

GEOCRES No:
31F-138

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
C.P.R. OVERHEAD (RENFREW)
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-194/1
GEOCRES Number: 31F-138**

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

August 27, 2004
File: 19-3745-0

C:\Thurber Projects 2003\19-3745-0\CPR Renfrew\CPR Renfrew Prelim Fdn Inv & Des Rep FINAL Aug 27.doc

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	Peat	3
5.2	Silty Clay to Clayey Silt	3
5.3	Silty Sand Till	4
5.4	Bedrock	4
5.5	Water Levels	5
6	GENERAL	7
7	STRUCTURE FOUNDATIONS	7
7.1	Foundation Alternatives	8
7.2	Spread Footings on Engineered Fill	9
7.3	Driven Piles	9
7.3.1	Axial Resistance	10
7.3.2	Downdrag on Piles	10
7.3.3	Lateral Resistance	11
7.3.4	Pile Installation	13
7.4	Augered Caissons	13
7.5	Frost Cover	14
8	EXCAVATION AND BACKFILL	14
8.1	General	14
8.2	Foundations	14
8.2.1	Earth Excavation	14
8.2.2	Rock Excavation	14
9	GROUNDWATER CONTROL	14
10	APPROACH EMBANKMENTS	15
10.1	Stability	15
10.2	Settlement	16
10.3	Embankment Construction	16

11	RETAINED SOIL SYSTEMS.....	17
11.1	Foundation.....	17
11.2	Global Stability	18
11.3	Internal Stability.....	18
11.4	Settlement.....	18
12	BACKFILL TO ABUTMENTS	18
13	EARTH PRESSURE	19
14	SEISMIC CONSIDERATIONS	20
14.1	Seismic Design Parameters	20
14.2	Liquefaction Potential	21
14.3	Retaining Wall Dynamic Earth Pressures	21
15	CONSTRUCTION CONCERNS	22

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Foundation Comparison
Appendix D	Figures
Appendix E	Borehole Locations and Soil Strata Drawing

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
C.P.R. OVERHEAD (RENFREW)
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-194/1
GEOCRES Number: 31F-138

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the location where a proposed overhead bridge will carry the new Highway 17 eastbound lanes over the existing C.P.R. railway tracks. The Ministry of Transportation (MTO) carried out a foundation investigation in 1971 for the existing overhead structure at this site. The factual data from that investigation has been used as a reference.

The purpose of this preliminary investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan and soil strata drawing with a stratigraphic profile, records of boreholes, laboratory test results and a written description of the subsurface conditions.

Thurber carried out the investigation as a sub-consultant to National Capital Engineering, under the MTO Agreement Number 4005-A-000157.

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

- MTO report titled "Foundation Investigation Report for the Overhead Structure at the Crossing of Proposed Hwy.17 'New' (W.B.L.) And Canadian Pacific Railway, Twp. Of Horton - Co. of Renfrew, District No. 9 (Ottawa), W.O. 71-11085, W.P. 5-67-01, GEOCRES 31F-19, dated December 1971 (Reference 1).

2 SITE DESCRIPTION

The site is located about 1.2 km east of the existing at grade intersection of Highway 17 and O'Brien Road and 1.2 km west of the existing at grade intersection of Highway 17 and County Road 6. The approximate mainline station of the proposed Highway 17 (E.B.L.) at this location is Sta. 22+438. This site is in the Township of Horton about 5 km east of the Town of Renfrew. The Borehole Locations and Soil Strata drawing in Appendix E contain further details on the general site location.

At this site, the C.P.R. track is on an embankment about 1.2 m above the surrounding ground. Approximately 40 m north of the site, the existing Highway 17 crosses over the CPR track via a three span bridge structure with approximately 9 m to 10 m high approach embankments. The site is relatively flat and low lying, and surficial drainage is generally poor. The area to the south of the track is swampy with standing water in some areas. The depth of standing water ranged from 0.6m to 0.9 m.

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. In this region, native soil deposits typically consist of glacio-lacustrine clayey silts to silty clays that were deposited when the Champlain Sea inundated the Ottawa – St. Lawrence lowland. This clay deposit varies in thickness over the region and is underlain by glacial till. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechère” graben. The overburden deposits are underlain by bedrock of the Ordovician Period and of the Precambrian age.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on September 30 and October 01, 2003 and consisted of drilling and sampling a total of two boreholes to depths ranging from 16.4 m to 17.2 m. The boreholes were numbered CPR-1 and CPR-2 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing.

Boreholes CPR-1 and CPR-2 were drilled at the proposed locations of the west abutment and the west pier (#1), respectively. Two attempts were made to drill the boreholes at the proposed locations of the east pier (#2) and the east abutment (once during the winter of 2003 and once in March 2004), but the Nodwell mounted rig was unable to access the borehole locations due to swampy conditions and significant depth of standing water.

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited who also provided us with the coordinates and geodetic elevations of the boreholes after drilling was completed. Utility clearances at the borehole locations were obtained 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 both boreholes. Auger drilling techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. Both boreholes were extended 3.3 m into bedrock by NQ size rotary coring techniques.

A standpipe piezometer was installed in each borehole to monitor the ground water level. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.8 m to 2.1 m long slotted screen were installed at or near the bottom of the open boreholes. The sand screens surrounding the pipes were about 2.5 m long. Bentonite holeplug seals were placed just above the sand screen

and just below ground surface in each installation. The remaining space in the boreholes was backfilled with drill cuttings.

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 in the field.

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 samples were subjected to gradation analysis. Atterberg Limit Tests were performed on samples retrieved from the cohesive deposit. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load tests were carried out on selected rock cores retrieved from the two boreholes and these results are presented in Table 1 attached immediately following the text of this report.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" in Appendix E of the report. A description of the stratigraphy is given in the following paragraphs.

In general, the site is underlain by peat followed by deposits of very stiff to firm silty clay to clayey silt and compact to very dense silty sand till overlying bedrock.

5.1 Peat

A 0.9 m thick deposit of peat was encountered at the two boreholes. This peat deposit extends to an elevation of 129 m. Standard Penetration tests in this deposit gave 'N' values of 4 to 6 blows for 0.3 m penetration indicating a soft to firm consistency. The moisture content of samples of the peat range between 71% and 83%.

5.2 Silty Clay to Clayey Silt

At the two boreholes, the peat is underlain by a deposit of silty clay to clayey silt containing occasional sand lenses and sand seams. This deposit was encountered at a depth of 0.9 m, Elevation 129 m, below ground surface and it extends to depths ranging from 4.1 m to 4.3 m, or from Elevations 125.6 m to 125.8 m.

Grain size analyses conducted on samples retrieved from this layer are presented in Figure B1 in Appendix B. These results show that the clay content is approximately 21% to 22%. Atterberg Limit Tests conducted on selected samples from this deposit are presented in Figure B2 of Appendix B. These results indicate that the soil is typically of low plasticity (CL).

Standard Penetration Tests conducted in this layer gave 'N' values ranging from 4 to 23 blows per 0.3 m penetration thereby indicating a very stiff to firm consistency. Lower 'N' values of 4 and 8 blows per 0.3 m penetration were recorded in the lower 1 m to 1.5 m of this deposit immediately above the underlying silty sand till. The measured natural moisture contents of samples retrieved from this layer range from 22% to 28%.

5.3 Silty Sand Till

Below the silty clay layer, a deposit of silty sand till was encountered at depths ranging from 4.1 m (Elevation 125.8 m) to 4.3 m (Elevation 125.6 m) below ground surface. This till deposit extends to depths of 13.1 m to 13.9 m or from elevations varying between Elevations 116 m and 116.8 m.

Samples from this layer were subjected to grain size distribution tests and the results are illustrated in Figure B3.

This deposit is cohesionless with trace to some clay sized particles. Glacial tills are known to contain cobbles and boulders. Standard Penetration Tests conducted within this stratum yielded 'N' values ranging from 20 blows for 0.3 m penetration to 50 blows for less than 0.3 m penetration, indicating a compact to very dense state. The measured natural moisture contents of samples retrieved from this layer range from 7% to 19%.

5.4 Bedrock

The soils described above are underlain by dolomitic marble bedrock. Bedrock was proven by coring in the two boreholes and the table below summarizes the depths and elevations of the bedrock surface.

Borehole Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
CPR-1	13.9	116.0
CPR-2	13.1	116.8

The bedrock is thinly bedded and is in a slightly to moderately weathered state. Its colour is grey, brown and white with sub-horizontal black banding.

Core recovery, measured in TCR, generally ranged between 92% and 100% and the RQD values ranged from 0% to 100%. Most RQD values ranged between 50% and 100% indicating a fair to excellent rock quality. Near the bottom of Borehole CPR-1, an infilled cavity was encountered at an approximate depth of 3 m below the rock surface. The infilling materials consist of sand with clayey silt to silty clay inclusions.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was low and generally ranged from 0 to 3 with occasional higher values of 4 and 5. The joint orientation was generally vertical to sub-vertical. The joint conditions were rough, but were mostly tight with no infilling or secondary weathering material except for the infilled cavity described above.

Point load Tests were conducted on rock cores at selected intervals. The inferred Unconfined Compressive Strength (UCS) of the rock cores ranges from 75 MPa to more than 100 MPa indicating that the intact rock is strong to very strong. A summary of the Point Load Test results is presented in Table 1 attached immediately following the text.

5.5 Water Levels

Standpipe piezometers were installed in the two boreholes and their water levels were measured on separate site visits after installation. These readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation m)
CPR-1	October 22, 2003	0.4	129.5
	February 04, 2004	0.6	129.3
	March 11, 2004	0.2	129.7
CPR-2	October 22, 2003	0.5	129.4
	February 04, 2004	0.0*	129.9
	March 11, 2004	0.0*	129.9

* Frozen at ground surface.

Based on these observations, local groundwater levels are anticipated to range between Elevations 129.3 m and 129.9 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 Geotechnical Engineer



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

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
C.P.R. OVERHEAD (RENFREW)
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-194/1
GEOCRES Number: 31F-138

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents preliminary 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 calls for construction of a new structure to carry the new eastbound lanes (EBL) of the twinned Highway 17 over the C.P.R. track. The existing Highway 17 will become the westbound lanes (WBL) of the highway. The new structure will be located at about 40 m south of the existing structure. The new structure will be similar to the existing structure and consists of three-span precast prestressed girders. Both approach spans will be approximately 24.5 m long and the centre span will be approximately 26.5m long. The abutment bearings will be skewed at 25°.

It is understood that the proposed Highway 17 EBL will remain at the same grade as that of the existing highway. This corresponds to approach fill heights of about 11 m at both abutments.

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

7 STRUCTURE FOUNDATIONS

The proposed bridge for this site will consist of a three-span structure with a total of four foundation elements: two abutments and two piers.

The stratigraphy encountered at the locations of the proposed west abutment and west pier consists of peat overlying deposits of very stiff to firm silty clay to clayey silt and compact to very dense silty sand till, containing possible cobbles and boulders, overlying dolomitic marble bedrock. Boreholes were not drilled at the east abutment and east pier locations due to access problems described previously. Additional boreholes should be drilled at the east abutment and east pier locations during the detailed design stage.

The elevations at which bedrock was encountered (including information from Reference 1) are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
West Abutment	CPR-1	129.9	116.0*
	1**	129.7	117.3**
West Pier	CPR-2	129.9	116.8*
	3**	129.8	116.6**
East Pier	CPR-3	Not drilled as location inaccessible	
	5**	130.0	114.8**
East Abutment	CPR-4	Not drilled as location inaccessible	
	6**	129.7	113.4**

* Bedrock proven by coring (present investigation).

** Bedrock proven by coring (MTO boreholes located to the north of the foundation element from Reference 1).

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
- Augered caissons (drilled shafts)

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

An integral or semi-integral abutment design is possible at this site.

Spread footings founded directly on the native, compressible cohesive foundation soils are not recommended at this site since the settlements under the footing loads will be unacceptable. Spread footings on compacted Granular A pads resting on the silty clay to clayey silt overlying compact to very dense silty sand till are considered feasible at the abutments, provided that the anticipated settlements are tolerable. Additional drilling will be required to obtain undisturbed samples for laboratory oedometer (consolidation) testing in order to better quantify the magnitudes of settlements. Spread footings are not feasible for the piers due to space limitations and the significant depths of excavation that may be required to reach suitable founding stratum.

In view of the above, it is considered that piled foundation consisting of steel piles driven to bedrock is the most feasible means of providing foundation support for the abutments and the piers. Given the proposed approach embankment height in the order of 11 m, a perched abutment design may be considered in conjunction with the piles. Alternatively, augered caissons socketted into bedrock may be used although there are installation risks associated with this option due to the cohesionless soils below the groundwater table, and the presence of boulders and cobbles.

7.2 Spread Footings on Engineered Fill

For perched abutment design, spread footings founded on an engineered fill pad may be considered at the abutments.

If an engineered fill pad is used, all the peat, organics, fill, soft soils and surficial deleterious materials should be sub-excavated and removed, and the new fill placed directly on the undisturbed surface of very stiff silty clay to clayey silt encountered at or below approximate Elevation 129 m. The engineered fill should consist of OPSS Granular "A" compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at $\pm 2\%$ of optimum moisture content (OPSS 501, Section 501.08.02 Method A) and conforming to the geometry illustrated in Figure D1 in Appendix D. It is recommended that the thickness of the fill pad be equal to or greater than the footing width, but should not be less than 2 m.

Provided a minimum footing width of 2 m is maintained, a footing founded on a compacted Granular A pad, resting on the native stiff clays, of thickness equal to or greater than the footing width may be designed for the following values:

- Factored geotechnical resistance of 450 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 300 kPa at Serviceability Limit States (SLS)

These values are for vertical, concentric loads only. 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 SLS value quoted above corresponds to a settlement of up to 25 mm that is expected to be complete by the end of construction.

Resistance to lateral forces / sliding resistance between the concrete footing and compacted Granular A subgrade should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.7.

7.3 Driven Piles

Steel piles driven to bedrock may be used to provide foundation support at the abutments and piers. Both an integral or conventional design may be used at the abutments, while a conventional design is applicable at the piers. Based on the borehole information, bedrock

at this site is present at between 13m and 14 m depths below existing ground surface. The following pile tip elevations are recommended for preliminary design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
West Abutment	CPR-1*	116 ±
West Pier	CPR-2*	116.8 ±
East Pier	5**	114.8 ±
East Abutment	6**	113.4 ±

* Present Investigation

** Reference 1

Additional drilling should be carried out during the detailed design stage to determine bedrock elevations at the east abutment and east pier locations and to confirm pile tip elevations.

Given the likely presence of boulders and cobbles above bedrock, the detailed investigation should also assess the frequency and distribution of boulders and cobbles as they may result in pile tip elevations higher than those presented above.

7.3.1 Axial Resistance

In the absence of more detailed information at this time, it is considered prudent to carry out preliminary design of HP 310 x 110 piles driven to refusal within the very dense glacial till using the following recommended preliminary pile capacities:

- Factored geotechnical resistance at ULS of 1,600 kN per pile.
- Geotechnical resistance at SLS of 1,200 kN per pile.

The above SLS capacity corresponds to 25 mm of pile tip settlement.

The above pile capacities should be reassessed during detailed design.

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

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

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

7.3.2 Downdrag on Piles

The area in the vicinity of the east approach is swampy and is likely covered by variable thickness of peat, organics and possibly soft soils. As discussed further in Section 10 Approach Embankments, it is recommended that all peat, organics and surficial soft soils

be sub-excavated prior to placing engineered fill. Pile driving should only commence after the completion of engineered fill placement.

Elastic settlements due to recompression of the over-consolidated portion of the foundation silty clay to clayey silt and the underlying cohesionless silty sand till is expected to occur as the fill is placed. Downdrag forces could be induced on the piles as a result of consolidation of the lower, more compressible zone of the cohesive deposit. Due to the small thickness of this lower, compressible zone (less than 2 m), it is estimated that the magnitude of downdrag force acting on the piles will be relatively insignificant and will not affect the geotechnical resistance recommended above. As long as the piles are installed after the completion of engineered fill placement, downdrag is not considered to be a design issue at this site.

7.3.3 Lateral Resistance

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

For lateral soil-pile interaction at this site, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

New Approach Fill (compact)

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

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

Note: Due to the proximity of the embankment slope to the uppermost portion of the pile, it is recommended that a reduction factor be applied to the k_s values calculated above. The reduction factor can be assumed to decrease from 1.0 to 0, corresponding to a horizontal distance of 8B to B, respectively, between the outermost pile and the slope surface.

Silty Clay to Clay (very stiff to firm)

$$k_s = 125 \cdot S_u / B \quad (\text{kPa/m})$$

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

where z = depth below underside of pile cap (conventional design), m
= depth below underside of CSP (integral design), m

B = pile width, m

n_h = 6,000 kPa/m (engineered fill compacted to at least 95%
Standard Proctor density)
= 4,500 kPa/m (compact silty sand till between Elevations 126 m
and 122 m)
= 11,000 kPa/m (very dense silty sand till below Elevation 122 m)

$$\begin{aligned}\gamma &= 20 \text{ kN/m}^3 \\ K_p &= 3.0 \text{ (passive earth pressure coefficient)} \\ S_u &= \text{undrained shear strength of silty clay to clayey silt} \\ &= 125 \text{ kPa (Elevations 129 m to 127 m)} \\ &= 40 \text{ kPa (below Elevation 127 m)}\end{aligned}$$

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

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

Since the piles are end bearing on rock, the vertical resistance will not be significantly affected by the pile spacing. Pile 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 pile 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 :

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

where B is the width of the pile, and spacing is measured centre to centre

Where a pile 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 :

Pile Spacing Parallel To Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 B	1.00
6 B	0.70
4 B	0.40
3 B	0.25

Intermediate values may be obtained by interpolation.

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. Further details for geotechnical design of integral abutments should be developed during detailed design.

7.3.4 Pile Installation

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

Prior to pile installation, an engineered fill core consisting of approved granular materials compacted to the specifications of OPSS 501 will be required. This granular core may have side slopes not steeper than 1.5H : 1V, should be free of boulders, cobbles and particles of nominal diameter not exceeding 75 mm, and should extend a minimum distance of 1.5 m beyond the perimeter of the pile cap.

Moderately sloping bedrock surface should be anticipated at this site. Boulders and cobbles will also be encountered within the silty sand till. In order to enhance adequate seating of the pile tips into bedrock and penetration through boulders, 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").

As per Standard Provision No. 903S01, retapping and/or re-driving of piles should be carried out as required.

7.4 Augered Caissons

The abutments and piers may be supported by augered caissons (drilled shafts) founded on bedrock. In order to found the caissons below the surficial, typically 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 in Section 7.

It is understood that steel piles driven to bedrock is the currently preferred foundation option at this site. Should the augered caisson option be pursued, further recommendations should be developed during detailed design.

7.5 Frost Cover

For footings founded on engineered fill or for conventional pile caps, frost protection should be provided. This may take the form of 1.9 m of earth cover, or equivalent thermal insulation, over the footing base (founding elevation) or underside of the pile cap.

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 maintained more than 2.5 m below the underside of the foundation.

8 EXCAVATION AND BACKFILL

8.1 General

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the silty clay to clayey silt and the underlying silty sand till below groundwater level 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.

8.2.1 Earth Excavation

Excavation for engineered pad and pile cap construction will be carried out through the peat into the very stiff to firm silty clay to clayey silt. Where space permits, it is anticipated that such excavations may be carried out as unsupported open cuts with inclined side slopes (according to OHSA). Where open cutting is not feasible due to space restrictions and other reasons, temporary shoring will be required. Recommendations for temporary shoring design should be developed during detailed design as required.

8.2.2 Rock Excavation

Rock excavation will not be required at this site. Should the caisson option be adopted for foundation support, recommendations for socketting the caissons should be developed during detailed design as required.

9 GROUNDWATER CONTROL

Given the situation where a majority of the site area is covered by peat and standing water, seepage will occur into the excavations for engineered fill pad or pile cap construction. Surface runoff will also contribute to water accumulation in the excavations. The Contractor must drain the swampy area immediately around any foundation location and control the groundwater seepage into the

excavations prior to placing concrete or compacting granular fill. Possible means to maintain reasonably dry excavations during construction include a combination of perimeter dykes consisting of cohesive soils, pumping from filtered sumps and possibly interlocking sheetpiles. Consideration should be given to winter construction when the swampy area is frozen in order to facilitate peat excavation, groundwater control and temporary excavation operations.

10 APPROACH EMBANKMENTS

For the purpose of preliminary embankment stability analyses, the 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 and long term conditions.

Preliminary estimates of foundation settlements have been made 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.

10.1 Stability

Approach fills will be placed on swampy areas at this site. Prior to fill placement, all peat, organics and soft soils must be removed from the footprint of the approach embankments. The 11 m high approach embankments for this structure may then be constructed over about 3.5 m of typically over-consolidated silty clay to clayey silt, underlain by up to 10 m of compact to very dense silty sand till overlying bedrock. Earth fill or rock fill may be used to construct the embankments. For the rock fill option, an engineered fill core is required to facilitate pile driving (as discussed earlier). The slope of the core may be formed not steeper than 1H : 1V for Granular A material and 1.5H : 1V for other types of cohesionless fill (the granular core should extend at least 1.5 m beyond the footing or pile cap perimeter). Provided that the core is constructed as recommended in this report, blast rockfill embankments formed with a slope inclination not steeper than 1.25H : 1V will be stable. Earth embankments constructed using granular or select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V. Stabilizing berms will not be required to maintain global stability.

Preliminary stability analyses results indicated that a minimum Factor of Safety (F.S.) of 1.3 can be achieved for both short and long term conditions. For earth or rock fill embankments, a mid-height berm will be required to address surficial stability as discussed in Section 10.3 Embankment Construction.

Additional boreholes will be required during detailed design to delineate the lateral and vertical extents of the swampy area within the approach embankment footprints, including the thickness of peat, organics and possibly soft soils. Based on all the available information, the approach embankment design may then be finalized.

10.2 Settlement

Some settlement 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.

It is recommended that all peat, organics and surficial soft soils within the embankment footprints be sub-excavated prior to placing engineered fill in order to minimize post construction settlement. Pile driving should only commence after the completion of engineered fill placement.

The new fill will induce some settlement within the foundation soils. Provided that the sub-excavation is carried out, as outlined above, to expose the very stiff silty clay to clayey silt subgrade, it is anticipated that elastic settlements in the order of 50 mm will occur as a result of compression of the over-consolidated cohesive deposit and of the cohesionless silty sand till. Post construction settlement will likely be less than 25 mm. This should be re-assessed during detailed design based on information obtained from the additional boreholes to be located within the swampy areas.

10.3 Embankment 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. 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. Granular materials or 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. It is noted that the requirements for a 2 m wide berm for a 10 m high rock fill, and for a 8 m high earth fill, are in place to address surficial stability and to provide access for post construction maintenance.

Preliminary assessment indicates that the approach embankment is stable against seismic activities at this site.

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

11 RETAINED SOIL SYSTEMS

A conventional concrete abutment will be required for the contemplated design but Retained Soil System (RSS) walls could be used for the wing walls. However, there is medium risk associated with using RSS walls at this site due to the anticipated settlements of the foundation soils.

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

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 well compacted embankment fill or native, very stiff silty clay to clayey silt. Where applicable, the RSS subgrade 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 mat for the levelling pad 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 320 kPa, and geotechnical resistance of 250 kPa at SLS on an engineered Granular A pad.
- Ultimate coefficient of friction between cast in-situ concrete levelling pad on Granular A is 0.7.

In addition to the requirements for the levelling pad, the main body of the RSS may be founded on the native, stiff silty clay to clayey silt, compacted earth fill or rock fill. All peat, topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

The following parameters may be used for the foundation design of an RSS wall:

- Factored geotechnical resistance of 225 kPa at ULS and geotechnical resistance of 150 kPa at SLS, founded on compact embankment fill above Elevation 130m.
- Factored geotechnical resistance of 300 kPa at ULS and geotechnical resistance of 200 kPa at SLS, founded on native stiff silty clay to clayey silt at or below approximate Elevation 129 m.

- Ultimate coefficient of friction between RSS mass and compact fill is 0.55.
- Ultimate coefficient of friction between RSS mass and native stiff 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, are likely to be as wing walls at the abutments. It is envisaged that the RSS will be founded on compact embankment fill, or native stiff clay.

Results of preliminary stability analyses yielded F.S. values not less than 1.3 indicating that global stability can be maintained for the assumed RSS configuration.

The actual design configuration must be checked for global stability during detailed design.

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 newly compacted embankment fill or native stiff clay will depend on the thickness of the pad, the material used, the conditions of the subgrade and the quality of construction. Preliminary calculations indicated that post construction settlement of RSS walls founded on the compact embankment fill and native stiff clay would be less than 25 mm.

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.

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:

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

where P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table 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	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. 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 site area is underlain by 12 m to 13 m of overburden consisting of very stiff to firm clays overlying silty sand till, underlain by bedrock. 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 0.091 m/sec. These values should be used for the seismic design of the bridge at this site.

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 x 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 x Zonal Acceleration Ratio, 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 bedrock using piles, there is no potential for soil liquefaction under the foundations.

The approach embankments will be founded on the native, stiff silty clay to clayey silt overlying compact to dense silty sand till. These soils 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, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that include the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients, the angle of friction, δ , between the wall and backfill material is assumed to be 0.5 ϕ , the angle of internal friction of the backfill.

For the design of retaining walls, the seismic earth pressure coefficients shown in the following table may be used:

Wall Condition	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient (K) for Earthquake Loading					
		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, the following:

- additional boreholes at the east pier and east abutment to confirm depth to rock and to assess frequency and distribution of boulders and cobbles.
- additional boreholes in the swampy area within the approach embankment footprints.
- laboratory oedometer (consolidation) tests on undisturbed clay samples to better quantify the magnitudes of time-dependent settlements.
- maintaining stability of the embankment fill at all times.
- confirming that elastic settlement has completed at the approaches before commencing deep foundation installation.
- potential for encountering boulders and cobbles during deep foundation installation.



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



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

Point Load Test Results
CPR Overhead (Renfrew)

TABLE 1
CPR - Renfrew
Point Load Test Results

Depth			Is50	UCS (MPa)				
feet	Inches	m						
CPR-1								
46	7	14.20	7.20	172.75	Total Rock Core Average Minimum Maximum MPa 129 13 208 Run # Average 1 172.75 2 130.62 3 123.03 4 74.79			
47	0	14.33	8.38	201.19				
48	0	14.63	3.77	90.59				
49	0	14.94	5.75	137.99				
49	11	15.21	3.86	92.70				
51	0	15.54	8.65	207.51				
52	0	15.85	7.46	179.07				
53	4	16.26	5.14	123.24				
54	4	16.56	0.53	12.64				
55	4	16.87	3.12	74.79				
56	0	17.07	0.00	0.00				
Note:					Point load test at 16.56 m was performed at hidden joint Point load test at 17.07 m was performed in sand infilling			

Depth			Is50	UCS (MPa)				
feet	Inches	m			Average	Minimum	Maximum	MPa
CPR-2								
43	2	13.16	6.89	165.38	}	Total Rock Core		
44	9	13.64	6.41	153.79		Run #	Average	
45	2	13.77	5.71	136.94		1	159.59	
47	3.5	14.41	5.40	129.56		2	142.99	
47	10	14.58	5.53	132.72		3	150.89	
48	9	14.86	7.20	172.75				
50	4	15.34	6.01	144.31				
51	4	15.65	5.84	140.10				
52	5	15.98	7.72	185.39				
53	0	16.15	5.57	133.78				

Appendix A

Record of Borehole Sheets

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR	 Shelby Tube	A - Casing
SAMPLE TYPE	 SPT	 Grab/Auger sample
	 No Recovery	 Core

- MC – Moisture Content (% by Weight) as determined by sample

 Water Level


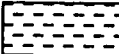



C_{vane}	Shear Strength Determination by Field Insitu Vane
C_{pen}	Shear Strength Determination by Pocket Penetrometer
C_{lab}	Shear Strength Determination using a Laboratory Vane Apparatus
C_u	Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test – refers to the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

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

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
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)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 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.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No CPR-1

1 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5037466.4, E 294377.6 (CPR Overhead, Renfrew) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 30.09.03 - 30.09.03 CHECKED BY SKP

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		
129.9	PEAT, some rootlets Firm Black		1	SS	6									
129.0	Silty CLAY to Clayey SILT, some sand seams Very Stiff Brown Moist to Wet (CL)		2	SS	19		129							
			3	SS	16		128							
	becoming firm, grey, wet		4	SS	6		127							
			5	SS	4		126							
125.6	Silty SAND, trace to some clay, trace gravel Compact to Dense Grey Wet (TILL) (ML-nonplastic)		6	SS	20		125							0 20 59 21
4.3	occasional inferred cobbles		7	SS	36		124							
			8	SS	50/ .076		123							
	becoming very dense		9	SS	53		122							
							121							
							120							2 52 35 12

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CPR-1

2 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5037466.4, E 294377.6 (CPR Overhead, Renfrew) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 30.09.03 - 30.09.03 CHECKED BY SKP

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 C	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)
								20	40	60			
	Silty SAND, trace to some clay, trace gravel Very Dense Grey Wet (TILL) (ML-nonplastic)		10	SS	50/ .127								
			11	SS	50/ .076								
116.0	SPOON SAMPLE REFUSAL AT 13.9m		12	SS	50/.050								
13.9	DOLOMITIC MARBLE (BEDROCK) Slightly to moderately weathered, thinly bedded, grey, brown and white with subhorizontal black banding, strong to very strong Vertical joints from 14.07m to 14.2m.		1	RUN	5								
			2	RUN	2								
			3	RUN	1								
			4	RUN	0								
112.7	Cavity infilled with sand, clayey silt to silty clay from 16.92m to 17.17m.												
17.2	END OF BOREHOLE AT 17.17m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 2.13m slotted screen. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 22/10/2003 0.4 129.5 04/02/2003 0.6 129.3 11/03/2004 0.2 129.7				FI								

No sample recovery, spoon sample refusal, probably on cobbles/ boulder

RUN 1#
TCR=100%,
SCR=100%,
RQD=0%,
UCS=173MPa
 RUN 2#
TCR=100%,
SCR=100%,
RQD=100%,
UCS=131MPa
 RUN 3#
TCR=100%,
SCR=100%,
RQD=100%,
UCS=123MPa

RUN 4#
TCR=100%,
SCR=100%,
RQD=50%,
UCS=75MPa

ONTMT4 7450CPR.GPJ 28/05/04

RECORD OF BOREHOLE No CPR-2

1 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5037451.2, E 294404.1 (CPR Overhead, Renfrew) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 30.09.03 - 01.10.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
129.9 0.0	PEAT Soft Black		1	SS	4									
129.0	Silty CLAY to Clayey SILT, occasional sand lenses Very Stiff Brown Moist (CL) becoming stiff		2	SS	23		129							
			3	SS	22		128							
			4	SS	14		127							0 9 69 22
			5	SS	8		126							
125.8														
4.1	Silty SAND, trace gravel, trace clay Compact to Dense Grey Wet (TILL) (ML-nonplastic) occasional inferred cobbles		6	SS	21		125							10 57 26 7
			7	SS	24		124							
			8	SS	44		123							
							122							2 51 39 8
			9	SS	50/ .076		121							
	becoming very dense, occasional inferred cobbles						120							

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

G.W.P.	647-92-00	LOCATION	N 5037451.2, E 294404.1 (CPR Overhead, Renfrew)	ORIGINATED BY	SL
HWY	HWY 17	BOREHOLE TYPE	Hollow Stem Augers, NQ Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	30.09.03 - 01.10.03	CHECKED BY	SKP

[illegible]

CONTMT4 7450CPR.GPJ 27/05/04

+ 3, × 3: Numbers refer to Sensitivity

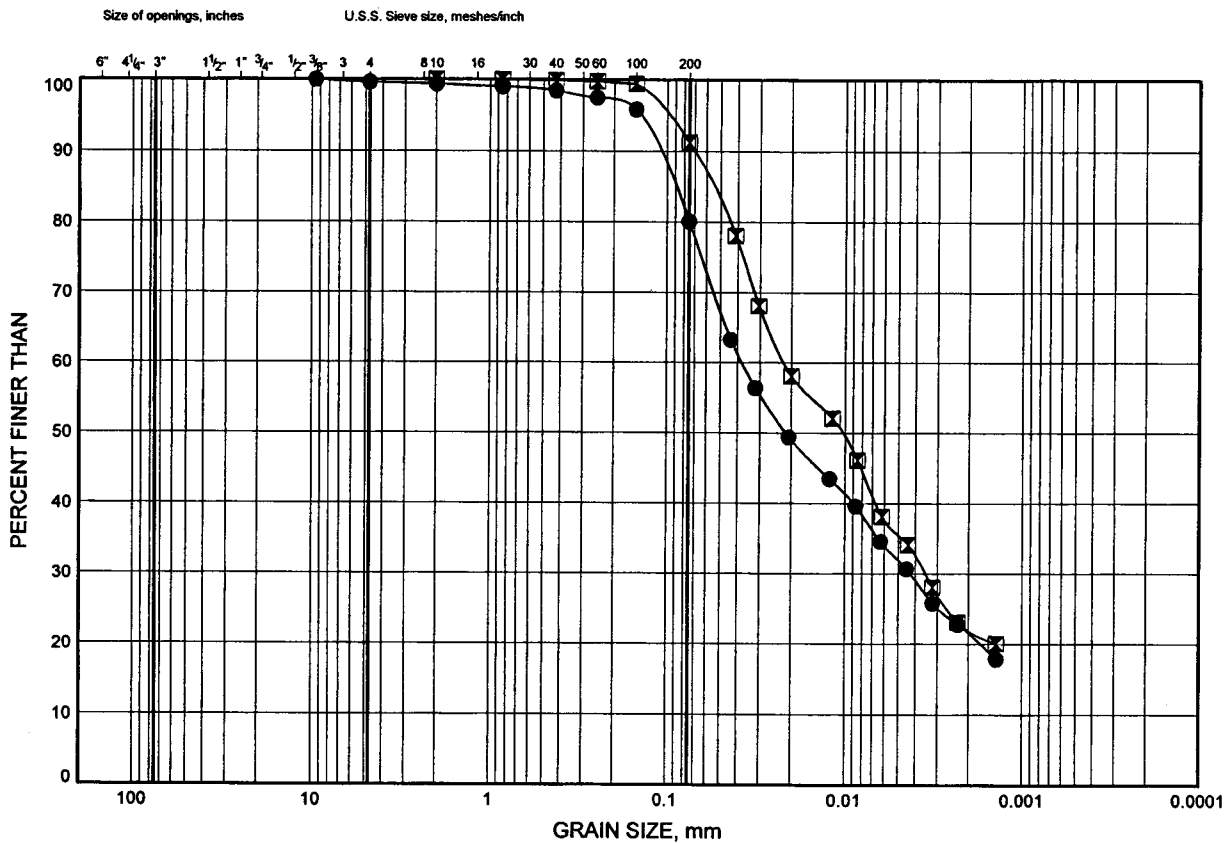
Appendix B

Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

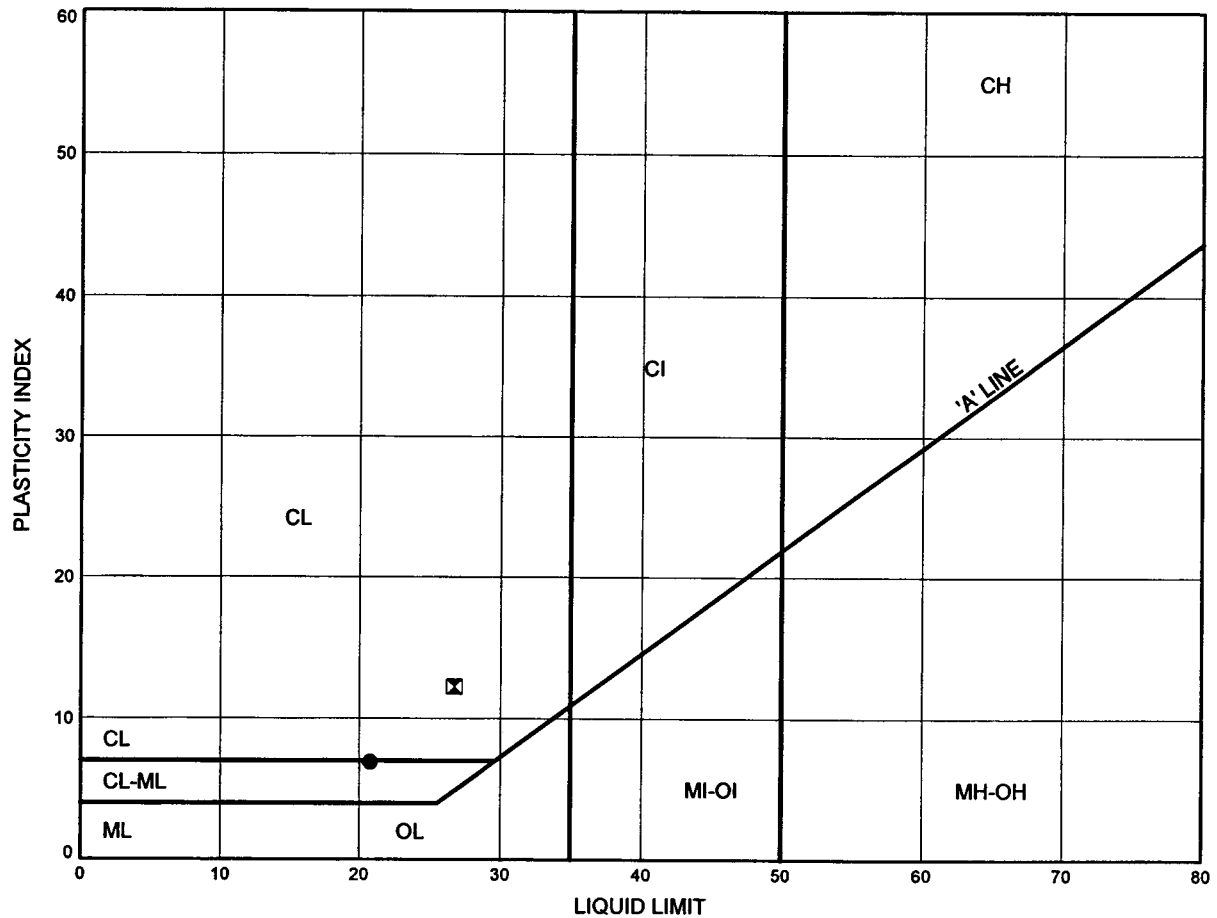
SILTY CLAY TO CLAYEY SILT



HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

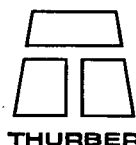
FIGURE B2

SILTY CLAY TO CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CPR-1	3.35	126.55
⊠	CPR-2	2.59	127.31

Date May 2004
 Project 647-92-00

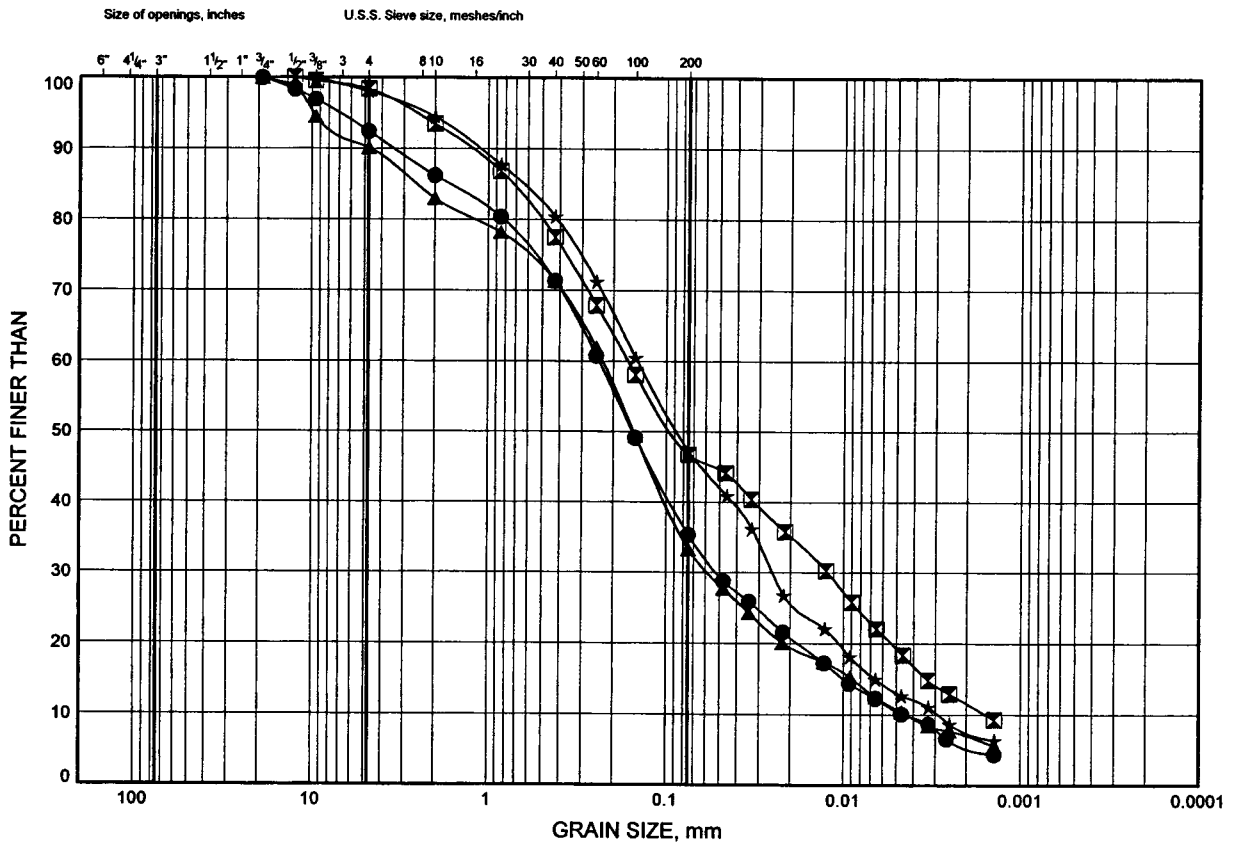


Prep'd SS
 Chkd. SP

HWY 17 Twinning, Amprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B3

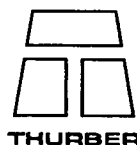
SILTY SAND TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CPR-1	4.88	125.02
⊠	CPR-1	9.45	120.45
▲	CPR-2	4.88	125.02
★	CPR-2	7.92	121.98

Date May 2004
Project 647-92-00



Prep'd SS
Chkd. SP

Appendix C

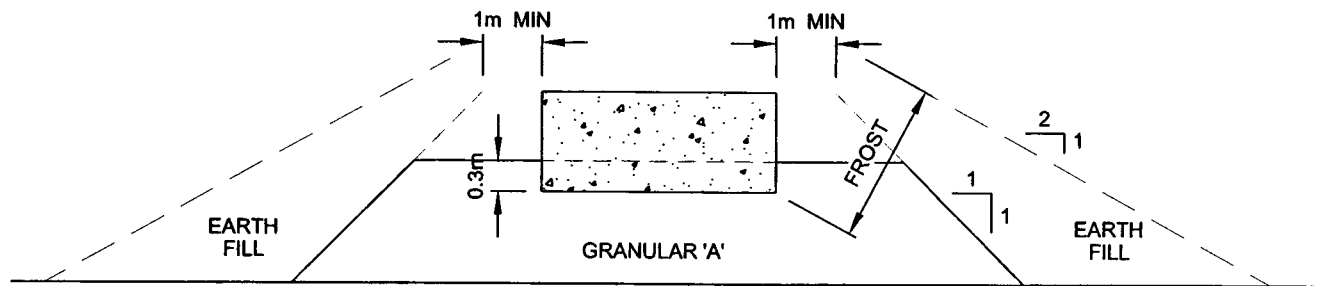
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

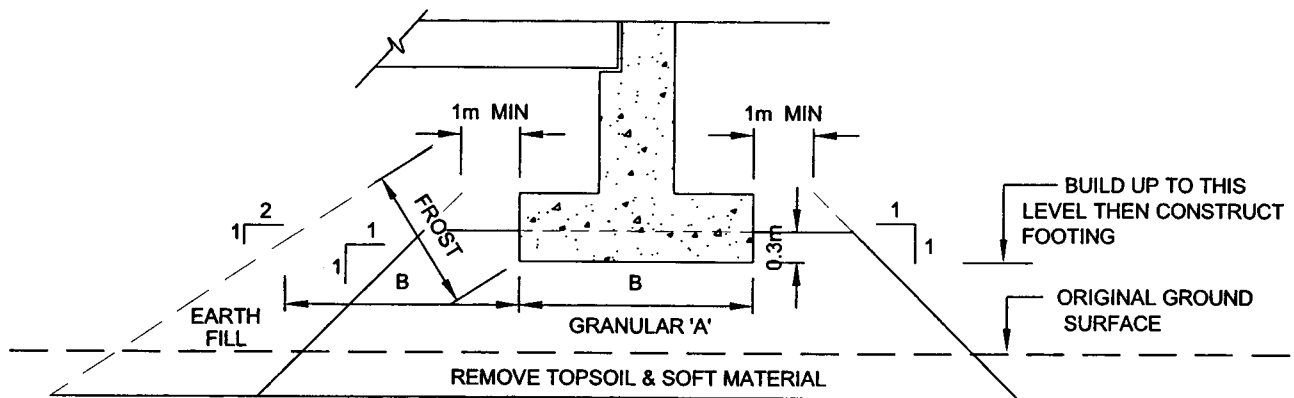
Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Augered Caisson
East and West Abutments	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively high pile capacity is available for end bearing on bedrock. ii. Minimal excavation, if any, is required for foundation construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Piles are required to be socketted into rock if used at integral abutments. ii. Approach fills must be placed prior to installing the piles. 	<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. Deep bedrock surface rendering the use of footing on bedrock impractical. ii. Footing on native compressible soils are undesirable due to potentially large post construction settlements under footing load. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Presence of compacted Granular A core stiffens the founding subgrade and limits post construction settlement to within acceptable limits. ii. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more economical footing dimensions. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. 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. Minimal excavation, if any, is required for foundation construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Nominal rock socketting is required to enhance adequate base contact with bedrock. ii. Not applicable for integral abutments.
East and West Piers	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively high pile capacity is available for end bearing on bedrock. ii. Minimal excavation, if any, is required for foundation construction. <p>Disadvantages:</p> <p>None identified.</p>	<p>Advantages:</p> <p>None identified.</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Deep bedrock surface rendering the use of footing on bedrock impractical. ii. Space limitations for footing construction at the pier locations. 	<p>Advantages:</p> <p>None identified.</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Deep bedrock surface rendering the use of footing on granular pad resting on bedrock impractical. ii. Space limitations for footing construction at the pier locations. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. ii. Minimal excavation, if any, is required for foundation construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Nominal rock socketting is required. ii. Approach fills must be placed prior to caisson installation.

Appendix D

Figures



CROSS-SECTION

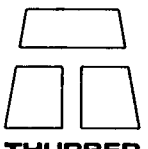


LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

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

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

Appendix E

Drawing

