PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
RECREATIONAL TRAIL
C.N.R. OVERHEAD (ABANDONED)
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-193
GEOCRES Number: 31F-135

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

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site where twin culverts are proposed to provide recreational access under Highway 17. The Ministry of Transportation (MTO) carried out a preliminary foundation investigation in 1971 for the overhead structure at the crossing of Highway 17 and the CNR. The factual data from that investigation has been used as a reference during the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, 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 Ministry of Transportation Ontario (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 Proposed Structure at the Crossing of Hwy #17 'New' E.B.L. and Canadian National Railway" Twp. of Horton – Co. of Renfrew, District No. 9 (Ottawa) W.O. 71-11087 – W.P. 7-67-02, GEOCRES 31F-20-2, dated November, 1971 (Reference 1).

2 SITE DESCRIPTION

The site is located at the presently abandoned railway (CNR) at about Station 25+279 on Highway 17 in the Township of Horton, County of Renfrew. The Borehole Locations and Soil Strata drawing in Appendix D contains further details on the general site location.

At this site Highway 17 presently crosses the abandoned railway right of way via a multi span overpass structure. The site is flat and both sides of the abandoned railway right of way are fairly well vegetated with patches of matured 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. 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. These clay deposits vary in thickness over the region and are interrupted by ridges of rock and sand. 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 dolomite bedrock of the Ordovician Age.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out on October 6 and 7, 2003 and consisted of drilling and sampling three boreholes on the existing abandoned railway embankment to depths ranging from 7.2 m to 8.0 m. The boreholes were numbered REC-1, REC-2 and REC-3 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing (Appendix D).

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited who also provided the coordinates and geodetic elevations of the boreholes. Utility clearances 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. 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). Auger refusal was encountered in all three boreholes at depths ranging from 7.2 m to 8.0 m below ground surface.

A piezometer was installed in each borehole to monitor the ground water level at this site. 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 a 1.5 m long slotted screen was installed at the bottom of the open boreholes. The sand screens surrounding the pipes were about 2m 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 appropriately 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 and secured the soil samples in labelled containers which were than transported to Thurber's Oakville laboratory for further examination and testing.

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 silty clay 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.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

Along the alignment of the abandoned railway embankment, the stratigraphy consists of railway embankment fill underlain by native, very stiff to firm silty clay to clayey silt which is then underlain by loose to very dense sandy silt to silt till.

5.1 Sand to Sand and Gravel Fill

The railway embankment fill was encountered across the site at all three borehole locations. The fill extends to depths ranging from 1.6 m to 2.2 m below the top of embankment, or between Elevations 148.8 m and 146. 5 m.

This fill consists predominantly of sand to sand and gravel, with a clayey silt interlayer present at the west and central portion (Boreholes REC-1 and REC-2) of the alignment. Trace organics and rootlets were found to have mixed with the soil matrix at some locations. Within the sand to sand and gravel, SPT 'N' values of 10 blows to 12 blows for 0.3 m penetration were recorded indicating a compact state. Within the clayey silt portion of the fill, SPT 'N' values of 12 blows and 14 blows for 0.3 m penetration were recorded indicating a stiff consistency. The measured moisture content of samples from this deposit ranged from 5% to 19%, with the higher values typically associated with the clayey silt portion.

5.2 Silty Clay to Clayey Silt

Underlying the embankment fill, a native deposit of silty clay was encountered at depths ranging from 1.6 m to 2.2 m below the ground surface. This cohesive deposit extends from approximate Elevations 147 m to below 142.3 m, or to depths ranging from 6.3 m to 7.2 m.

Grain size analyses were conducted on four samples from this unit and the results are shown in Figure B1. These results indicate that the clay content of this soil ranges between 25% and 34%. The plasticity chart in Figure B2 illustrates the results of Atterberg Limit Tests conducted on selected samples from this silty clay deposit. The tested samples had measured plasticity indices of between 9% and 21% indicating low to medium plasticity (group symbol CL to CI). Furthermore, the moisture contents of samples subjected to the Atterberg Limits Tests are generally close to the Plastic Limit values.

Standard Penetration Tests conducted within this layer gave 'N' values ranging from 4 to 21 blows per 0.3 m penetration. The consistency of this deposit changes from very stiff to firm with depth, and its colour changes from brown to grey at approximate depths of 4 m to 5 m. The measured moisture content of samples from this deposit ranges from 19% to 38%.

From Reference 1, the following information on consolidation characteristics is quoted:

Initial void ratio, e_0	0.68 to 0.76
Compression Index, C_c	0.11 to 0.16
Degree of Preconsolidation, $P'_c - P'_0$	200 to 300 kPa
Undrained Shear Strengths, C_u	45 to 96 kPa

The above information indicates that this silty clay to clayey silt deposit is heavily over-consolidated.

5.3 Sandy Silt to Silt (Till)

The silty clay to clayey silt deposit is further underlain by a grey coloured sandy silt to silt till containing trace to some clay and trace of gravel. This layer was encountered between approximate Elevations 142.3 m and 140.7 m, or depths ranging from 6.3 m to 6.6 m below ground surface. The boreholes were terminated in this deposit at depths ranging from 7.2 m to 8.0 m below ground surface due to auger refusal probably on boulders or bedrock.

A selected sample from this layer was subjected to a grain size distribution test and the results are presented in Figure B3.

The blow counts of Standard Penetration Tests conducted in this layer ranged from 8 to more than 50 blows for 0.3 m penetration. Based on these results the unit is considered to have a loose to very dense relative density. Although not encountered, glacial tills are known to contain cobbles and boulders. The moisture contents of samples retrieved from this layer range from 15% to 24%.

5.4 Bedrock

Bedrock was inferred from auger refusal at the borehole locations.

From Reference 1, the following information on bedrock is quoted:

Type of Bedrock	Crystalline Dolomite
Top of Bedrock	Elevations 140 to 144 m
Conditions of Bedrock	Jointed (vertical) and fractured within upper 2 m

5.5 Water Levels

Standpipe piezometers were installed in all three boreholes. During the first site visit water levels were measured in Boreholes REC-1 and REC-2 and the piezometer installation in Borehole REC-3 was found to be destroyed. Another site visit was made on February 04, 2004 but the remaining two piezometer installations were also found to be destroyed. The following table presents the water level records.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
REC-1	October 22, 2003	2.9	145.7
	February 4, 2004	*	*
REC-2	October 22, 2003	3.1	145.6
	February 4, 2004	*	*
REC-3	October 22, 2003	*	*

* Piezometer installation destroyed.

Based on these observations and information provided in Reference 1, local groundwater levels may exist at elevations ranging between Elev. 145.5 m and 146 m.

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

LICENSED PROFESSIONAL ENGINEER
S. PANG
Sept. 9/04
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 9/04
PROVINCE OF ONTARIO

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

5 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 a suitable foundation system for the proposed structures.

Based on the preliminary general arrangement drawings, two hexagonally shaped box culverts are proposed under the twinned Highway 17 eastbound (EBL) and westbound (WBL). It is understood that the existing bridge which presently carries Highway 17 over the CNR embankment will be demolished. The culverts below the EBL and the WBL will be approximately 57.5 m and 44.5 m long and their centrelines intersect the highway centreline at a skew angle of approximately 32° at mainline Sta. 25+278 and 25+345, respectively. Both culverts will have a 3.7 m high opening and the internal width is expected to vary from 4 m at the top and bottom to about 5.6 m at mid height. Profile grades at approximate Elevations 157.5 m and 154 m are proposed above the culverts at the centre line of Highway 17 E.B.L and W.B.L, respectively. The bottom elevation of both culverts will be about Elev. 147 m.

Reference to the preliminary general arrangement drawings show that about 5 m of fill is proposed above the top of the culvert at the E.B.L location and approximately 1 m of fill will cover the top of the culvert at the W.B.L location. .

The discussion and recommendations presented in this preliminary 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 boreholes at this site show the presence of approximately 1.6 m to 2.2 m of fill underlain by firm to very stiff silty clay to clayey silt. These deposits are further underlain by loose to very dense sandy silt to silt till, containing cobbles and boulders, encountered at the east and west extremities of the site. Bedrock was not proven at this site within the depths of investigation but is inferred to exist (based on auger refusal in the three boreholes) at depths ranging from 7.2 m to 8.0 m below ground surface. Existing MTO logs from Reference 1 indicated that crystalline dolomite bedrock is present at approximate Elevations 140 m to 144 m. Groundwater level exists at elevations ranging from 145.6 m to 145.7 m.

7.1 Foundation Alternatives

This section discusses the feasible foundation alternatives, provides geotechnical design parameters and recommends preferred foundation alternative(s) for this site.

Initial consideration was given to the following foundation types:

- Box and footings on compacted Granular A pad
- Box and footings on native soils
- Augered caissons (drilled shafts)
- Piles

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

The recommended option is hexagonal box culverts and headwall footings founded on compacted Granular A pad resting on the existing railway embankment subgrade. Augered caissons socketted into bedrock may also be considered for use to provide foundation support. Piles driven to bedrock is a possible alternative provided that lateral stability requirements can be satisfied.

The option of box culvert and footings founded directly on native soils is not considered feasible due to the anticipated magnitudes of foundation settlements resulting from the varying heights of new embankment fill and the potentially variable compressibility of the foundation soils.

Further investigation at the detailed design stage would be required to reassess the feasibility of such foundation alternatives.

7.2 Culvert Foundations on Compacted Granular A

Along the alignment of the existing railway embankment, the boreholes indicate that the subsurface consists of compact sand to sand and gravel (with clayey silt interlayer) fill overlying silty clay to clayey silt at elevations ranging from 146.5 m to 147 m. Below the cohesive soils, a deposit of sandy silt to silt till overlies inferred bedrock at approximate Elevation 141 m.

Consideration may be given to founding the culvert box and headwall footings on the railway embankment. However, fills are inherently variable in terms of composition and density, even though available information indicates that the existing sandy fill is in a compact state at the borehole locations. In order to enhance uniformity along the culvert alignment, mitigate differential settlement and increase the load carrying capacity of the upper zones of the foundation soils, it is recommended that a minimum 300 mm thick bedding layer of compacted Granular A be placed on top of the existing fill prior to culvert construction.

The railway embankment subgrade should be proof-rolled and all organics, loose/soft and wet fill and other deleterious materials should be removed and replaced with OPSS Granular A compacted to OPSS 501, Method A specifications (100% of its Standard Proctor maximum dry density (SPMDD) at ±2% of optimum moisture content). The 300 mm thick bedding layer consisting of OPSS Granular A compacted to OPSS 501, Method A specifications may then be placed prior to culvert construction.

It is recommended that the preliminary design of closed bottom culverts and adjacent headwall footings founded on the railway embankment subgrade, prepared as outlined above, be carried out based on the following geotechnical resistances:

- Factored Geotechnical Resistance at ULS of 225 kPa
- Geotechnical Resistance at SLS of 150 kPa

These values are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

The SLS value quoted above corresponds to an estimated post construction settlement in the range of 15 mm to 25 mm. This settlement will essentially occur as a result of elastic recompression of the silty clay to clayey silt deposit. Settlement of the headwall footings will be proportionally less due to the smaller footing width.

The settlement considerations outlined above refer only to settlement due to culvert load. Embankment fill loading will be the predominant cause of settlement that could potentially have adverse effects on the structures. The varying height of new fill to be placed on and around the proposed culverts, and the potentially variable subsurface conditions, could result in total and differential settlements that must be taken into consideration during culvert design. Based on the anticipated embankment heights above the culvert (shown on preliminary GA drawing), preliminary estimates indicated that settlements in the range of 75 mm to 125 mm could be induced near the westerly limit of the EBL culvert; whereas settlements in the range of 40 mm to 70 mm could be induced near the easterly limit of the EBL culvert / westerly limit of the WBL culvert. More detailed settlement analyses must be carried out during the detailed design stage.

Since it is anticipated that the settlements discussed above are largely due to elastic recompression of the over-consolidated clays, consideration could be given to preloading the embankment footprint prior to culvert construction. However, preloading should be carried out with caution such that the existing bridge and approaches will not be adversely affected.

The interior of the proposed culverts will be exposed to freezing temperatures. It will be difficult to prevent the soils adjacent to the culvert from freezing. If the adjacent embankment consists of granular fill, or rock fill, with granular backfill adjacent to the culvert walls, frost actions against the culvert should be minimal. To enhance positive drainage of the surrounding fill, consideration should be given to installing filtered subdrains on both sides of the culvert at sufficient depth below the culvert invert.

It is recommended that the culvert be designed to resist frost forces, lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loadings, surcharge due to construction equipment, stockpiled materials and the like.

The sliding resistance between concrete and the Granular A bedding may be computed on the basis of an ultimate coefficient of friction of 0.7.

7.3 Frost Cover

Frost protection should be provided to the headwall footings at this site. This may take the form of 1.9 m of earth cover, or equivalent thermal insulation, over the underside of the headwall footings.

7.4 Augered Caissons

If higher bearing capacities are required and in order to minimize structure settlement at this site, the culverts may be supported on augered caissons (drilled shafts) founded on bedrock. Based on the preliminary investigations, it is believed that bedrock is likely to exist at depths ranging from 7.2 m to 8 m below the existing top of embankment. Further investigation will be required at the detailed design stage to confirm the depth to bedrock across the site and to assess its engineering properties.

7.4.1 Axial Resistance

For a caisson nominally socketed for 500 mm into bedrock, the axial capacity is assumed to be derived from end bearing only. It is anticipated that a factored geotechnical resistance at ULS of 5,000 kPa may be used for preliminary design assuming sound crystalline dolomite bedrock (Reference 1). This axial resistance should be confirmed at the detailed design stage when the bedrock properties are determined.

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

7.4.2 Downdrag

Downdrag forces will be induced on the caissons as a result of the consolidation of the cohesive foundation soils under the loading of the new embankment 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.

Further investigations will be required at the detail design stage to assess the engineering properties of the silty clay to clayey silt deposit and provide estimates of the magnitude of the downdrag forces. Due to the relatively small thickness and the over-consolidated nature of the cohesive foundation soils, it is estimated that the magnitudes of downdrag force acting on a caisson would be relatively small comparing with its end bearing resistance.

7.4.3 Lateral Resistance

At this site, the caissons would be nominally socketed 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.

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:

<u>Sand and Gravel Fill (compact)</u>

k_s	=	$n_h\ z/D$	(kPa/m)
p_{ult}	=	$3\ \gamma\ z\ K_p$	(kPa)

<u>Silty Clay to Clayey Silt (firm to stiff)</u>

k_s	=	$250\ S_u/D$	(kPa/m)
p_{ult}	=	$9\ S_u$	(kPa)

where z = depth below culvert base in metres
D = caisson diameter in metres
n_h = 6000 kPa/m for compact sand and gravel fill above groundwater table
2000 kPa/m for native sandy silt to silt below the groundwater table
γ = 20 kN/m^3
K_p = 3.0 (passive earth pressure coefficient)
S_u = 50 kPa for undrained shear strength of firm to stiff silty clay to clayey silt

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.

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 diameter (m) and L is the length (m) of the caisson segment or element used for the analysis.

Since the caissons will be 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.

Where the lateral resistance derived from the soils is insufficient to withstand the design lateral loads, consideration may be given to extending the caisson further into bedrock. Further investigations of the bedrock will be required at the detailed design stage to assess its quality and to provide parameters for detailed design.

7.4.4 Caisson Installation

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

Based on the preliminary information and assuming that bedrock exists at depths of 7.2 m to 8.0 m below the top of embankment, the caisson installation would be carried out through railway fill, silty clay to clayey silt, sandy silt to silt till containing cobbles and boulders, and socketted into bedrock. The caissons should be constructed 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.

It is anticipated that a liner advanced into the bedrock will provide cut-off to water seepage expected from the sandy silt to silt overlying probable bedrock. Should water seepage be encountered, the caisson hole should be pumped dry to allow visual inspection of the base. The concrete should be placed using good tremie techniques.

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 fill which comprises of clayey silt, sand, sand and gravel can be classified as Type 3 soil. The native silty clay to clayey silt is classified as a Type 2 soil above the water table and a Type 3 soil below the water table. The sandy silt to silt till is classified as Type 4 below the water table.

8.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

9 GROUNDWATER CONTROL

The groundwater level at the site is about 3 m below existing top of embankment, and below the base of the fill. Since excavations along the culvert alignment are expected to be minor and are not likely to exceed 3 m, no major seepage problems are anticipated. However, the Contractor must make provisions to control any seepage or infiltration of water into excavations by adequate measures such as sump pumps as and where required.

10 EMBANKMENTS ADJACENT TO AND OVER CULVERTS

For the purpose of 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 term (total stress, undrained) and long term (effective stress, drained) conditions where applicable.

Preliminary estimates of foundation settlements have been made based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

Approximately 5 m of fill is proposed directly above the culvert under Highway 17 EBL, and about 1 m of fill is anticipated over the culvert to be constructed under Highway 17 WBL. Immediately outside of the culvert footprints, the depths of the approach fill will increase to about 10 m and 6 m at the EBL and WBL locations, respectively.

10.1 Stability

Within the limits of the railway embankment, the embankment fill will be constructed on fill underlain by very stiff to firm silty clay to clayey silt. Beyond the railway embankment, the new fill will be placed directly on the clay deposit.

Earth fill or rock fill may be used to construct the embankments. Blast rockfill embankments formed with a slope inclination not steeper than 1.25H : 1V is expected to be stable. Earth embankments constructed using granular or select subgrade material is expected to have stable side slopes at inclinations not steeper than 2H : 1V. For the proposed fill configuration, preliminary stability analyses using assumed parameters for the over-consolidated clays under the existing embankment indicated that Factors of Safety (F.S.) in the order of 1.4 and 1.3 can be achieved for both short term (undrained) and long term (drained) conditions, respectively. Figures E1 and E2 present selected stability analyses results. Further analyses using soil information from beyond the existing embankment will be required during detailed design. For an earth fill embankment, a mid-height berm will be required to address surficial stability as discussed in Section 10.3.

10.2 Settlement

Immediate settlement will occur in the fill and the underlying sandy silt to silt till as the new embankment fill is placed. Within the culvert footprints, preliminary calculations indicate that settlement due to elastic recompression of the clay deposit under 5 m of new fill could be in the order of 75 mm to 125 mm as reported previously. Post construction settlement may be considered negligible. Beyond the culvert footprints where up to 10 m of approach fill is proposed, the post construction settlement could be in the order of 25 mm or greater. Differential settlements should also be expected between fill placed over the existing railway embankment and fill placed on the native soils adjacent to the railway embankment. Further investigation, laboratory testing and engineering analyses will be required at the detailed design stage to better quantify the range of settlements and to assess their implications to embankment and culvert foundation design.

As discussed previously, preloading may be considered to accelerate and to enhance uniformity of foundation settlements prior to culvert installation. If the existing bridge is to remain in operation during culvert and fill construction at the EBL, the effects of foundation settlements on the performance of the bridge due to preloading must be evaluated during detailed design.

10.3 Embankment Construction

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

Where earth fill embankments are higher than 8 m, mid-height berms will be required. The berms should have a minimum width of 2 m 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. This requirement of a 2 m wide berm for an 8 m high earth embankment is in place to address surficial stability and to provide access for post construction maintenance.

The approach embankments are considered 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

As an alternative to conventional head walls, retained soil system (RSS) walls may be considered. There is generally low risk associated with using RSS walls at this site. However, further analyses will be required during detailed design in order to confirm the predicted magnitudes of settlements.

12 EARTH PRESSURES

Backfill to the culvert and head walls should consist of free draining, non-frost susceptible granular materials such as Granular A or B (with less than 5% passing the No. 200 sieve) conforming to OPSS 1010 (Special Provision 110F13) requirements. Reference should be made to the backfill arrangements stipulated in OPSD 803.01 or OPSD 803.02, as appropriate. The existing fill material comprising the railway embankment, if excavated, is not suitable for backfilling adjacent to the culvert walls and head walls.

All fills should be placed in loose lifts not exceeding 200 mm thick and be compacted to 98% of its SPMDD at a moisture content within ± 2% of the optimum value. The backfill should be placed and compacted in simultaneous lifts on both sides of the culverts, and the top of the backfill elevation should be he same on both sides of the culverts at all times. Heavy compaction equipment should not be used adjacent to the walls and roofs of the culverts.

For a rigid culvert structure, at-rest horizontal earth pressures should be used for design. The culvert wall should also be designed to resist hydrostatic pressure assuming a groundwater level at the top of the culvert.

If the support system allows yielding of the head 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 $\phi = 35°; \gamma = 22.8$ kN/m^3		OPSS Granular B Type I (modified) $\phi = 32°; \gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°; \gamma = 19.0$ 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	-

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.

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.

Consideration should be given to installing filtered subdrains along the outside walls of the culverts to enhance backfill drainage.

Depending on whether adequate drainage can be provided for the culvert backfill at all times, recommendations on designing for frost forces will be provided, as required, during detailed design.

13 SEISMIC CONSIDERATIONS

13.1 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 culvert alignment is underlain by existing cohesionless and cohesive fill, very stiff to firm clays and discontinous layers of sand silt to silt till, for a combined thickness of 8 m, overlying 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.

13.2 Liquefaction Potential

For culvert foundations and approach fills founded on the existing railway embankment fill (above groundwater level) overlying predominantly native clayey soils, there is negligible potential for soil liquefaction under the foundations.

Beyond the railway embankment, the new approach fills will probably be placed directly on native cohesive soils overlying bedrock at relatively shallow depths. It is likely that the potential for soil liquefaction below the embankments in this area will also be negligible. Further assessment will be required during detailed design to confirm the subsurface conditions.

13.3 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 internal 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	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient (K) for Earthquake Loading					
		Granular A or Granular B Type II $\phi = 35°, \delta = 17.5°$		OPSS Granular B Type I $\phi = 32°, \delta = 16°$		Rock Fill $\phi = 42°, \delta = 21°$	
		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

14 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:

- confirmation that the existing railway embankment fill subgrade is adequately prepared prior to culvert construction.
- confirmation that the culvert backfills and approach fills are adequately placed and compacted to specifications, and that the required drainage and frost protection measures are correctly implemented.
- hand cleaning and inspection of the rock bearing surface if caissons are used.

LICENSED PROFESSIONAL ENGINEER
S. PANG
Sept.9/04
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 9/04
PROVINCE OF ONTARIO

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

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 SAMPLE TYPE

■ Shelby Tube	A - Casing
☐ 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

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

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.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
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

THURBER

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RECORD OF BOREHOLE No REC-1

METRIC

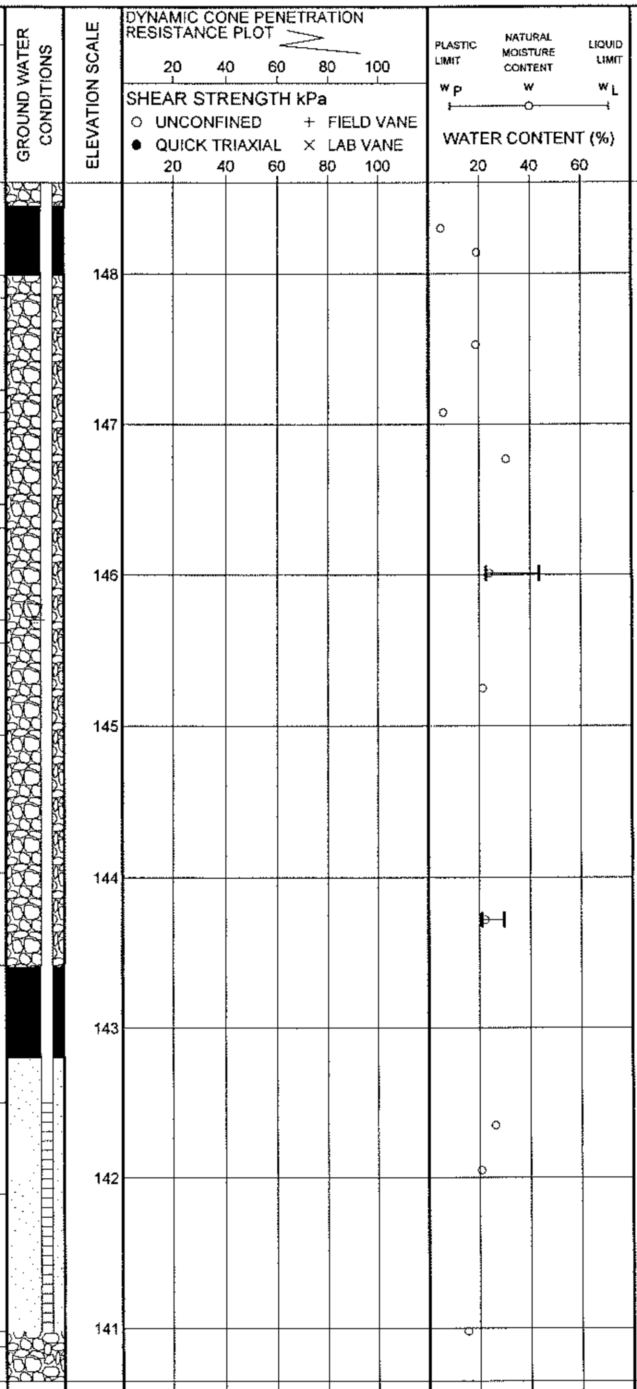
G.W.P. 647-92-00 LOCATION N 5 035 231.4, E 296 165.9 C.N.R. Overhead (Abandoned) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 06.10.03 - 06.10.03 CHECKED BY SP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
148.6 / 0.0	**SAND and GRAVEL** Brown Moist (FILL)				
148.4 / 0.3		1	SS	11	
147.9 / 0.7	Clayey **SILT**, some sand Stiff Brown Moist (FILL)	2	SS	12	
147.0 / 1.6	**SAND**, trace to some silt, trace rootlets, trace organics Compact Brown Moist (FILL)	3	SS	12	
	Silty **CLAY** to Clayey **SILT**, trace sand Stiff to Very Stiff Brown Moist (CL-CI)	4	SS	15	0 8 67 25
		5	SS	21	
	Grey	6	SS	13	0 8 59 34
142.3	with thin sand seams				
6.3	Sandy **SILT** to **SILT**, trace to some clay, trace gravel Compact to Very Dense Grey Wet (TILL)	7	SS	20	
		8	SS	50/.101	
140.7 / 8.0	END OF BOREHOLE AT 7.95m. AUGER REFUSAL AT 7.95m ON PROBABLE BEDROCK OR BOULDERS. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:				

DATE	ELEVATION (m)
22/10/03	145.7
04/02/04	destroyed



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

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THURBER

RECORD OF BOREHOLE No REC-2

1 OF 1 **METRIC**

G.W.P. 647-92-00 LOCATION N 5 035 207.0, E 296 228.0 C.N.R. Overhead (Abandoned) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 07.10.03 - 07.10.03 CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
148.7										
0.0	**SAND and GRAVEL** Compact Brown Moist (FILL)	1	SS	12						
148.0 / 0.7	Clayey **SILT**, some topsoil Stiff Dark Brown Moist (FILL)	2	SS	14	148					
147.3 / 1.5	**SAND**, trace to some silt, trace rootlets Compact Brown Moist (FILL)	3	SS	15	147					
146.5 / 2.2	Silty **CLAY** to Clayey **SILT** Firm Brown Moist to Wet	4	SS	7	146					
145.7 / 3.0	some sand seams Very Stiff	5	SS	17	145		0	6	68	25
144.7 / 4.0	trace gravel Firm Grey	6	SS	7	144					
		7	SS	4	143 / 142					
141.5 / 7.2	END OF BOREHOLE AT 7.16m. AUGER REFUSAL AT 7.16m ON PROBABLE BEDROCK OR BOULDERS									

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

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	ELEVATION(m)
22/10/03	145.6
04/02/04	destroyed

+^{3}, ×^{3}: Numbers refer to Sensitivity   15 ⊕ 5 (20 / 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No REC-3

METRIC

G.W.P. 647-92-00 LOCATION N 5 035 186.7, E 296 279.4 C.N.R. Overhead (Abandoned) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 07.10.03 - 07.10.03 CHECKED BY SP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
148.8 0.0	**SAND**, trace gravel, trace silt, trace organics Compact Brown Moist (FILL)	1	SS	11		
		2	SS	10	148	
146.8 2.0		3	SS	10	147	
	Silty **CLAY** to Clayey **SILT**, some sand seams Very Stiff Brown Moist to Wet (CL-CI)	4	SS	20	146	
		5	SS	13	145	
	Grey Stiff	6	SS	9	144	0 4 64 32
					143	
142.3 6.6		7	SS	8		
	Sandy **SILT** to **SILT**, trace to some clay Loose Grey Wet (TILL) (ML)				142	0 14 69 17
141.3 7.5	END OF BOREHOLE AT 7.52m. AUGER REFUSAL AT 7.52m ON PROBABLE BEDROCK OR BOULDERS Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 destroyed					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

+^{3}, ×^{3}: Numbers refer to Sensitivity   (%) STRAIN AT FAILURE

Appendix B

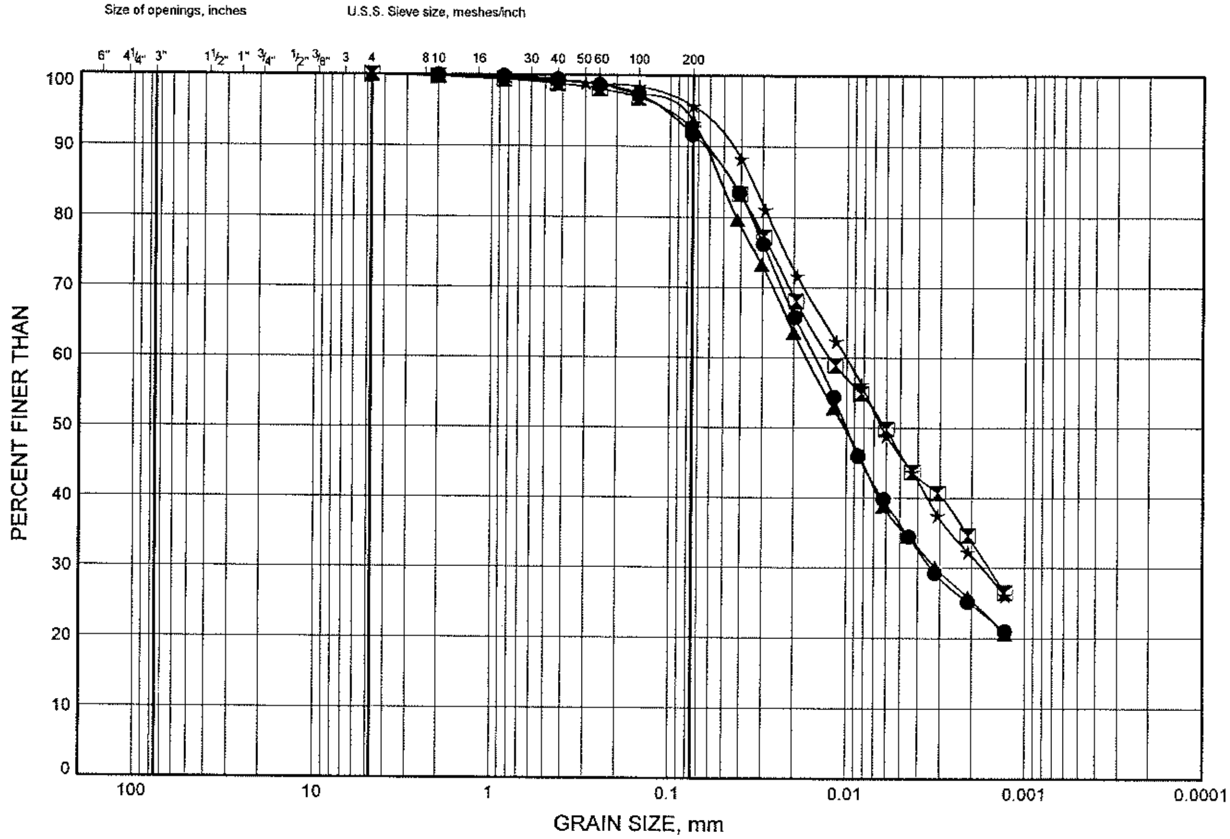
Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY TO CLAYEY SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	REC-1	2.59	146.01
⧗	REC-1	4.88	143.72
▲	REC-2	3.35	145.35
★	REC-3	4.88	143.92

THURBGSD 7450REC.GPJ 07/05/04

Date May 2004

Project 647-92-00

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Prep'd SP

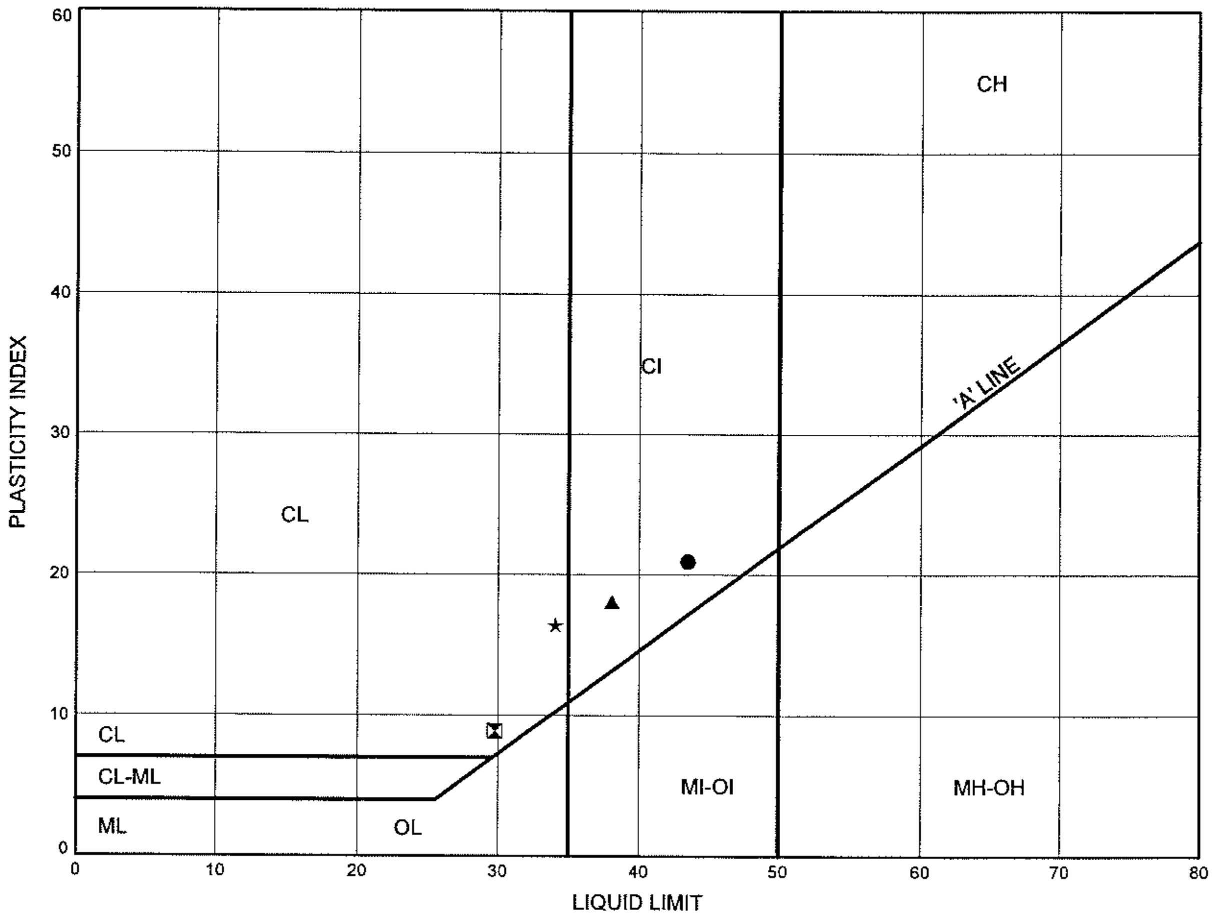
Chkd. SS

HWY 17 Twinning, Arnprior to Renfrew

ATTERBERG LIMITS TEST RESULTS

FIGURE B2

SILTY CLAY TO CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	REC-1	2.59	146.01
⧗	REC-1	4.88	143.72
▲	REC-2	3.35	145.35
★	REC-3	4.88	143.92

THURBALT 7450REC.GPJ 07/05/04

Date May 2004

Project 647-92-00

THURBER

Prep'd SP

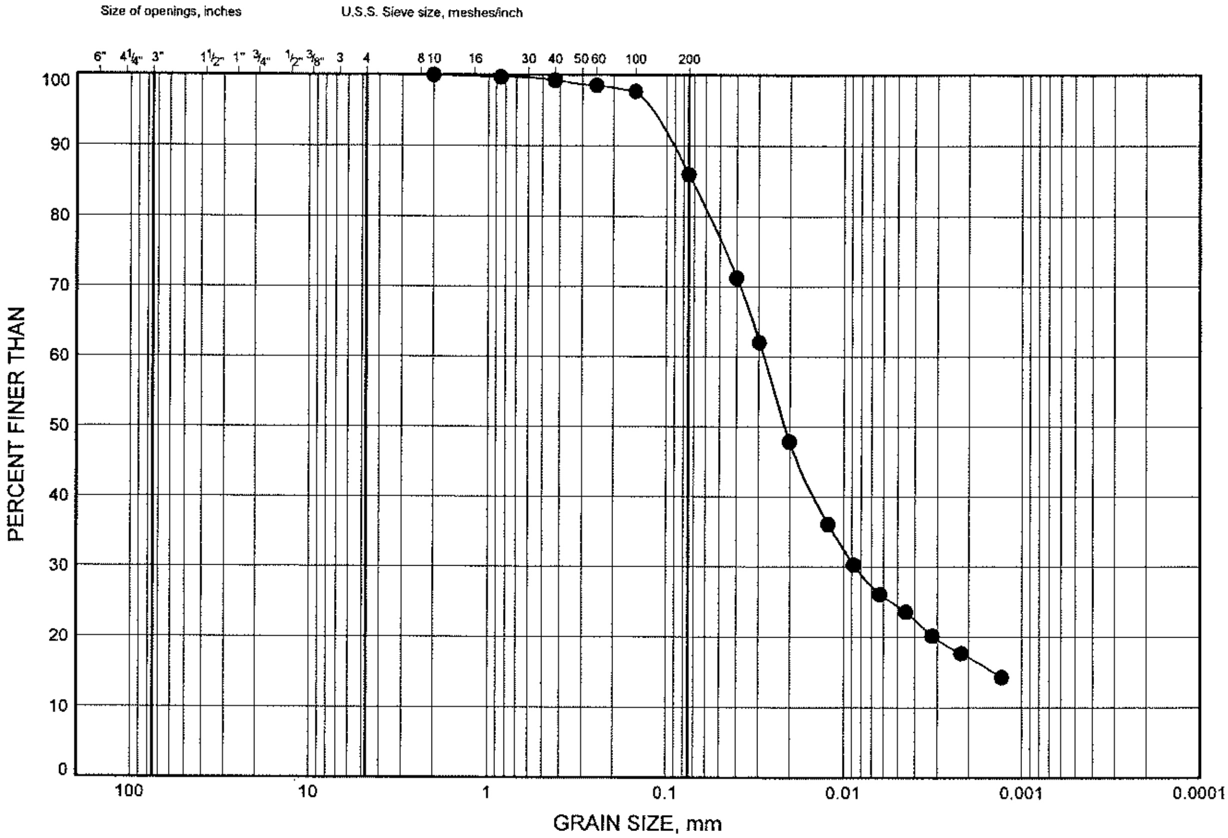
Chkd. SS

HWY 17 Twinning, Arnprior to Renfrew

GRAIN SIZE DISTRIBUTION

FIGURE B3

SANDY SILT TO SILT, some clay



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	REC-3	6.63	142.17

THURBGSD 7450REC.GPJ 07/05/04

Date May 2004

Project 647-92-00

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Chkd. SS

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

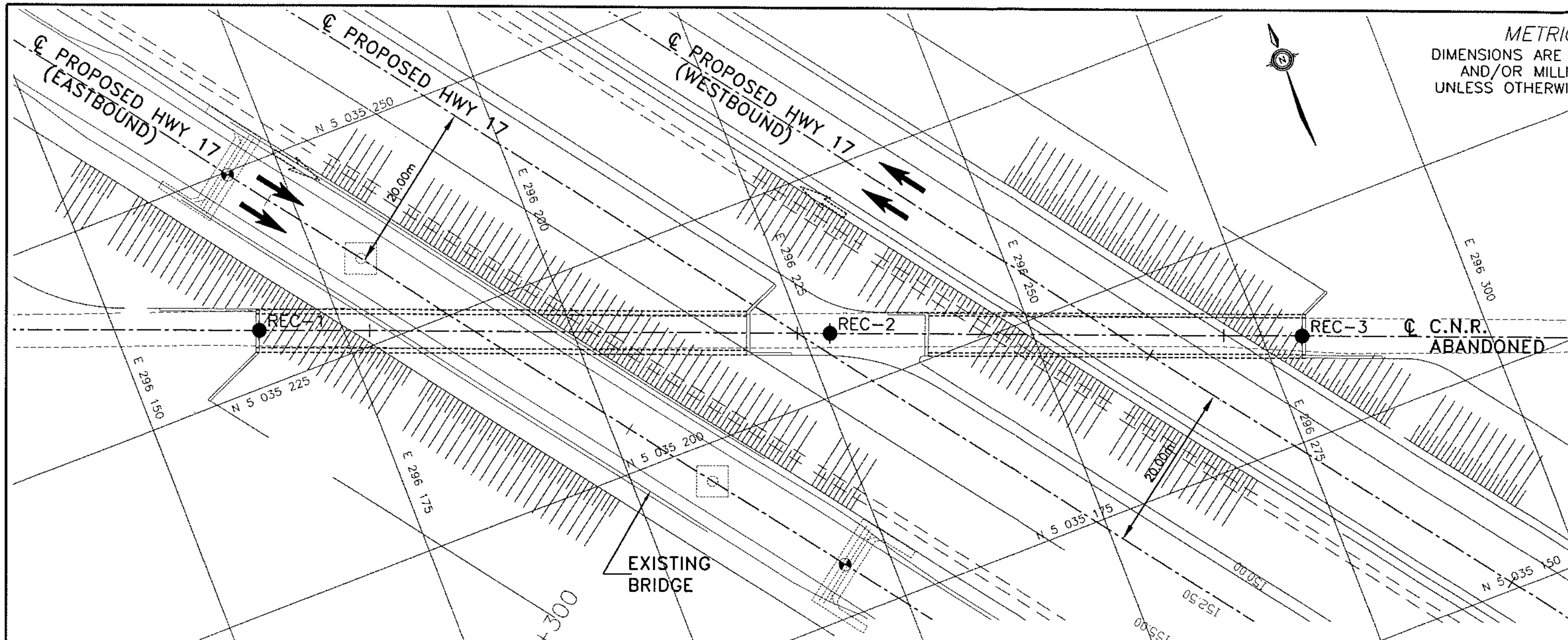
Foundation Element	Driven Pile	Spread Footing	Box Culvert on Railway Embankment Fill Subgrade	Augered Caisson
Culverts below Twinned Highway 17 EBL and WBL	*Advantages:* i. Relatively high pile capacity is available for end bearing on bedrock. *Disadvantages:* i. Relatively shallow bedrock surface rendering the use of driven piles impractical, though technically possible.	*Advantages:* i. High values of geotechnical resistance are available on the bedrock. *Disadvantages:* i. Footing on bedrock requires excavations in the order of 7 m to 8 m making it impractical. ii. Footing on native soils possible, but feasibility will depend on the magnitudes of settlement due to embankment fill loading.	*Advantages:* i. Relative ease of foundation construction and likely the most cost effective alternative. *Disadvantages:* i. Total and differential foundation settlements due to new embankment fill placement must be taken into account during culvert design.	*Advantages:* i. High values of geotechnical resistance are available on the bedrock. *Disadvantages:* i. Rock socketting is required to enhance seating on bedrock. ii. Specific design and construction requirements between caissons and culvert. iii. Likely less cost effective as the culvert on fill subgrade alternative.

THURBER

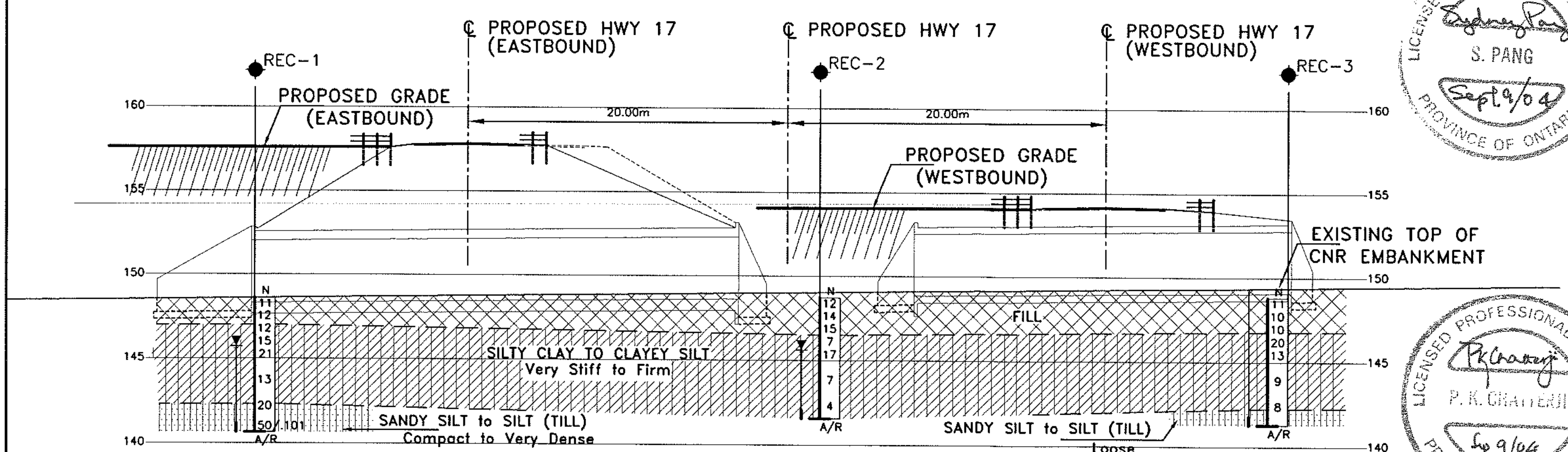
Appendix D

Drawing

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



PLAN



PROFILE ℄ C.N.R. ABANDONED

LICENSED PROFESSIONAL ENGINEER
S. PANG
Sept 9/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 9/04
PROVINCE OF ONTARIO

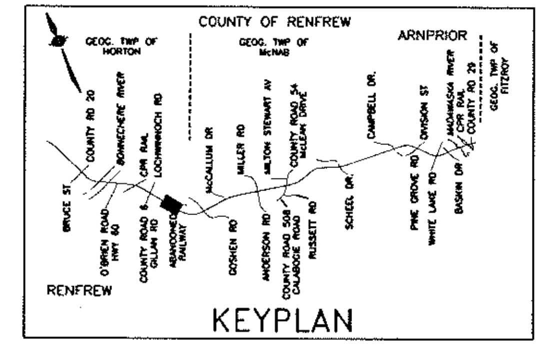
HWY.17
GWP NO. 647-92-00

SHEET

HIGHWAY 17 TWINNING
C.N.R. ABANDONED
BOREHOLE LOCATIONS AND SOIL STRATA

TSH engineers architects planners

THURBER ENGINEERING LTD.



KEYPLAN

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
REC-1	148.6	5 035 231.4	296 165.9
REC-2	148.7	5 035 207.0	296 228.0
REC-3	148.8	5 035 186.7	296 279.4

— 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
APR. 04	SP	ISSUED AS DRAFT FOR REVIEW

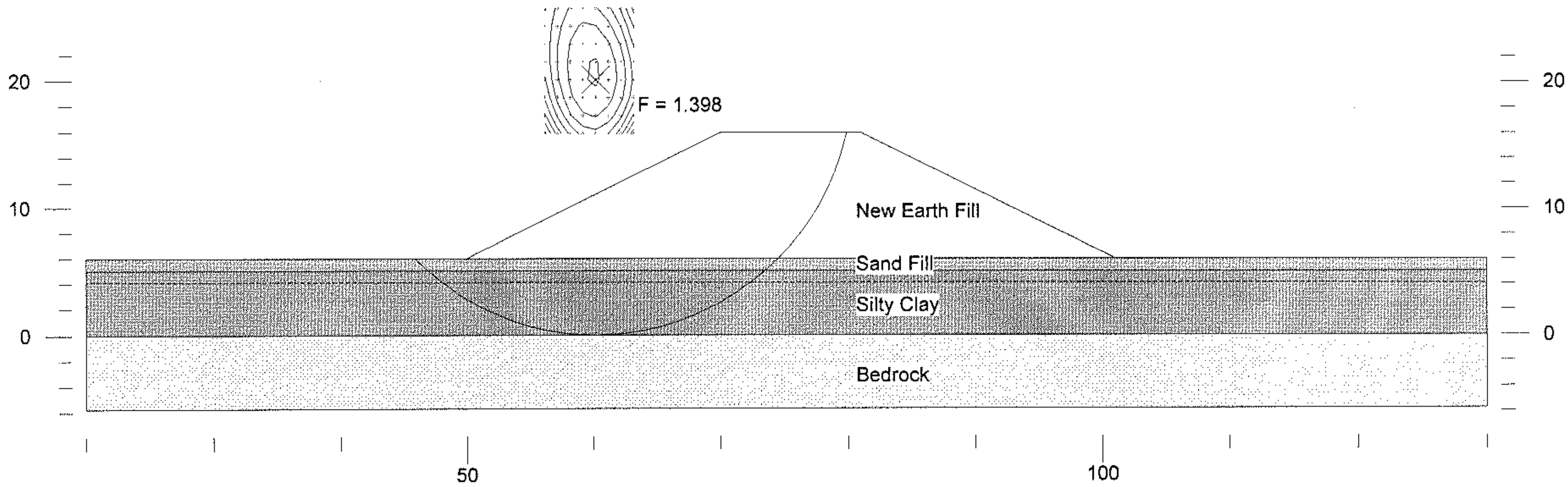
DESIGN SP	CHK PKC	CHBDC 2000	LOAD	DATE SEP. 2004
DRAWN SS	CHK SP	SITE 29-193	STRUCT	DWG.

Appendix E

Selected Stability Analyses Results

Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning - Recreational Trail
September 3, 2004
Stability of Embankment Slope
Figure E1 Undrained Analysis - 10 m high embankment

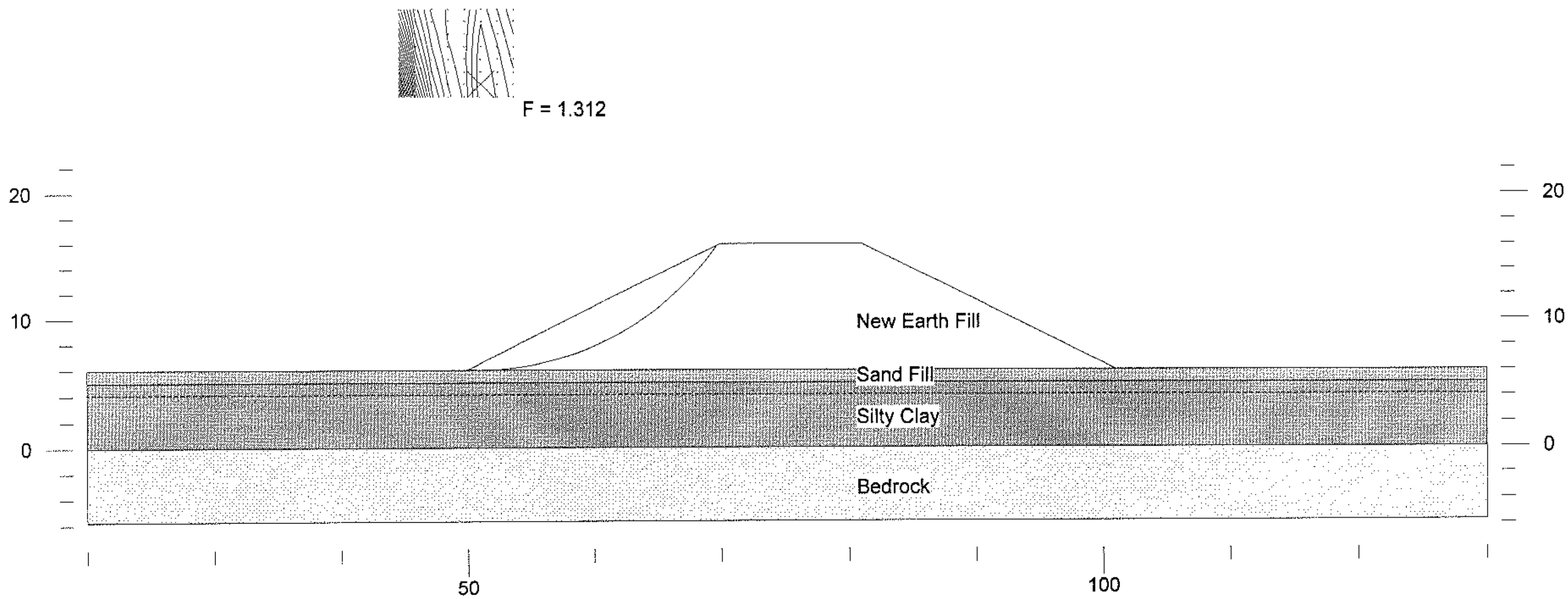
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Earth fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Silty Clay	19	50	0	0	0
Bedrock	(Infinitely Strong)				



03/09/2004 1:24:44 PM C:\SPANG\THURBE~8\19-374~1\RECREA~1\FIGE1.GSL Thurber Engineering Ltd. - Toronto F = 1.398

Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning - Recreational Trail
September 3, 2004
Stability of Embankment Slope
Figure E2 Drained Analysis - 10 m high embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Earth fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Silty Clay	19	0	28	0	0
Bedrock	(Infinitely Strong)				



03/09/2004 1:36:33 PM C:\SPANG\THURBE~8\19-374~1\RECREA~1\FIGE2.GSL Thurber Engineering Ltd. - Toronto F = 1.312