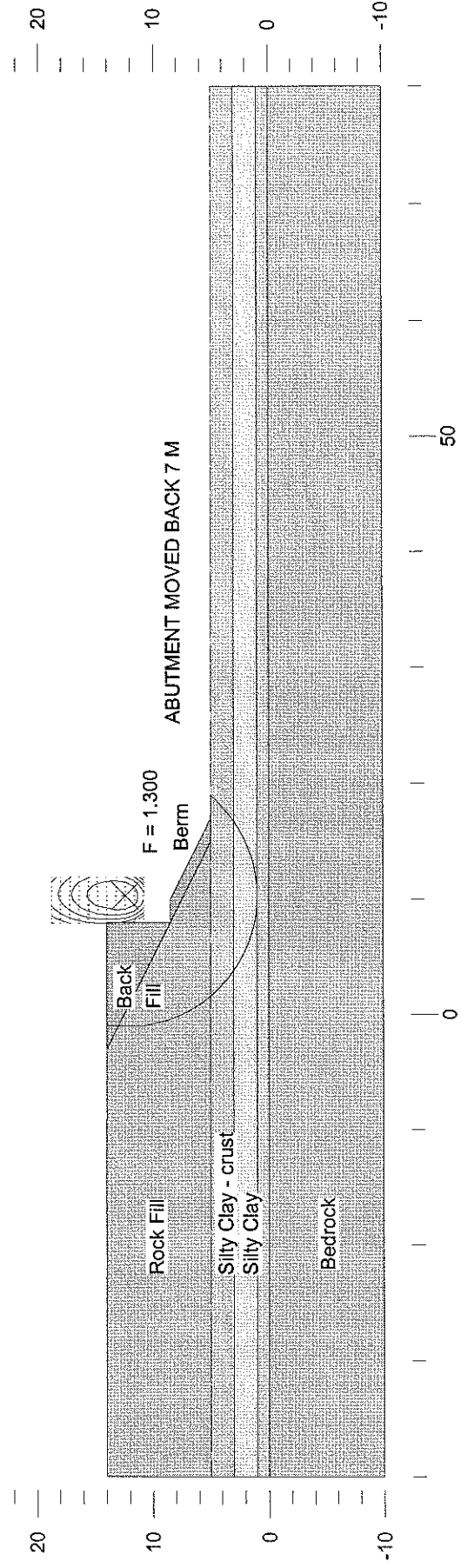
	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Wall Backfill	21	0	32	0	0
Rock Fill	19	0	42	0	0
Footing (Fig.)	25	500	0	0	1
Silty Clay-crust	18	0	28	0	1
Silty Clay-firm	18	0	27	0	1
Sand-Silt Till	20	0	30	0	1
Bedrock	(Infinitely Strong)				



	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Wall Backfill	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	1
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			



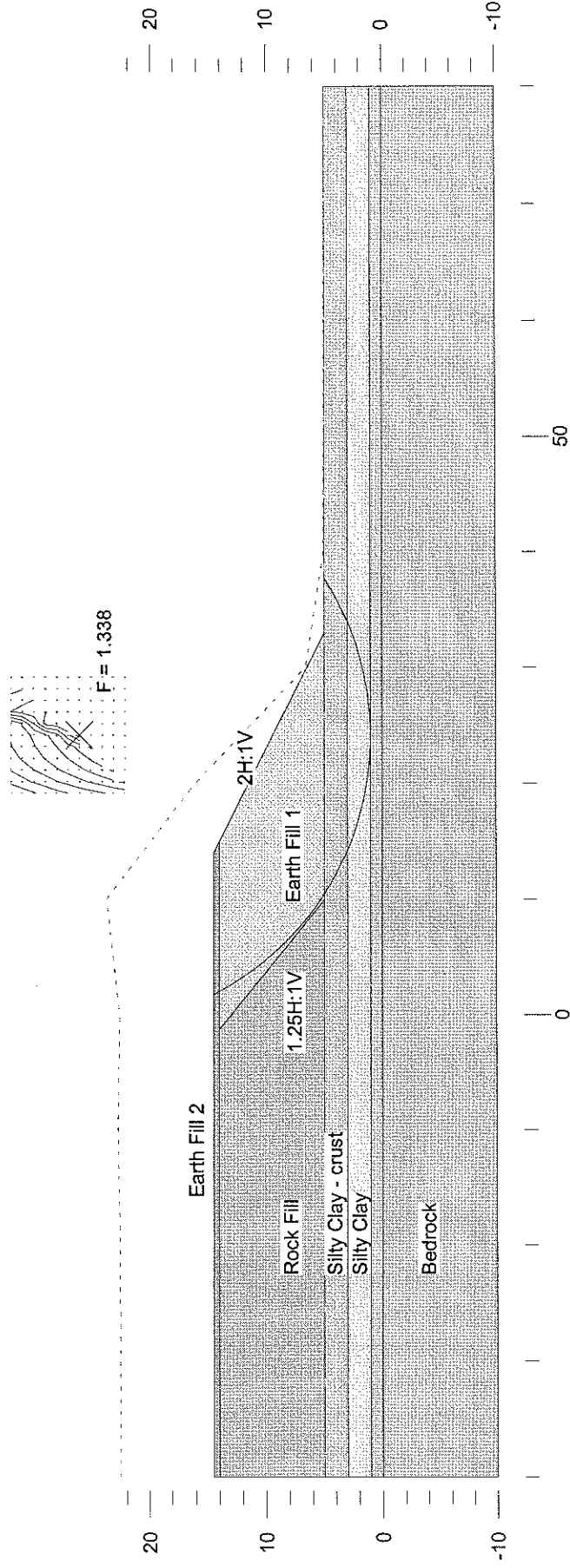
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Mid-height Berm	21	0	0	0
Wall Backfill	21	0	0	0
Rock Fill	19	0	0	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	1
Sand-Silt Till	20	0	0	1
Bedrock	(Infinitely Strong)			
Material 7	(Infinitely Strong)			



	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Earth Fill 1	21	0	32	0
Earth Fill 2	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	.25
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

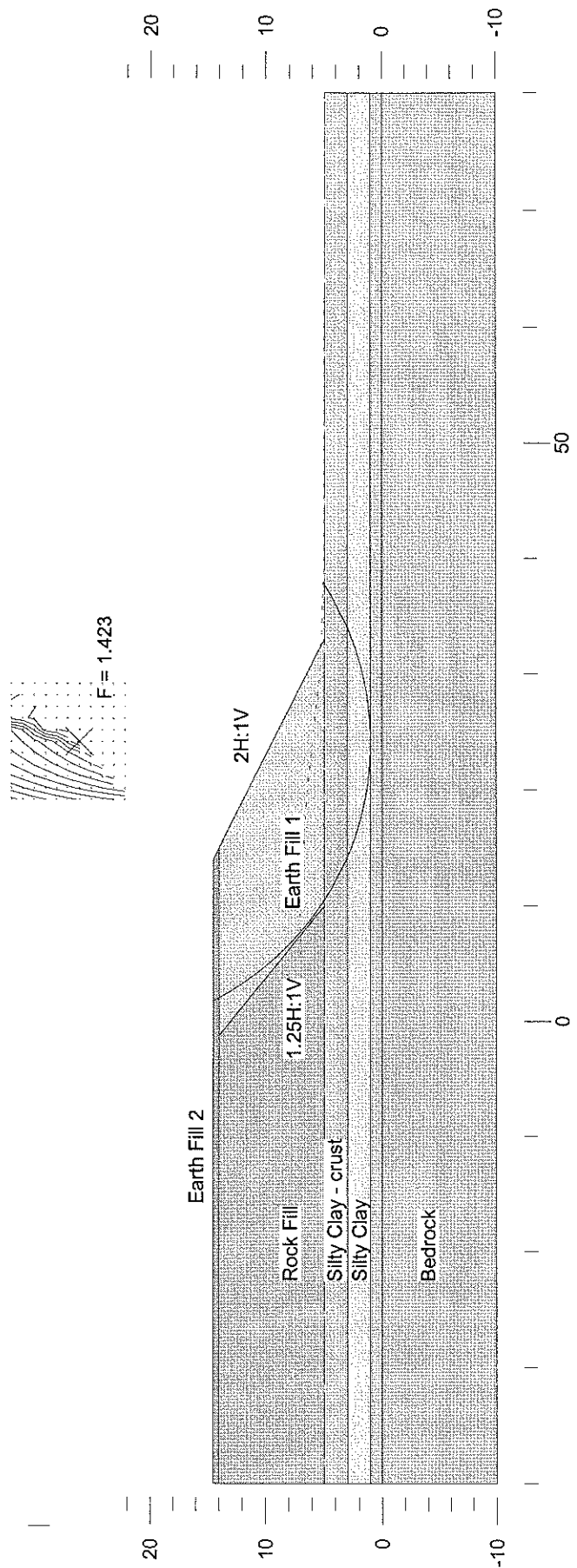
All fill placed in one continuous operation

Pore pressure parameter, $B = 0.9$ immediately after fill placement



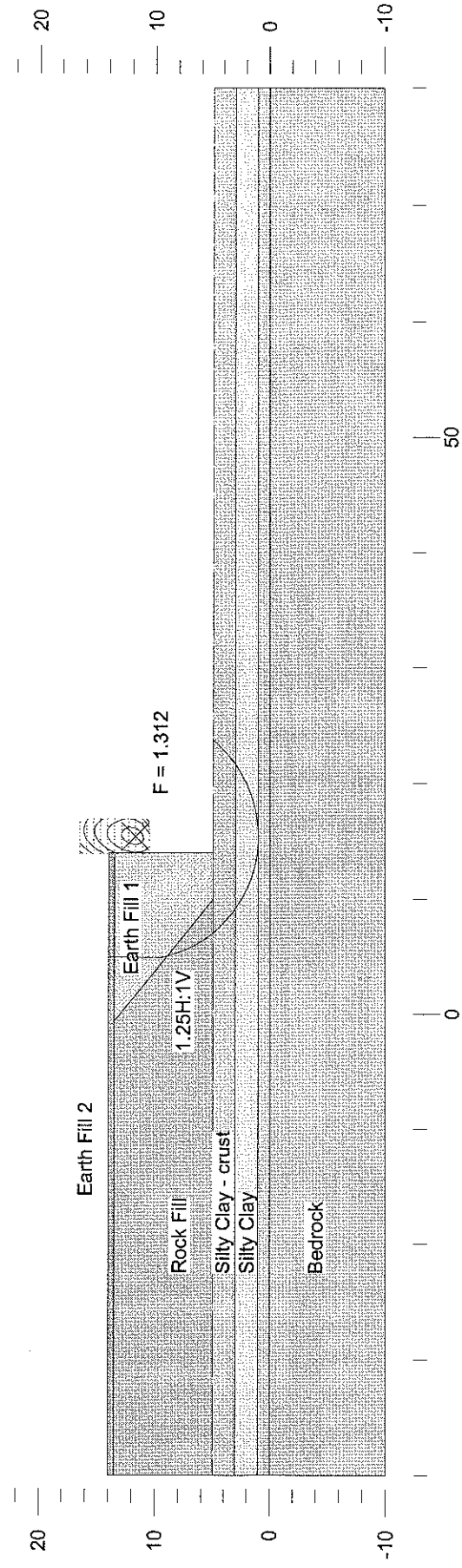
	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Earth Fill 1	21	0	32	0
Earth Fill 2	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	.25
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

All fill left in place for 6 months (waiting period)
 Pore pressure parameter, B = 0.1 six months after fill placement



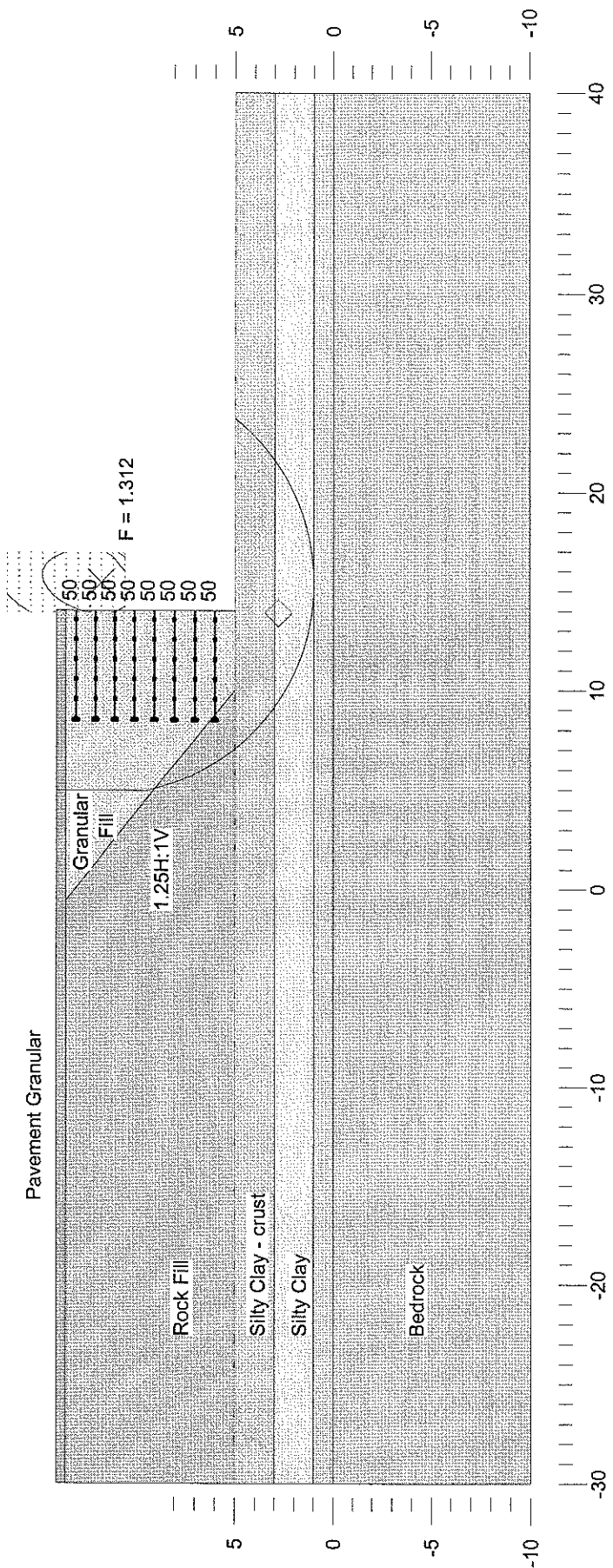
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill 1	21	0	32	0
Earth Fill 2	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	2
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

After abutment is constructed and all fill removed in front of abutment face
 All excess pore pressures dissipated



	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Pavement Gran.	21	0	32	0
Granular Fill	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	60	0	1
Silty Clay-firm	18	30	0	2
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

After stabilization of all foundation settlements
All excess pore pressures dissipated



Thurber Engineering Ltd. - Toronto
 19-3745-0
 Highway 17 Twinning - Campbell Drive Underpass
 March 22, 2004
 Stability of Forward Slope with RSS Wall - North Approach
 FIGURE G13 Drained Analysis - Rock Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Pavement Gran.	21	0	32	0
Granular Fill	21	0	32	0
Rock Fill	19	0	42	0
Silty Clay-crust	18	0	28	1
Silty Clay-firm	18	0	27	2
Sand-Silt Till	20	0	30	1
Bedrock	(Infinitely Strong)			

After stabilization of all foundation settlement
 All excess pore pressures dissipated

