

FOUNDATION INVESTIGATION AND DESIGN REPORT

MADAWASKA RIVER BRIDGE

HIGHWAY 17 TWINNING

ARNPRIOR TO RENFREW, ONTARIO

G.W.P. 647-92-00, SITE NO. 29-191/1

GEOCRES Number: 31F-130

Report to

National Capital Engineering

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out at the location where a new bridge will carry Highway 17 westbound lanes over the Madawaska River. During previous investigations for the existing Highway 17, boreholes were drilled by the Ministry of Transportation (MTO) in the vicinity of the existing bridge. The factual data obtained from those investigations has been used as reference during the preparation of this report.

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

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

The following documents are referenced in the preparation of this report:

- MTO report titled "Foundation Investigation and Design Report, Madawaska River Bridge (Arnprior Diversion)", Hwy. 17N, Dist. 9, W.P. 198-62-00, Str. Site 29-191, GEOCRES No. 31F-94, dated July 19th, 1977 (Reference 1).
- MTO report titled "Preliminary Foundation Report For Structure Crossings of Revised Hwy #17 From Antrim Westerly to Locheil Creek, Regional Municipality of Ottawa, Carleton and Renfrew County", District No. 9 (Ottawa), W.J. 69-F-86 and W.P.'s 5-67 & 190-67, GEOCRES No. 31F-23 dated March 12, 1970 (Reference 2).
- Internal memoranda between the MTO Structural Planning and Design Offices and the Soil Mechanics Section, Geotechnical Office, Downsview.

2 SITE DESCRIPTION

The site is located immediately to the north of the existing Madawaska River Bridge and to the south of the Town of Arnprior, Township of McNab, County of Renfrew (between approximate mainline Stations 30+850 and 31+200) on the existing Highway 17. This project area is located just downstream (north) of the existing Hydro Dam of the Arnprior Generating Station. The general site location is shown on the Borehole Locations and Soil Strata drawing in Appendix F.

At the bridge crossing, the Madawaska River, which flows from south to north, is in the order of 100 m wide and is deeply incised into the surrounding lands. Available information indicates that the river is approximately 14 m deep at this location. Within the river valley, the terrain is fairly rugged with bedrock outcrops, moderately to steeply sloping rockfaces and bedrock underlying thin veneer of soils. The east bank is lightly vegetated, whereas the west bank is heavily vegetated with large deciduous and coniferous trees.

The project area is located between the Laurentian upland to the north and west, and the Ottawa lowland to the south and east. This area is situated within a physiographic region known as the Ottawa Valley Clay Plains. 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. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechère” graben. Bedrock in the site area consists of crystalline limestone of the Ordovician Period that had been subjected to faulting, weathering and erosion most notably by the Madawaska River.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of August 6 to 8, 2003 and on August 13, 2003. The investigation program consisted of drilling and sampling twelve (12) boreholes (numbered MRB-1, MRB-7, MRB-8, MRB-10, MRB-12, MRB-13 to MRB-14, MRB-19 to MRB-22) to depths ranging from 0.6 to 8.7 m. At three additional locations, test pits (numbered MRB-5, MRB-9A and MRB-11) were excavated using a rubber tire backhoe because these locations were inaccessible with a conventional drill rig. At eight other locations (numbered MRB-2, MRB-3, MRB-4, MRB-6, MRB-9, MRB-15, MRB-16 and MRB-17), rock was either exposed or covered by a veneer of topsoil or surficial soil. At these locations, a hand shovel was used to remove the topsoil and the rock surface was visually inspected. The approximate locations of the boreholes, machine and hand dug pits are shown on the Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were staked or marked in the field by surveyors from J. D. Barnes Limited, who subsequently surveyed the actual hole locations and provided the coordinates and geodetic elevations. At the time of preparation of this draft report, the elevations for Boreholes MRB-10 and MRB-18 were interpolated. Utility clearances at the investigated locations were obtained by Thurber prior to commencing the field investigations.

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 at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) where overburden soils were encountered. At each of the seven locations across the site, approximately 3m of bedrock was cored by NQ size rotary coring techniques.

Groundwater conditions in the open boreholes and test pits were observed throughout the drilling operations. One standpipe piezometer was installed in Borehole MRB-8 on the west bank for monitoring of the groundwater level. It was not practical to install a piezometer on the east bank due to very shallow bedrock and outcrop. At this site, a 19 mm diameter Schedule 40 PVC pipe with a 1.52 m long slotted screen was installed in the borehole open to the surface of bedrock. The sand screen surrounding the pipe was about 1.9 m long. The remaining space in the borehole was grouted with a bentonite-based grout.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport 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.

On completion of drilling and sampling, the boreholes and test pits were appropriately backfilled.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were subjected to gradation analysis. Atterberg Limit Tests were performed on selected samples retrieved from the cohesive deposits. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point load tests were performed at selected locations on rock cores retrieved from the boreholes and these results are shown in Table 1 attached immediately following the text.

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 also presented on the "Borehole Locations and Soil Strata" drawing in Appendix F. A generalized description of the stratigraphy is given in the following paragraphs.

In general, the limestone bedrock is either exposed at the site or covered by relatively shallow deposits of overburden. The overburden soils range in thickness from 0 to 5.7 m and consists of topsoil, silty clay, sand and silt, cobbles and boulders and rockfill.

5.1 Topsoil

Topsoil was encountered across the site in nineteen borehole or test pit locations with thickness ranging from 25 mm to 425 mm as shown below.

Borehole	Topsoil Thickness (mm)
MRB-1	425
MRB-2	300
MRB-3	50
MRB-4	25
MRB-5	400
MRB-7	100
MRB-8	100
MRB-9	150
MRB-9A	175
MRB-10	75
MRB-11	175
MRB-12	75
MRB-13	175
MRB-15	225
MRB-17	50
MRB-19	100
MRB-20	100
MRB-21	125
MRB-22	150

Topsoil thicknesses may vary between and beyond borehole locations.

5.2 Silty Clay

At some of the borehole locations at the west pier, west and east abutments (MRB-8, MRB-9A, MRB-10, MRB-11, MRB-12, MRB-20 and MRB-22), the topsoil is underlain by a silty clay deposit encountered at depths of 0.1 m to 0.2 m below existing ground surface, or from Elevation 83.5 m to 79.5m. This deposit extends to depths of 0.6 to 4.3 m, or from Elevations 82.9 m to 76.9m. The silty clay is brown in colour becoming grey below a depth of 2 m to 3 m.

Grain size analyses conducted on two silty clay samples are presented in Figure B1. Atterberg Limit tests were also conducted on two selected samples from this stratum and the results are illustrated on the plasticity chart in Figure B2. The silty clay samples had measured plasticity indices ranging between 18% and 23%, and corresponding liquid limits of 34% and 42%, respectively, indicating clayey soils of low to medium plasticity (group symbols of CL to CI).

Standard Penetration Tests conducted within this deposit gave 'N' values ranging typically between 23 blows and 4 blows per 0.3 m penetration. Based on these results, the consistency of the deposit is considered to be generally stiff to very stiff with some firm zones such as that below 3 m depth in Borehole MRB-8. It should be pointed out that the presence of cobbles, boulders and rock pieces within the deposit (see Boreholes MRB-20 and MRB-22) may have resulted in occasional higher 'N' values of 50 blows for less than 0.3 m penetration. The measured moisture contents of samples recovered from this unit ranged from 13% to 33%.

5.3 Sand and Silt

A deposit of sand and silt was encountered, underlying the silty clay or directly below the topsoil, at locations near the west bank of the Madawaska River (Boreholes MRB-8, MRB-9 and MRB-10) and the east limit of the river valley (Borehole MRB-21). This deposit contains trace to some gravel, occasional cobbles, boulders and rock pieces. In Borehole MRB-8 where frequent cobbles and boulders were encountered below a depth of 4.3 m, a combination of coring and wash-boring techniques were required to extend the borehole. At this location the sand and silt layer is inferred to exist based on an examination of the soil cuttings in the drill water. The deposit was encountered at depths ranging from 0.1 to 4.3 m and it extends to depths varying from 0.6 to 5.7 m below the ground surface.

Samples from this deposit were subjected to grain size analyses and the grain size distribution curves are shown in Figure B3. The results show that the soil matrix contains trace to some gravel and trace to some clay sized particles.

Standard Penetration Tests conducted in this deposit yielded 'N' values of 38 blows to more than 50 blows per 0.3 m penetration. Based on these blow counts, the deposit is considered to be in a dense to very dense state. The measured moisture contents ranged from 3% to 7%.

5.4 Rockfill

Near the east bank of the Madawaska River at the location of Borehole MRB-18, rockfill was observed to extend beyond the river bank and below the water level in the river.

5.5 Bedrock

The soils described above are underlain by crystalline limestone bedrock. Bedrock was proved by coring in Boreholes MRB-1, MRB-7, MRB-8, MRB-13, MRB-14, MRB-19 and MRB-20. The bedrock surface was inferred from auger refusal or from visual inspection of the rock surface in the remainder of the boreholes. The bedrock profile is variable and outcrops are visible across the site. At most locations, the bedrock is either covered by a veneer of topsoil, or covered with up to 5.7 m of overburden soils near the west bank of the Madawaska River. The table below summarizes the depth to bedrock.

Borehole Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
MRB-1	0.4*	87.6*
MRB-2	0.3	87.9
MRB-3	0.05	87.7
MRB-4	0.025	88.0
MRB-5	0.4	86.0
MRB-6	Outcrop	87.4
MRB-7	0.1*	88.2
MRB-8	5.7*	75.9
MRB-9A	3.2	76.8
MRB-10	2.9	79.0**
MRB-11	1.3	78.4
MRB-12	0.6	81.5
MRB-13	0.2*	81.6
MRB-14	Outcrop*	79.1
MRB-15	0.2	79.6
MRB-16	Outcrop	79.1
MRB-17	0.1	79.7
MRB-19	0.1*	78.8
MRB-20	0.6*	82.9
MRB-21	1.6	85.0
MRB-22	1.0	82.6

Notes : * Proven by coring

** Borehole elevation not yet available

The limestone is very thinly to thinly bedded and generally in a fresh to slightly weathered state. In Borehole MRB-14, the upper 0.5 m of cored bedrock was noted to be moderately weathered. The rock cores are grey in colour with dark grey and white horizontal and sub-vertical banding.

The measured Total Core Recovery (TCR) for the core runs vary between 88% and 100%. The Rock Quality Designation (RQD) values ranged from 56% (in Borehole MRB-14) to 100%, but typically between 70% and 100% indicating a fair to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 3, except in Boreholes MRB-8, MRB-14, and MRB-19 where occasional higher FI values of 5 and 6 were recorded. The joints were generally oriented sub-vertically and were mostly rough and tight with no infilling or secondary weathering material.

The inferred Unconfined Compressive Strength (UCS) of intact rock cores (expressed as average value per run) range between 62 MPa to greater than 100 MPa indicating that the intact rock is strong to very strong. In Borehole MRB-7, the strength of the rock cores was inferred to be in the order of 49 MPa between 0.1 m to 1.6 m inferring moderately strong rock within this zone. These estimated rock strength values are based on point load tests that were conducted at selected locations on rock cores recovered from the boreholes. A summary of the Point Load Test results is presented in Table 1 attached immediately following the text.

5.6 Water Levels

A standpipe piezometer was installed in Borehole MRB-8 advanced near the west bank of the Madawaska River. The water level in this piezometer was measured during three separate visits after the completion of drilling. These readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
MRB-8	October 22, 2003	3.3	78.3
	February 04, 2004	3.5	78.1
	March 11, 2004	3.0	78.6

Based on these observations, the local groundwater level at the river banks appears to exist at about Elevation 78 m and is likely influenced by the water level in the Madawaska River. According to Reference 1, the river water level at this site is controlled by the hydro dam and generally fluctuates between Elevations 77.4 m and 78.6 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.



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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

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

It is understood that the design plan calls for the construction of a new bridge to carry the two new westbound lanes (WBL) of the twinned Highway 17 over the Madawaska River. The existing bridge will carry the existing lanes that will become the twinned Highway 17 eastbound lanes (EBL). The west and east limits of the new bridge and approaches correspond to approximate mainline Stations 30+850 and 31+200, respectively.

Based on the preliminary general arrangement (GA) drawing, it is understood that the proposed bridge will be parallel to, and located to the north of, the existing bridge. It will have three-spans comprising steel plate girders. The bridge will be approximately 300 m long, with two equal approach spans of 85 m and a centre span of 130 m. A profile grade of approximate Elevations 96m and 101 m for the proposed west and east abutments, respectively, are inferred from the preliminary GA drawing. Consequently, approach fill heights of up to 10 m and 18 m will be required at the west and east approaches, respectively.

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 this investigation.

7 STRUCTURE FOUNDATIONS

7.1 General

The proposed bridge at this site will have three spans supported on four foundation elements: two abutments and two piers.

The subsurface stratigraphy at the abutment and pier locations mainly consists of surficial topsoil and shallow silty clay deposits with occasional sands and silts, cobbles and

boulders overlying bedrock. The overburden is slightly thicker at the west pier location and rock fill overlies bedrock at the east pier location. Bedrock outcrops are also present at several locations.

The elevations at which bedrock was proven, observed or inferred at the foundation elements are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
West Abutment			
Northwest corner	MRB-1	88.0	87.6*
Northeast corner	MRB-2	88.2	87.9±
West centre	MRB-3	87.8	87.7±
East centre	MRB-4	88.0	88.0±
Southwest corner	MRB-6	87.4	87.4**
Southeast corner	MRB-7	88.3	88.2*
West Pier			
Northwest corner	MRB-8	81.6	75.9*
Northeast corner	MRB-9A	80.0	76.8±
West centre	MRB-10	81.9***	79.0±***
East centre	MRB-11	79.7	78.4±
Southwest corner	MRB-12	82.1	81.5±
Southeast corner	MRB-13	81.8	81.6*
East Pier			
Northwest corner	MRB-14	79.1	79.1**
Northeast corner	MRB-15	79.8	79.6±
West centre	MRB-16	79.1	79.1**
East centre	MRB-17	79.8	79.7±
Southwest corner	MRB-18	79.0***	Below rockfill
Southeast corner	MRB-19	78.9	78.8*
East Abutment			
Northwest corner	MRB-20	83.5	82.9*
Southeast corner	MRB-22	83.6	82.6±

* Proven by coring.

** Outcrop.

*** Elevation interpolated from adjacent boreholes (survey data not yet available).

7.2 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.

While an integral abutment is possible at the east abutment, it is considered impractical to have an integral abutment at the west abutment due to limited height of proposed fill above the bedrock surface. Should an integral abutment design be pursued, any foundation piles will have to be socketted into bedrock. The span lengths currently anticipated may be too long for a semi-integral abutment design.

At the west abutment location, the proposed embankment height of about 8 m renders it impractical to use driven piles or augered caissons. The option of footing on engineered fill is also not considered feasible due to the limited embankment height between the bedrock surface and the proposed Highway 17 grade. At the east abutment where the proposed embankment height is up to 18 m, a perched abutment design should be considered where piles driven to bedrock are suitable for foundation support. Consideration may also be given to using augered caissons founded on bedrock. Both of these deep foundation options will require construction of an engineered fill platform to accommodate pile or caisson installation equipment. This fill may constitute the core of the east approach embankment. Deep foundations are not suitable for use at the piers.

In view of the above, it is recommended that spread footings founded on bedrock be used at the west abutment and both piers. Perched abutment founded on steel piles driven to bedrock is the preferred option for the east abutment. At the east abutment, the use of footings on an engineered fill pad is possible, provided that the longitudinal differential settlement along the east approach span, due to lower bearing resistance available from the fill pad, is acceptable.

7.3 Spread Footings on Bedrock

7.3.1 General

Based on the subsurface stratigraphy and the proposed vertical alignment of the highway, this option is feasible for the west abutment and the two piers.

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

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

Where practicable, the underside of the concrete footing should be designed to lie approximately 200 mm above the local top of rock and the difference made up using mass concrete fill. Where necessary, the footing may be stepped down across the width of the structure to accommodate changes in the elevation of the top of bedrock. This approach will reduce the risk of having to excavate bedrock under a footing. The recommended design top of rock is as follows.

West Abutment

The top of rock varies between approximate Elevations 87.4 m and 88.2 m across this foundation.

West Pier

The top of rock varies from approximate Elevation 75.9 m at the northwest corner of the proposed footing to Elevation 81.6 m at the southeast corner of the proposed footing.

In view of the close proximity of the proposed west pier to the steeply sloping rock faces adjacent to the river bank and the potentially adverse joint orientation associated with the bedrock, it is recommended that the footing be positioned such that a 45° degree line drawn from any point of the base of the footing should not intersect with the exposed face of the rock slope at the river bank. In addition, a minimum horizontal distance of 1.5 m should be maintained between the outermost edge of the footing base and the crest of the rock slope at the river bank. It is also recommended that the west pier footing be designed for a lower bearing resistance than at the abutments (see later Section 7.3.2) in order to take into account the close proximity of the steep river banks.

Depending on the location, orientation and height of the exposed rock slope with respect to the pattern of joints or fractures within the rock mass, potentially unstable rock wedges may exist below the pier foundation. After the rock subgrade is exposed, the Contractor should remove any loosen rock and the Contract Administrator (CA) should retain a rock slope stability/rock mechanics specialist to examine the rock slope and subgrade. Should any potentially unstable wedges or over-break zones be identified, rock dowelling, rock anchoring and/or mass concreting may be necessary to reinforce the rock mass prior to footing construction. Any remedial work should be designed by and carried out under the supervision of the rock slope stability/ rock mechanics specialist retained by the CA.

The contract should include an NSSP to this effect.

It is recommended that construction be carried out during the season of lowest river level. Control of water influx will be required for footing construction since the bedrock surface, and therefore the footing base, is below the river level at some locations. Consideration may be given to measures such as an impervious earth dyke in conjunction with sumps and pumps to provide water cutoff surrounding the footing.

It is recommended that river bank and scour protection be provided to the foundations. Such measures of protection may include suitably graded rip-rap and/or armour stone, and should be designed based on river hydraulics by qualified professionals experienced in this field.

East Pier

The top of rock varies between approximate Elevations 78.8 m and 79.7 m across this foundation.

Rock fill overlies bedrock at the proposed east pier footing location. This rock fill should be removed to its full depth and the pier footing founded directly on sound bedrock. Results of existing boreholes indicate that the rock quality and strength at this location is lower than elsewhere at this site. It is, therefore, recommended that the east pier footing be designed for a lower bearing resistance than that at the abutments (see later Section 7.3.2).

Recommendations for the west pier on issues including rock slope and face stability, unwatering during construction, river bank and scour protection are also applicable to the east pier, where required.

7.3.2 Bearing Resistance

Footings bearing on crystalline limestone bedrock encountered at this site may be designed for the following resistances:

- Factored geotechnical resistance of 3,000 kPa at Ultimate Limit States (ULS) for the west abutment.
- Factored geotechnical resistance of 1,500 kPa at Ultimate Limit States (ULS) for the west and east piers.

These recommended bearing resistances are calculated based on the Hoek-Brown rock strength criterion used in conjunction with rock mass classification procedures. These methods require input of UCS and RQD values for the rock which are as low as 50 MPa and 56%, respectively, and unfavourable sub-vertical joint orientations at the pier and abutment locations. The recommended bearing values are generally consistent with those recommended in Reference 1 (MTO foundation report for the existing Madawaska bridge).

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

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

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

7.3.3 Horizontal Resistance of Footings

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

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

The dowel may be considered as acting as a fully embedded short pile in the rock. Using a lower bound value of 30 MPa for the unconfined compressive strength of the rock (reduced to allow for fracturing near the surface) or the strength of the grout, an ultimate horizontal resistance of 1.3 MN was calculated for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing.

The structural capacity of the selected dowel should not be exceeded. It is noted that the above design assumes that a drilled hole in bedrock has a diameter just large enough to accommodate the dowel and, as such, the compressive strength of the rock governs design.

7.4 Spread Footings on Engineered Fill

For a perched abutment design at the east approach, spread footings may be founded on an engineered fill pad that is itself resting on bedrock. As discussed previously, this option is feasible provided that differential settlement of the east approach span (between the east pier and the east abutment) in the order of 20 mm can be accommodated.

If an engineered fill pad is used to support the footing, all overburden materials including topsoil, fill and native soils should be removed, and the new fill placed directly on the bedrock surface. 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 Granular A pad has a minimum thickness equal to the width of the footing.

For a footing founded on a compacted Granular A pad resting on bedrock, the design may be carried out assuming the following minimum values:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 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.

7.5 Driven Piles

Steel piles driven to bedrock may be used to provide foundation support for a conventional abutment at the east approach. Based on the borehole information, bedrock is present at within 1 m depth below existing ground surface at the east abutment. The following pile tip elevations are recommended for design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
East Abutment	MRB-20, MRB-22	82.6± to 82.9±

7.5.1 Axial Resistance

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

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

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

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

7.5.2 Downdrag on Piles

Due to the shallow depth of about 1 m to bedrock at this site, downdrag is not considered to be a design issue provided that the engineered fill core, through which the piles are to be installed, is well compacted (see Section 7.5.4) prior to driving the piles.

7.5.3 Lateral Resistance

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

Given the shallow depth to bedrock and the proposed final grade of the highway, much of the soil lateral resistance would be derived from the approach fill. For lateral soil-pile interaction analysis, 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:

Engineered Sand and Gravel Fill (compact)

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

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa}) \quad (\text{at and above Elevation 83 m})$$

where z = depth below abutment base in metres

D = pile diameter in metres

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

γ = 20 kN/m³

K_p = 3.0 (passive earth pressure coefficient)

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, $K = k_s \times L \times D$ (MN/m), where k_s is the coefficient of horizontal subgrade reaction (MPa/m), D 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.

7.5.4 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 and cobbles 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 the east abutment location. In order to enhance adequate seating into bedrock, it is recommended that the pile tips be reinforced with rock points such as the Titus "H" Bearing Pile Point, Rock Injector design, or equivalent.

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

7.6 Frost Cover

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

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

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

- The foundation is underlain by at least 2.5 m of free-draining, non-frost susceptible granular fill or by rock fill, and
- The water table is maintained at 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 native stiff silty clay deposit can be classified as a Type 2 soil above the water level. This silty clay and the underlying sand and silt below the water level are classified as Type 3 soils.

8.2 Foundations

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

8.3 Earth Excavation

In addition to SP 902S01, a NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles, boulders or rock pieces in the overburden, especially at the west pier location.

It is anticipated that earth excavation to expose bedrock will be relatively shallow at the west abutment, east pier and east abutment locations. At the west pier location, excavation for footing construction will extend through the existing silty clay and occasional sand and silt with boulder deposits for a combined depth of up to 5.7 m. It is anticipated that excavation for foundation construction may need to be carried out in conjunction with a temporary shoring system at this location.

Rock fill will have to be removed from the east pier location.

Consideration may be given to using a braced soldier pile and lagging wall as temporary shoring at the west pier. The soldier piles will need to be socketted into bedrock by pre-bored holes in order to develop the required fixity.

At both the west and east piers where it is required to maintain a reasonably dry excavation for footing construction, consideration may be given to using cofferdams. One possible type of cofferdam consists of an impervious earth dyke surrounding the perimeter of the footing. Sump pumping should also be used to maintain a reasonably dry excavation base. Design of such temporary works will be the responsibility of the Contractor who should retain a professional engineer experienced in cofferdam design.

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

For a temporary shoring wall, the lateral pressure diagram as shown in Figure D2 may be used for design using the parameter values shown below.

γ	=	20 kN/m ³
γ_w	=	10 kN/m ³
K_a	=	0.4 (silty clay)
h_w	=	0, assuming that there is no hydrostatic pressure build-up behind a presumably permeable soldier pile and lagging wall.
H	=	depth to base of excavation (rock surface) (m)

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

$$P_p = 6 c B L$$

where c = 1000 kPa (equivalent Mohr-Coulomb cohesion based on the Hock and Brown rock mass classification)

L = depth of wall in rock, (m)

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

8.4 Rock Excavation

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

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

Should blasting be proposed, the Contractor's blasting and monitoring plan should take into account nearby structures such as the hydro dam. The contract documents should alert the contractor to these structures and that no damage will be induced on any adjacent structure (such as the hydro dam and its facilities) due to any blasting scheme. The Contract Administrator should retain a blasting expert for review of the Contractor's blasting procedures prior to approving them.

9 GROUNDWATER CONTROL

At the two pier locations where certain portions of the footing bases are below the river level, perimeter dykes constructed of relatively impervious materials such as silty clay, or other types of cofferdam, may be required to maintain a reasonably dry excavation for footing construction. These measures should be used in conjunction with sump pumps to remove any accumulated water

from the footing base prior to placing concrete. Design of such temporary works will be the responsibility of the Contractor who should retain a professional engineer experienced in such designs.

At the two abutment locations, the shallow and relatively impervious native deposit of silty clay should not yield significant amounts of seepage water in the short term. At locations where the more pervious zones such as the underlying sand and silt, and water-bearing seams within the silty clay are exposed in an excavation, water seepage will occur into the excavation.

The design of foundations bearing on bedrock will not be influenced by the water but the Contractor must make provision to control the groundwater seepage by using sump pumps to remove any accumulation of water from the footing base prior to placing concrete or compacting granular fill.

It is recommended that an NSSP for temporary dewatering requirements be included in the contract documents.

10 APPROACH EMBANKMENTS

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

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

10.1 Stability

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

The approach embankments for this structure will essentially be constructed on a veneer of stiff silty clay overlying bedrock or directly on bedrock. After the topsoil, organics and a thin veneer of native soils are removed to expose the rock subgrade, the required embankment will be in the order of 18 m in height at the east abutment and in the order of 8 m to 10 m in height at the west abutment.

At the east approach, with the exception of the granular fill core for pile installation purposes, the remainder of the embankment may consist of rockfill. The slope of the

granular core may be formed not steeper than 1.5H : 1V. It is recommended that rockfill embankments be formed with an outer slope inclination of 1.25H : 1V and a mid-height berm of 7 m in width. Alternatively, the rockfill embankments may have a slope inclination of 1.5H : 1V and a mid-height berm of 5 m in width. Stability analyses were carried out for the above slope configurations and yielded Factors of Safety (F.S.) in the order of 1.3 for long term (drained) conditions, and in the order of 1.5 for short term (undrained) conditions. Figures G1 to G4 present selected stability analyses results for the east approach.

At the west approach, fill will be placed on the exposed bedrock after the topsoil or a veneer of surficial native soil is stripped. The new fill may consist of rock fill or earth fill. It is recommended that rockfill embankments be formed with a slope inclination of 1.25H : 1V. Alternatively, it is recommended that earth fill with a slope inclination of 2H : 1V and a 2 m wide mid-height berm be used. Stability analyses were carried out for the above slope configurations and yielded Factors of Safety (F.S.) in the order of 1.3 to 1.4 for long term (drained) conditions. Figure G5 shows stability analysis results for the drained condition. Undrained conditions do not apply at the west approach.

The berms recommended above for the forward slopes serve the dual purpose of maintaining a minimum F.S. of 1.3 and satisfying surficial stability requirements. Beyond the forward slope areas, the 2 m wide berms are required for surficial stability only (see Section 10.3 below).

10.2 Settlement

At the east approach, some settlement will occur within the rockfill and the granular core. This settlement should be complete by the end of construction. Settlement due to recompression of the underlying, over-consolidated silty clay should be negligible.

At the west approach, the new fill may consist of rockfill or approved inorganic earth fill. Some settlement will occur within the new fill, but should be complete by the end of construction.

10.3 Construction

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

Rock fill embankments at the east abutment are higher than 10 m. In order to achieve a minimum F.S. of 1.3 against rotational type failures, berms should be incorporated at mid-height and should be 5 m or 7 m wide (depending on slope angles recommended previously) and extend for the length where the granular core is present. Beyond the zone of the granular core, the required berm width is 2 m and the berm should extend for the length through which the embankment height exceeds 10 m. Where earth fill embankments are higher than 8 m, berms should be incorporated at mid-height and 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 embankment, and 8 m high earth embankment, are in place to address surficial stability and to provide access for post construction maintenance.

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

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

11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used at this site. A conventional concrete abutment will be required for the contemplated design, but RSS could be used for wing walls and other retaining structures that might be required. Given the settlement due to embankment loading that is anticipated to be complete by the end of construction, the risk of using RSS walls at this site is considered low to medium.

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 the compacted granular core (east abutment) or exposed bedrock (west abutment). Where applicable, the native soil and the granular core under the RSS foundation should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at $\pm 2\%$ of its optimum moisture content. The engineered fill 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 above approximate Elevation 85m.
- 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 exposed bedrock or on compacted granular core. All 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 at ULS of 3,000 kPa for walls founded directly on limestone bedrock between Elevations 87.4 m and 88.2 m at the west abutment (SLS is not applicable for foundations on rock).
- Factored geotechnical resistance at ULS of 300 kPa, and geotechnical resistance of 200 kPa at SLS at approximate Elevation 93 m on the compacted granular core within the rockfill at the east abutment. If the core is composed of Granular A materials compacted to OPSS 501 Method A requirements, then values of 900 kPa for ULS and 350 kPa for SLS conditions may be used.
- Ultimate coefficient of friction between RSS mass and granular core is 0.55.
- Ultimate coefficient of friction between RSS mass and sound bedrock is 0.6.

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 for wing walls at the abutments.

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

East Abutment

- Engineered fill core – granular materials or SSM compacted to 100% SPMDD at $\pm 2\%$ optimum moisture content, with a slope of 1.5H : 1V (angle of internal friction, ϕ , of 30° , cohesion of 0, and unit weight, γ , of 20 kN/m^3).

- Rock fill outer shell – outer slope of 1.25H : 1V with mid-height berm of 7.5 m wide (angle of internal friction, ϕ , of 42° , cohesion of 0, and unit weight, γ , of 19 kN/m^3).
- Groundwater level at bedrock surface.
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 50% of the height, founded on silty clay overlying shallow bedrock.

Results of the analyses yield Factors of Safety in the order of 1.3 to 1.4 which indicate that global stability can be maintained for the assumed RSS configuration.

At the west abutment, global stability for an RSS wall founded directly on bedrock is not anticipated to be a design issue at this site.

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

11.3 Internal Stability

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

11.4 Settlement

Settlement will be negligible for RSS walls founded directly on bedrock at the west abutment. At the east abutment, settlement of RSS walls founded on native, very stiff to stiff over-consolidated silty clay overlying shallow bedrock are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

12 BACKFILL TO ABUTMENTS

In the cases where the approach embankment consists of rock fill, the backfill to the abutment wall should consist of OPSS Granular “B” Type II.

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

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

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

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

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

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

13 EARTH PRESSURES

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 below)

γ = unit weight of retained soil (see below)

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

q = value of any surcharge (kPa)

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

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical unfactored values are shown in the following table.

Wall Conditions	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 $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	.28*
At rest (Restrained Wall)	0.43	-	0.47	-	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 0.2

The subsurface at the two abutments and the east pier consist of topsoil and/or thin veneer of very stiff silty clay overlying very shallow bedrock within 1 m depth. The Soil Profile Type at these locations is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC is, associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1.0.

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

Clause C4.6.4 of the CHBDC suggests that the value of k_h used in calculating the earth pressure coefficients for yielding structures is equivalent to 0.5 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 all abutments and piers are to be founded directly on bedrock by means of spreads footings or driven piles, there is no potential for liquefaction under the foundation.

The approach embankments will be founded directly on bedrock or on very stiff silty clay overlying very shallow bedrock, and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur in the approach embankments but this is expected to be minor in nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

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

ϕ = angle of internal friction of backfill

δ = angle of internal friction between the wall and the backfill

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

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as Percentage of Wall Height	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17.5^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
		Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})	40%	0.33	0.70	0.37	0.90*	0.26	0.40
Passive (K_{PE})	33%	3.5	-	3.0	-	4.8	-
At Rest (K_{OE})**	45%	0.67	-	0.72	-	0.58	

* Slope may undergo movement for short durations during seismic activities

** After Woods

15 CONSTRUCTION CONCERNS

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

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

- presence of potentially unstable rock wedges and adverse joint systems at the pier locations, in particular the west pier; rock slope stability/rock mechanics specialist(s) must be retained by the Contract Administrator to assess exposed rock faces and rock subgrade in the vicinities of the piers and other critical locations; the rock specialist should design and implement rock stabilization measures as required.
- maintaining stability at all stages of construction at the approach embankments.
- cofferdam design and construction to allow pier(s) to be constructed in the dry.
- disturbance of the bedrock under the foundations due to excavation and other procedures.
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation, in particular at the west pier.
- boulders and cobbles may be encountered during construction.



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Madawaska River Bridge
Point Load Test Results

TABLE 1
Madawaska River Bridge
Point Load Test Results

Depth			Is50	UCS (MPa)
feet	Inches	m		
MRB-20				
2	8	0.81	4.78	114.82
4	3	1.30	4.78	114.82
5	3	1.60	5.27	126.40
6	5	1.96	4.92	117.98
8	2	2.49	2.50	60.04
9	3	2.82	4.39	105.34
10	1	3.07	4.26	102.18
11	0	3.35	4.61	110.60

Total Rock Core			
Average	Minimum	Maximum	
107	60	126	MPa
Run # Average			
1	118.68		
2	96.38		
3	110.60		

Depth			Is50	UCS (MPa)
feet	Inches	m		
MRB-14				
0	8.5	0.22	0.26	6.32
1	4	0.41	3.12	74.79
2	0	0.61	4.61	110.60
2	11	0.89	3.16	75.84
4	1	1.24	4.04	96.91
5	10	1.78	4.48	107.44
6	11	2.11	4.61	110.60
8	1	2.46	4.43	106.39
9	3	2.82	3.29	79.00
9	11	3.02	5.40	129.56

Total Rock Core			
Average	Minimum	Maximum	
90	6	130	MPa
Run # Average			
1	72.89		
2	106.60		

Depth		m	Is50	UCS (MPa)
feet	Inches			
MRB-19				
0	11	0.28	0.53	12.64
2	7	0.79	2.19	52.67
3	4	1.02	4.30	103.23
4	2	1.27	3.99	95.86
5	6	1.68	2.94	70.58
6	9	2.06	3.03	72.68
8	1	2.46	4.70	112.71
9	2	2.79	3.95	94.80
9	9	2.97	4.21	101.12

Total Rock Core			
Average	Minimum	Maximum	
80	13	113	MPa
Run # Average			
1	66.10		
2	90.38		

Madawaska River Bridge
Point Load Test Results

TABLE 1 (continued)
Madawaska River Bridge
Point Load Test Results

Depth			Is50	UCS (MPa)				
feet	Inches	m						
MRB-1								
1	10	0.56	3.95	94.80				
2	9	0.84	4.48	107.44				
3	11	1.19	4.17	100.07				
5	4	1.63	4.04	96.91				
6	6	1.98	3.25	77.95				
7	10	2.39	3.73	89.54				
8	10	2.69	4.43	106.39				
9	8	2.95	4.52	108.50				
10	4	3.15	3.34	80.06				
11	1	3.38	4.43	106.39				
					Total Rock Core			
					Average	Minimum	Maximum	MPa
					97	78	108	
					Run #	Average		
					1	100.77		
					2	95.86		
					3	93.22		
Depth			Is50	UCS (MPa)				
feet	Inches	m						
MRB-7								
0	10	0.25	1.19	28.44				
1	9	0.53	0.92	22.12				
2	7	0.79	3.64	87.43				
3	4	1.02	0.75	17.91				
4	4	1.32	3.73	89.54				
5	7	1.70	0.92	22.12				
6	10	2.08	4.04	96.91				
7	10	2.39	3.29	79.00				
9	0	2.74	3.99	95.86				
9	11	3.02	3.25	77.95				
					Total Rock Core			
					Average	Minimum	Maximum	MPa
					62	18	97	
					Run #	Average		
					1	49.09		
					2	74.37		
Depth			Is50	UCS (MPa)				
feet	Inches	m						
MRB-8								
19	6	5.94	2.02	48.45				
20	6	6.25	4.43	106.39				
21	6	6.55	4.65	111.66				
22	7	6.88	1.84	44.24				
24	0	7.32	3.60	86.38				
25	5	7.75	3.82	91.64				
26	5	8.05	3.47	83.22				
27	0	8.23	3.51	84.27				
28	4	8.64	0.88	21.07				
					Total Rock Core			
					Average	Minimum	Maximum	MPa
					75	21	112	
					Run #	Average		
					2	77.42		
					3	83.48		
					4	62.85		

Madawaska River Bridge
Point Load Test Results

TABLE 1 (continued)
Madawaska River Bridge
Point Load Test Results

Depth			Is50	UCS (MPa)					
feet	Inches	m				Average	Minimum	Maximum	
MRB-13					}	Total Rock Core			
1	6	0.46	4.48	107.44		115	79	216	MPa
2	10	0.86	4.74	113.76					
3	11	1.19	3.60	86.38					
5	5	1.65	4.83	115.87					
6	8	2.03	3.29	79.00		Run #	Average		
7	9	2.36	3.82	91.64		1	102.53		
8	6	2.59	9.00	215.94		2	122.61		
9	1	2.77	4.61	110.60					

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.
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No MRB-1

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 185.2 E 316 485.8 (Madawaska River Bridge, West Abutment-1) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
88.0 0.0	TOPSOIL (425mm)						88							GR SA SI CL
87.6 0.4	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, very thinly bedded, grey with dark grey and white horizontal banding, strong Subvertical joints at 0.7m, 1.0m, 1.75m, 3.3m Rough joint surfaces		1	RUN	2 1 0 2		87							RUN 1# TCR=98%, SCR=98%, RQD=91%, UCS=100MPa
			2	RUN	1 1 0 1 0		86							RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=96MPa
84.5			3	RUN	1		85							RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=93MPa
3.5	END OF BOREHOLE AT 3.53m.													

RECORD OF BOREHOLE No MRB-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 184.9 E 316 491.8 (Madawaska River Bridge, West Abutment-2) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Manual Method (Hand Shovel and Visual Inspection) COMPILED BY SS
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	w _p	w	w _L		
88.2																	
0.0																	
87.9	TOPSOIL (300mm)																
0.3	END OF BOREHOLE AT 0.3m. SURFACE OF PROBABLE BEDROCK OBSERVED AT 0.3m.																

METRIC

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

ONTMT4 7450MRB.GPJ 22/04/04

+ 3, × 3: Numbers refer to Sensitivity

METRIC

DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

+ 3, x 3 Numbers refer to Sensitivity

RECORD OF BOREHOLE No MRB-5 (Test Pit) 1 OF 1 METRIC

G.W.P. 647-92-00 LOCATION N 5 031 173.5 E 316 469.8 (Madawaska River Bridge, West Abutment-5) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Test Pit 3 COMPILED BY SS
 DATUM Geodetic DATE 13.08.03 - 13.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
86.4 0.0	TOPSOIL with some clay pockets													
86.0														
0.4	PROBABLE BEDROCK END OF BOREHOLE AT 0.38m.						86							

RECORD OF BOREHOLE No MRB-6

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 172.2 E 316 484.8 (Madawaska River Bridge, West Abutment-6) ORIGINATED BY SL
HWY HWY 17 BOREHOLE TYPE Visual Inspection COMPILED BY SS
DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			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	20 40 60	W _p W W _L						
87.4 0.0	WEATHERED BEDROCK AT GROUND SURFACE.						87										

RECORD OF BOREHOLE No MRB-7

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 171.8 E 316 490.8 (Madawaska River Bridge, West Abutment-7) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE							W p W W L		
						● QUICK TRIAXIAL	×	LAB VANE											
88.3 86.8	TOPSOIL (100mm)				FI	20	40	60	80	100	20	40	60						
0.1	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly bedded, grey with dark grey and white horizontal and subvertical banding, moderately strong to strong		1	RUN	1										RUN 1# TCR=93%, SCR=93%, RQD=93%, UCS=49MPa				
					1														
					1														
					1														
			2	RUN	2												RUN 2# TCR=98%, SCR=98%, RQD=97%, UCS=74MPa		
					1														
					0														
					1														
85.2			2		2														
3.2	END OF BOREHOLE AT 3.15m.																		

RECORD OF BOREHOLE No MRB-8

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 177.7 E 316 572.0 (Madawaska River Bridge, West Pier-1) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
81.6 84.5 0.1	TOPSOIL (100mm) Black Silty CLAY, some sand, trace rootlets Stiff Brown Moist (Cl) occasional topsoil lenses Some sand seams Firm		1	SS	11		81							0 15 67 18
			2	SS	19		80							
			3	SS	10		79							
			4	SS	10		78							
			5	SS	5		77							
			6	SS	4		76							
77.3	AUGER REFUSAL AT 4.34m. Boulder at 4.3m to 4.9m Till-like soil from 4.9m to 5.7m in water return. inferred sand and silt, some gravel, trace clay, occasional cobbles		1	RUN			75							RUN 1# TCR=24%, SCR=24%, RQD=24%
75.9					FI		74							
5.7	CRYSTALLINE LIMESTONE (BEDROCK) Slightly weathered, moderately weathered at joints, very thin to thinly bedded, grey with dark grey and white horizontal and subvertical banding, moderately strong to strong. Subvertical joint at 5.7m, 8.1m, 8.3m Rough joint surface		2	RUN	3 1 1 0 1 2 1 2 5		73							
			3	RUN										
			4	RUN										RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=77MPa RUN 3# TCR=100%, SCR=98%, RQD=98%, UCS=83MPa RUN 4# TCR=97%, SCR=87%, RQD=87%, UCS=62MPa
72.9	END OF BOREHOLE AT 8.71m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. DATE ELEVATION (m) 22/10/03 78.3 04/02/04 78.1 11/03/04 78.6													
8.7														

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MRB-9

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 177.3 E 316 577.0 (Madawaska River Bridge, West Pier-2) ORIGINATED BY SL
HWY HWY 17 BOREHOLE TYPE Manual Method COMPILED BY SS
DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
80.0														
79.9	TOPSOIL (150mm)		1	SS	60*									
0.2	SAND and SILT, some clay, some rootlets		2	SS	60*									
79.4	Brown													0 53 30 17
0.6	END OF BOREHOLE AT 0.61m.													
	Borehole advanced manually using a 27.2kg hammer dropping a vertical distance of 0.76m. * Blowcounts reported Refer to record of borehole (MRB-9A) for further information													

RECORD OF BOREHOLE No MRB-9A (Test Pit) 1 OF 1 METRIC

G.W.P. 647-92-00 LOCATION N 5 031 177.3 E 316 577.0 (Madawaska River Bridge, West Pier) ORIGINATED BY SL
HWY HWY 17 BOREHOLE TYPE Test Pit 2 COMPILED BY SS
DATUM Geodetic DATE 13.08.03 - 13.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
80.0							80							
79.8	TOPSOIL (175mm)						79							
0.2	Black Silty CLAY Firm to Stiff (inferred) Brown some sand seams Becoming grey						78							
76.8							77							
3.2	PROBABLE BEDROCK END OF BOREHOLE AT 3.2m.													

RECORD OF BOREHOLE No MRB-10

1 OF 1



METRIC

G.W.P. 647-92-00 LOCATION N 5 031 172.4 E 316 571.1 (Madawaska River Bridge, West Pier-3) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
81.9 80.6 0.1	TOPSOIL (75mm) Black Silty CLAY, trace rootlets, trace sand Stiff to Very Stiff Brown (CI-CL)		1	SS	11									
			2	SS	13									
			3	SS	17									
79.8														
2.1	SAND and SILT, some gravel, trace clay, occasional cobbles, occasional limestone pieces. Very Dense Brown		4	SS	75									16 38 36 10
79.0														
2.9	END OF BOREHOLE AT 2.9m. AUGER REFUSAL AT 2.9m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY ON COMPLETION.													

RECORD OF BOREHOLE No MRB-11 (Test Pit) 1 OF 1 METRIC

G.W.P. 647-92-00 LOCATION N 5 031 171.3 E 316 576.5 (Madawaska River Bridge, West Pier-4) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Test Pit 1 COMPILED BY SS
 DATUM Geodetic DATE 13.08.03 - 13.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
79.7								20	40	60	80	100	w _p	w	w _L	kN/m ³	GR SA SI CL
79.0																	
79.5	TOPSOIL (175mm)																
0.2	Silty CLAY, some sand seams Brown Stiff to Very Stiff (inferred)						79										
78.4																	
1.3	PROBABLE BEDROCK END OF TEST PIT AT 1.27m.																

RECORD OF BOREHOLE No MRB-12

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 165.7 E 316 571.0 (Madawaska River Bridge, West Pier-5) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
82.1														
82.0	TOPSOIL (75mm)													
0.1	Silty CLAY, some sand, trace rootlets Firm		1a	SS	7		82							0 19 66 15
81.5	Brown		1b	SS	507									
0.6	(CL-CI) Sampler refusal at 0.53m END OF BOREHOLE AT 0.58m. AUGER REFUSAL AT 0.58m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY ON COMPLETION.				.076									

+³, x³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MRB-13

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 165.3 E 316 576.0 (Madawaska River Bridge, West Pier-6) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
81.8					FI									
80.8	TOPSOIL (175mm)													
0.2	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with dark grey and white horizontal and subvertical banding, strong to very strong Subvertical joint at 0.2m, 2.2m, 2.5m, 2.7m Rough joint surface		1	RUN	2		81							RUN 1# TCR=88%, SCR=86%, RQD=76%, UCS=102MPa
					1									
					2									
					1									
					0									
					0		80							RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=122MPa
			2	RUN	2									
					1									
					2									
					3		79							
78.6														
3.2	END OF BOREHOLE AT 3.2m.													

RECORD OF BOREHOLE No MRB-14

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 167.1 E 316 701.6 (Madawaska River Bridge, East Pier) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)			
								20	40	60			80	100	20	40
79.1 0.0	CRYSTALLINE LIMESTONE (BEDROCK) Moderately weathered to 0.5m, fresh to slightly weathered below 0.5m, very thinly to thinly bedded, grey with dark grey and white subvertical banding, moderately weak to 0.4m, strong to very strong below 0.4m Numerous fractures from surface to 0.4m Rough joint surface		1	RUN	FI								RUN 1# TCR=91%, SCR=82%, RQD=56%, UCS=72MPa			
0					6									0	2	1
			2	RUN		1	0	1	1	2				RUN 2# TCR=100%, SCR=100%, RQD=92%, UCS=106MPa		
76.1																
3.1	END OF BOREHOLE AT 3.05m.															

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

METRIC

DATUM	Geodetic	DATE	07.08.03 - 07.08.03	CHECKED BY	SP
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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	w _p	w		
79.8 0.0 79.6 0.2	TOPSOIL (225mm) Black END OF BOREHOLE AT 0.23m. SURFACE OF PROBABLE BEDROCK OBSERVED AT 0.2m.						79						

RECORD OF BOREHOLE No MRB-16

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 161.2 E 316 701.0 (Madawaska River Bridge, East Pier-3) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Visual Inspection COMPILED BY SS
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _P	W	W _L		
79.1 0.0	PROBABLE BEDROCK OBSERVED AT GROUND SURFACE.						79										

ONTMT4 7450MRB GPJ 22/04/04

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L				
79.8 78.8 0.1	TOPSOIL (50mm) Black END OF BOREHOLE AT 0.05m. SURFACE OF PROBABLE BEDROCK OBSERVED AT 0.05m.						SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100 20 40 60						
							79							

RECORD OF BOREHOLE No MRB-18

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 155 2 E 316 700.6 (Madawaska River Bridge, East Pier-5) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Visual Inspection COMPILED BY SS
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
79.0 0.0	SURFICIAL ROCK FILL OBSERVED OVER PROBABLE BEDROCK BELOW WATER LEVEL OF RIVER.					79							GR SA SI CL

ONTMT4 7450MRB.GPJ 23/04/04

RECORD OF BOREHOLE No MRB-19

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 154.8 E 316 705.6 (Madawaska River Bridge, East Pier-6) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 06.08.03 - 07.08.03 CHECKED BY SP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
78.9													
78.8	TOPSOIL (100mm)												
0.1	CRYSTALLINE LIMESTONE (BEDROCK) Fresh to slightly weathered, very thinly to thinly bedded, grey with dark grey and white subvertical banding Subvertical joint at 0.5m, 1.4m, vertical joint at 1.8m Rough joint surface loss of water due to mechanical failure at 1.72m		1	RUN		78							RUN 1# TCR=91%, SCR=89%, RQD=70%, UCS=66MPa
			2	RUN		77							RUN 2# TCR=93%, SCR=88%, RQD=72%, UCS=90MPa
75.8						76							
3.2	END OF BOREHOLE AT 3.15m.												

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MRB-21

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 159.0 E 316 807.8 (Madawaska River Bridge, East Abutment-3) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 06.08.03 - 06.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
86.6														
86.6	TOPSOIL (125mm)													
0.1	SAND and SILT, trace gravel, trace clay, trace rootlets Dense Brown		1	SS	38		86							8 18 64 9
	frequent cobbles and/ or boulders below 0.9m.		2	SS	50/ .127									
85.0														
1.6	END OF BOREHOLE AT 1.6m. AUGER REFUSAL AT 1.6m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.		3	SS	50/ .076		85							

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MRB-22

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 147.2 E 316 791.8 (Madawaska River Bridge, East Abutment-7) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 06.08.03 - 06.08.03 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
83.6	TOPSOIL (150mm)													
83.5	Black		1	SS	23									
0.1	Silty CLAY, trace gravel and rootlets Very Stiff Brown		2	SS	20/									
82.6	Occasional limestone pieces at 0.8m				101									
1.0	END OF BOREHOLE AT 1.04m. AUGER REFUSAL AT 1.04m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.													

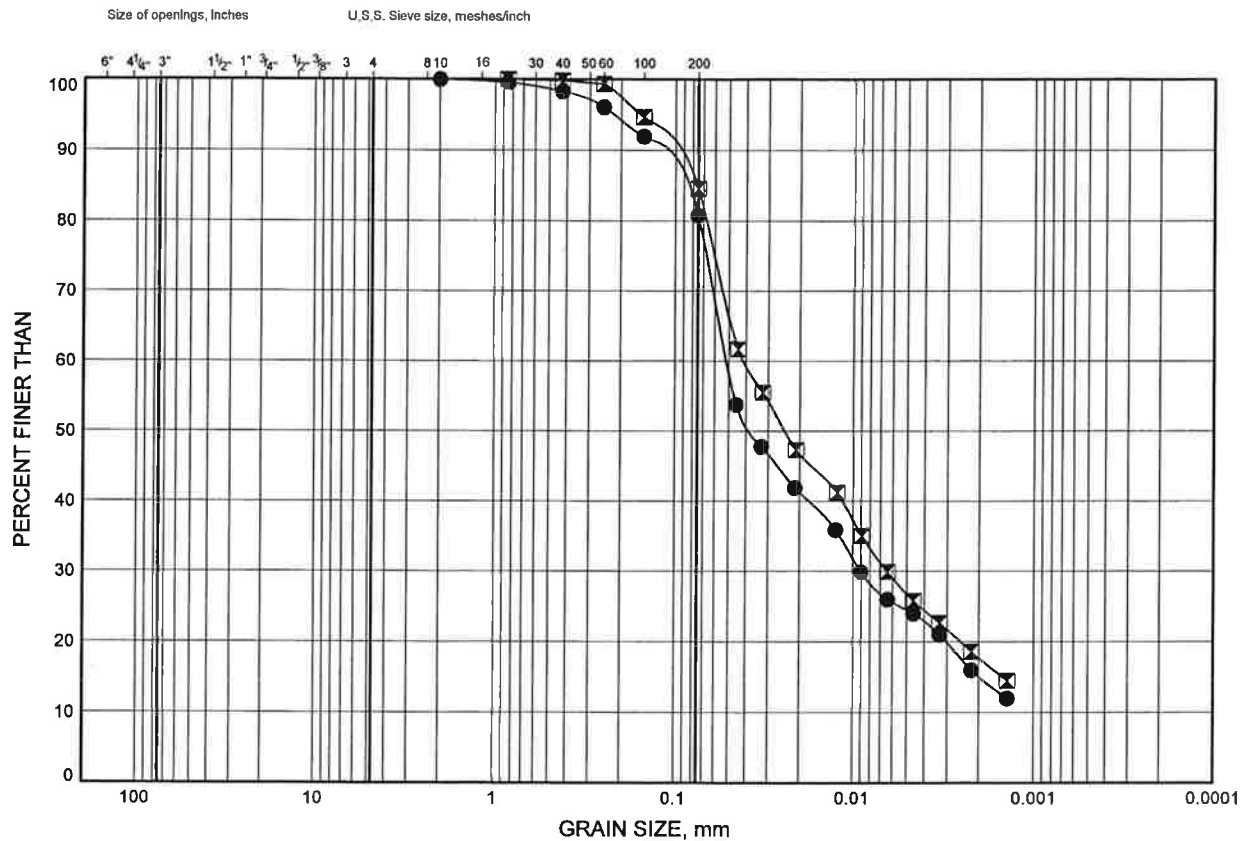
Appendix B

Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY



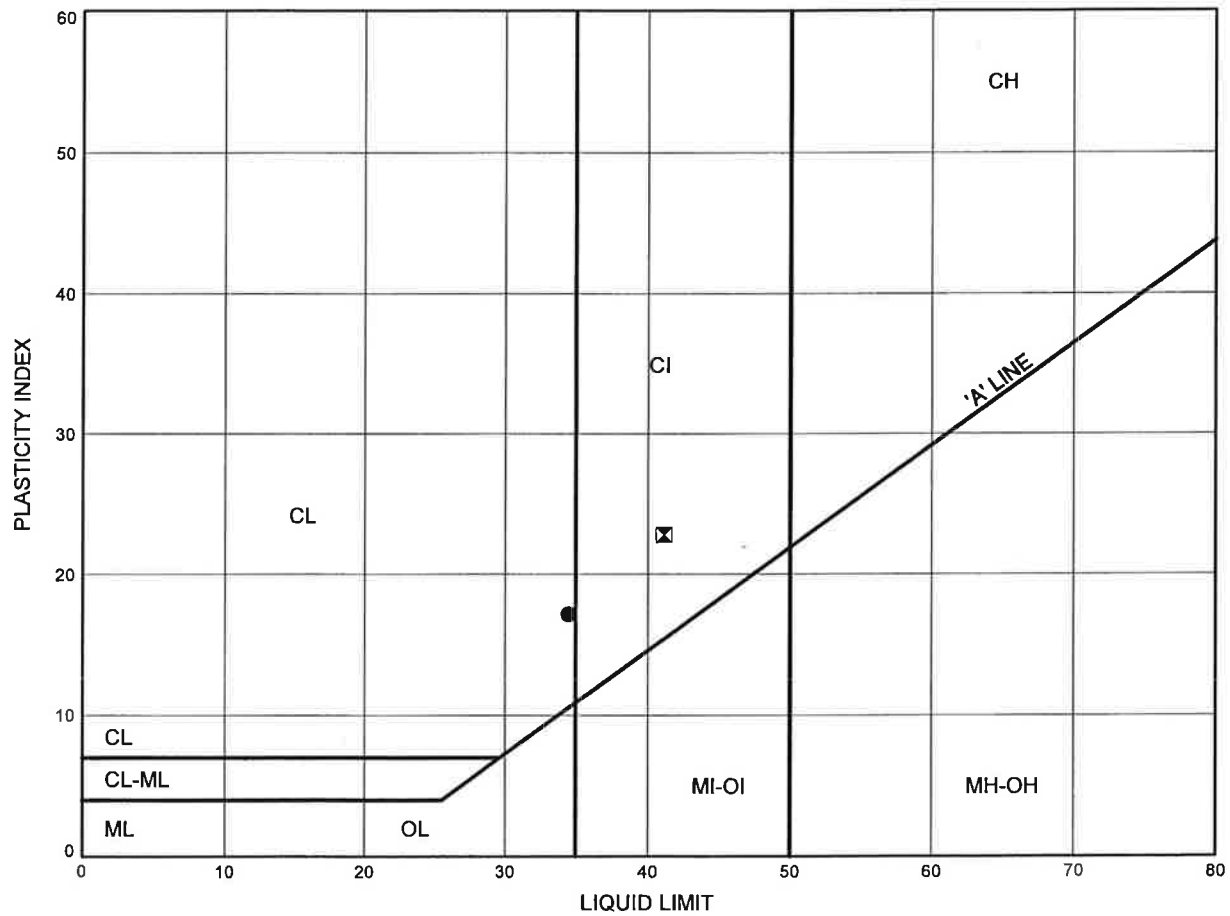
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	MRB-12	0.23	81.87
x	MRB-8	0.99	80.61

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

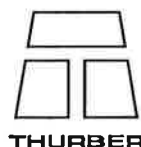
FIGURE B2

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	MRB-10	0.99	87.01
⊠	MRB-8	3.28	78.32

Date April 2004
Project 647-92-00

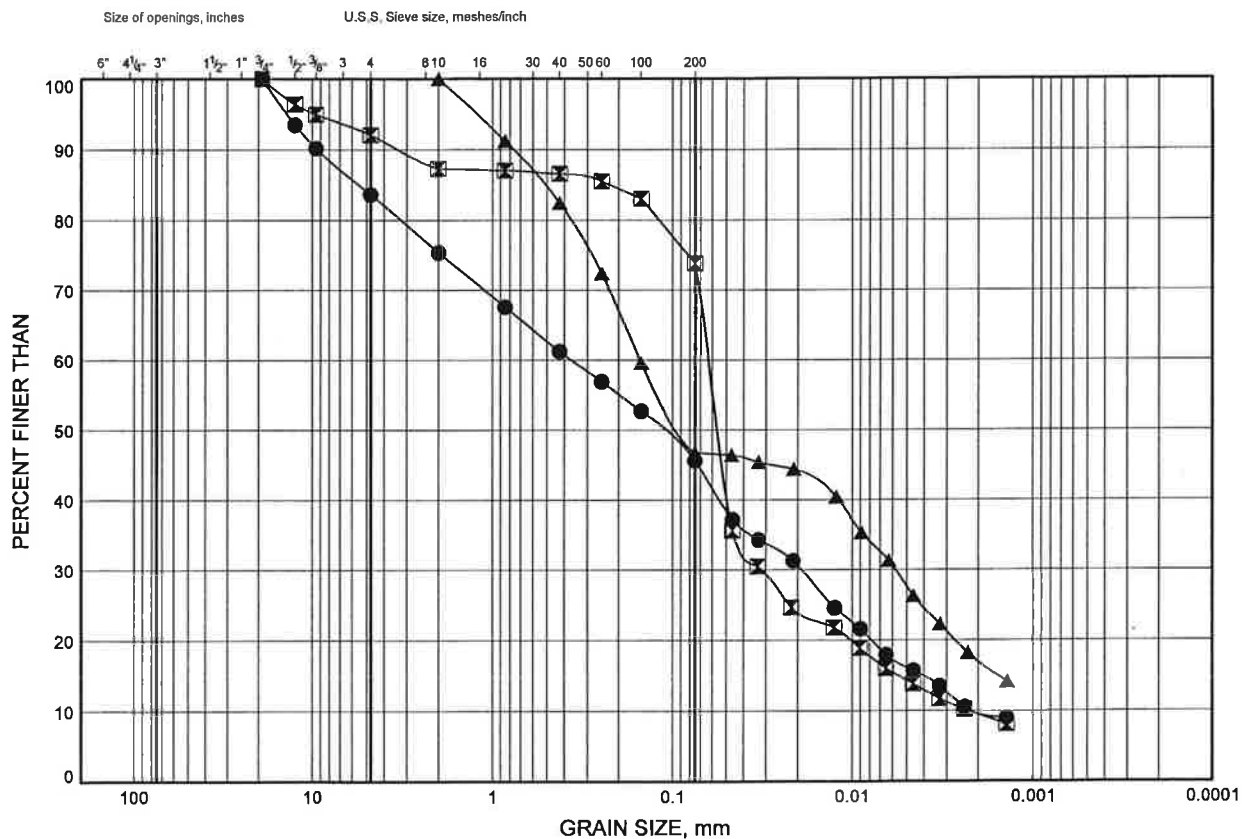


Prep'd SS
Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B3

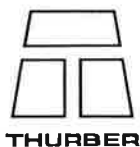
SAND AND SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	MRB-10	2.51	85.49
☒	MRB-21	0.30	86.30
▲	MRB-9	0.45	79.55

Date April 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
West Abutment	<p>Advantages: Required if an integral abutment design is pursued.</p> <p>Disadvantages: i. Shallow bedrock surface rendering the use of driven piles impractical. ii. Piles may have to be socketted into rock for integral abutment design.</p>	<p>Advantages: High values of geotechnical resistance are available on the bedrock.</p> <p>Disadvantages: i. Stepped footing may be required. ii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface.</p>	<p>Advantages: i. Possibility of shortening the abutment height.</p> <p>Disadvantages: i. Insufficient height between top of bedrock and proposed highway grade. ii. Lower geotechnical resistance than bedrock.</p>	<p>Advantages: i. High values of geotechnical resistance are available on the bedrock.</p> <p>Disadvantages: i. Shallow bedrock surface rendering the use of augered caissons impractical, though technically possible. ii. Caissons will have to be socketted into bedrock.</p>

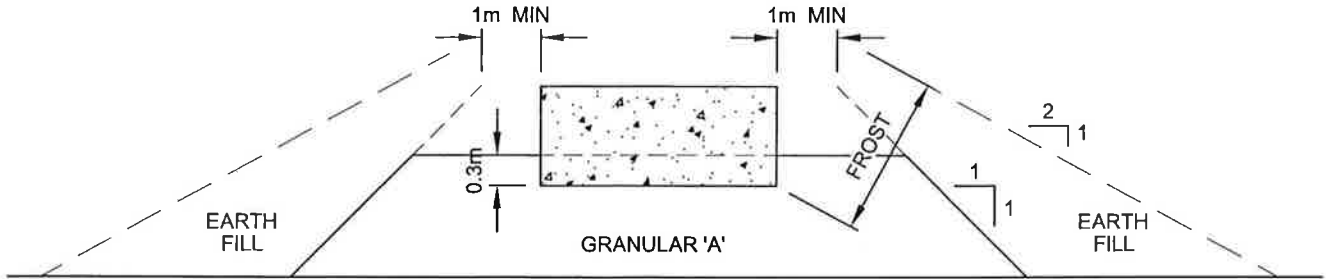
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT (Cont'd)

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Augered Caisson
West and East Piers	<p>Advantages: None identified.</p> <p>Disadvantages: i. Shallow bedrock surface, or outcrop at some locations, rendering the use of driven piles impractical and unnecessary.</p>	<p>Advantages: i. High values of geotechnical resistance are available on the bedrock.</p> <p>Disadvantages: i. Stepped footings may be required. ii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface.</p>	<p>Advantages: None identified.</p> <p>Disadvantages: i. Lower geotechnical resistance than bedrock. ii. Shallow bedrock surface, or outcrop at some locations, rendering the use of augered caissons impractical and unnecessary.</p>	<p>Advantages: None identified.</p> <p>Disadvantages: i. Shallow bedrock surface, or outcrop at some locations, rendering the use of augered caissons impractical and unnecessary.</p>
East Abutment	<p>Advantages: i. Conventional perched abutment design with piles driven to bedrock is feasible. ii. Required for an integral abutment design.</p> <p>Disadvantages: i. Engineered fill core required as a platform to accommodate pile installation equipment.</p>	<p>Advantages: i. High values of geotechnical resistance are available on the bedrock.</p> <p>Disadvantages: i. Due to the proposed height of approach fill (in the order of 18m), the required dimensions of the abutment wall would become impractical and uneconomical. ii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface.</p>	<p>Advantages: i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more practical footing dimensions.</p> <p>Disadvantages: i. Lower geotechnical resistance than bedrock. ii. May not be practical if differential settlement along the span (between east pier and east abutment) is too large. iii. Footing may have to be moved further back resulting in longer span.</p>	<p>Advantages: i. Conventional perched abutment design with augered caissons founded on bedrock is feasible.</p> <p>Disadvantages: i. Engineered fill core required as a platform to accommodate caisson installation equipment. ii. Caissons will have to be socketted into bedrock..</p>

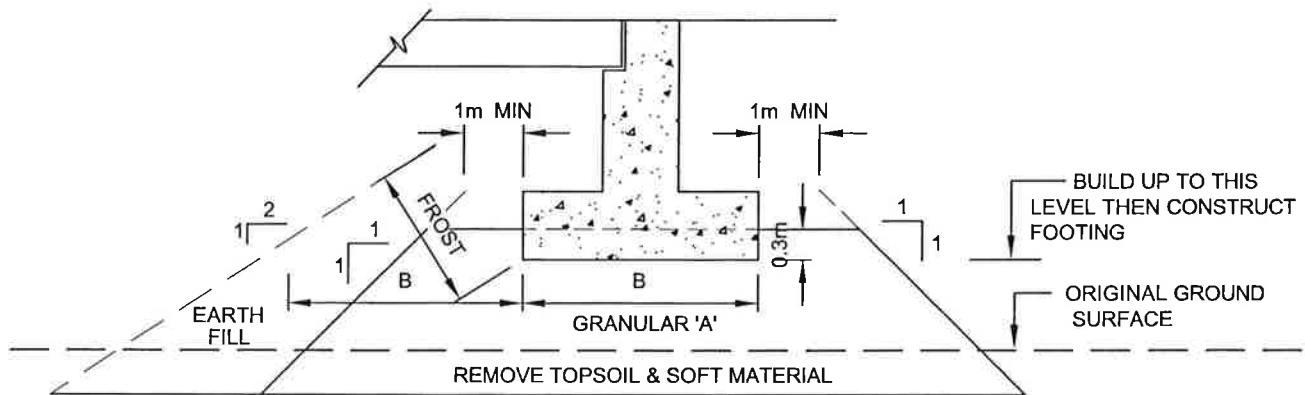
Madawaska River Bridge
Highway 17 Twinning, Arnprior to Renfrew

Appendix D

Figures



CROSS-SECTION



LONGITUDINAL SECTION

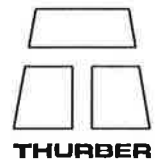
NOT TO SCALE

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.

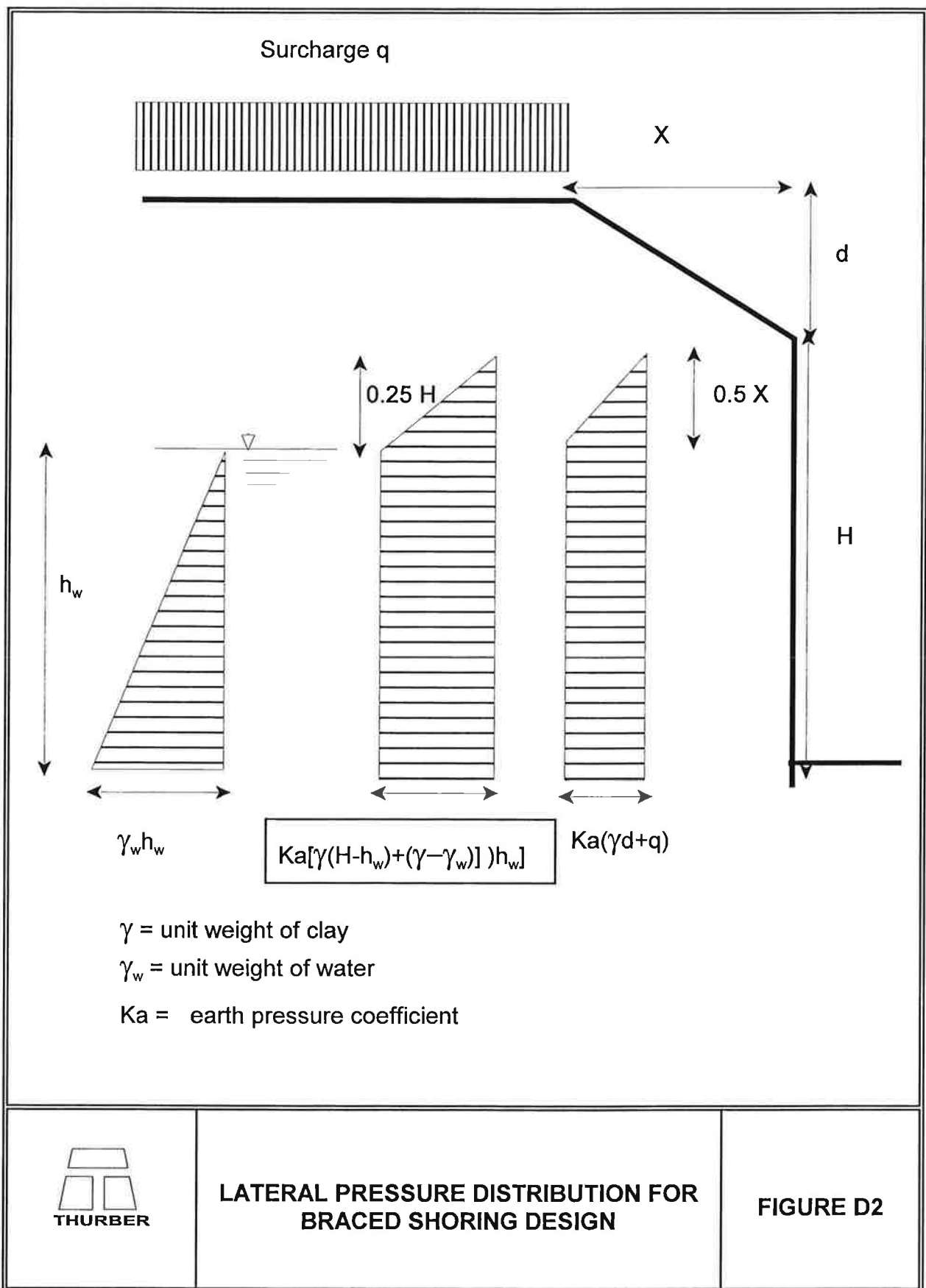
ENGINEER	SP
DRAWN	SS
DATE	April , 2004
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR A CORE



DWG. NO.

FIGURE D1



Madawaska River Bridge
Highway 17 Twinning, Arnprior to Renfrew

Appendix E

Special Provisions

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

Special Provision No. 902S01

September 2003

Excavation and Backfilling-Structures

902.02 REFERENCES

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

OPSS 510

902.03 DEFINITIONS

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

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

902.04 SUBMISSION AND DESIGN REQUIREMENTS

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

902.04.01 Site Survey

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

902.04.02 Working Drawings

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

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

902.04.03 Submission of Certificate of Conformance

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

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

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

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

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

902.05.04 Protection System

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

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

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

902.07.02 Excavation

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

902.07.02.01 General

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

902.07.02.02 Excavation for Foundation

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

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

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

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

902.07.02.03 Excavation for Backfill and Frost Tapers

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

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

902.07.02.04 Preservation of Channel

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

902.07.02.05 Removals

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

902.07.03 Unwatering Structure Excavation

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

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

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

902.07.04 Backfilling

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

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

Backfilling shall be according to OPSS 501.

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

902.09 Measurement for Payment

902.09.01 Structures

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

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

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

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

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

902.10 Basis of Payment

902.10.01 Excavation and Backfill

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

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

WARRANT: Always with these tender items.

AMENDMENT TO OPSS 120, AUGUST, 1994

Special Provision

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

Construction Special Provision for Rock Excavation Utilizing Blasting

120.01 SCOPE

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

120.02 REFERENCES

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

Canadian Standards Association:

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

Ministry of Transportation Publications:

Ontario Traffic Manual Book 7

Federal Government Publication:

Explosives Act (Canada)

Department of Fisheries and Oceans Publication:

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

120.03 DEFINITIONS

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

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

Blasting Consultant: means an Engineer, with a minimum of five (5) years experience related to blasting design, blasting operations and monitoring of rock excavation blasting. The Blasting Consultant shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

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

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

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

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

120.04 SUBMISSION AND DESIGN REQUIREMENTS

120.04.01 General

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

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

120.04.02 Blasting Design and Monitoring

120.04.02.01 Blasting Consultant

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

120.04.02.02 Blasting Design

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

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

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

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

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

120.04.02.03 Blasting Monitoring

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

The blasting monitoring equipment shall meet the following requirements:

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

120.04.03 Blasting Permits, Approvals and/or Agreements

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

120.04.04 Pre-Blast Survey

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

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

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

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

120.04.05 Trial Blasting

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

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

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

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

120.04.06 Post Blast Survey and Reporting

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

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

120.04.07 Certificate of Conformance

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

120.05 MATERIAL

120.05.01 Explosives

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

120.05.02 Handling and Storage

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

120.06 EQUIPMENT

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

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

120.07 CONSTRUCTION

120.07.01 General

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

120.07.02 Safety Precautions

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

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

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

120.07.03 Notice

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

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

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

120.07.04 Vibration Monitoring

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

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

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

Structures

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

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

Fresh Concrete

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

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

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

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

120.07.05 Utilities

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

120.07.06 Excessive Vibration Readings – Work Stoppage

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

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

AMENDMENT TO OPSS 206, DECEMBER 1993

Special Provision

November 25, 2002

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

206.01 SCOPE

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

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

206.04 SUBMISSION AND DESIGN REQUIREMENTS

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

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

206.06 EQUIPMENT

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

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

206.07 CONSTRUCTION

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

206.07.01.03 Compaction

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

206.07.01.03.01 Compaction of Earth Embankments

Compaction of earth materials shall conform to OPSS 501.

206.07.01.03.02 Compaction of Rock Embankments

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

206.07.05 Rock Excavation, Grading

206.07.05.01 General

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

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

206.07.08 Rock Embankments

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

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

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

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

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

Placement in layers and compaction is not required for rock to be placed under water. Rock placed underwater may be placed by end dumping. End dumping shall only be used to an elevation of 1.0 m above the water level after which rock embankments shall be constructed using the equipment and method specified in this special provision. The rock used for end dumping shall be deposited on the

surface of the embankment and pushed forward by blading or dozing over the edge of the embankment. The materials shall be well distributed to form a solid embankment constructed to full width as the work progresses, or as stage construction allows.

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

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

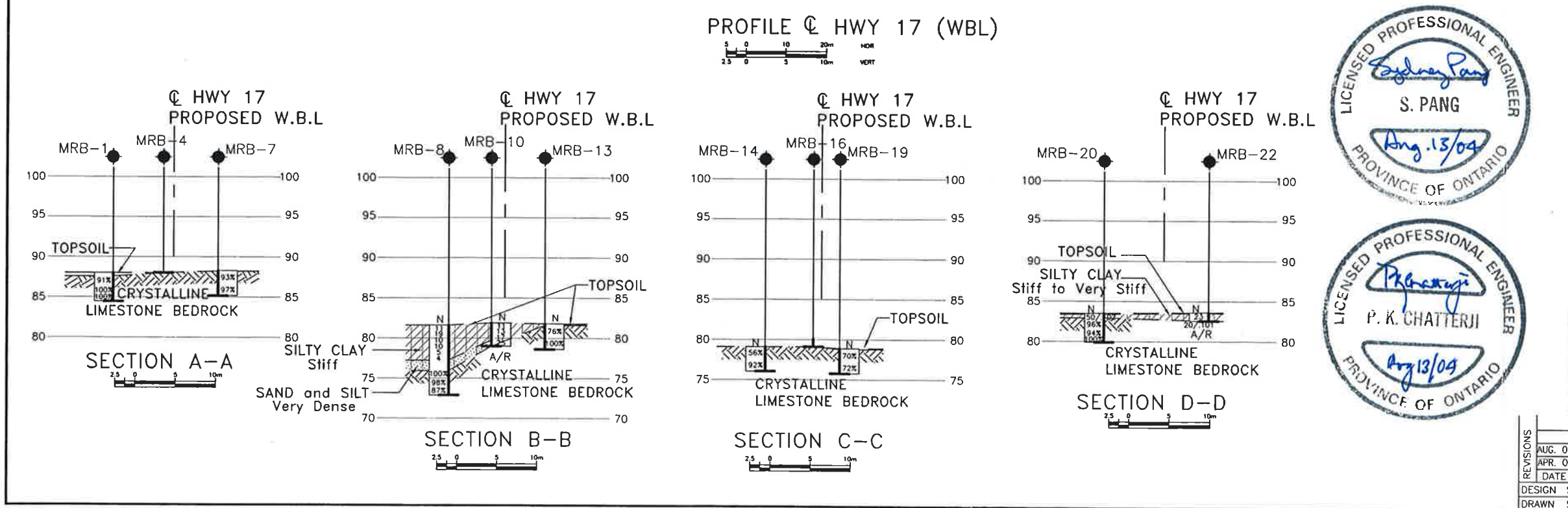
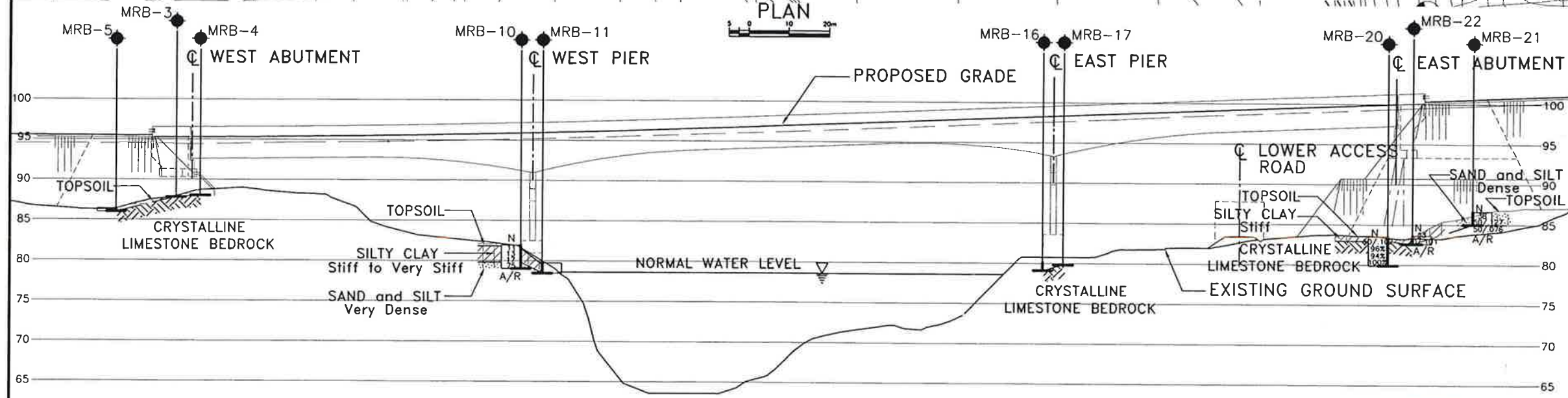
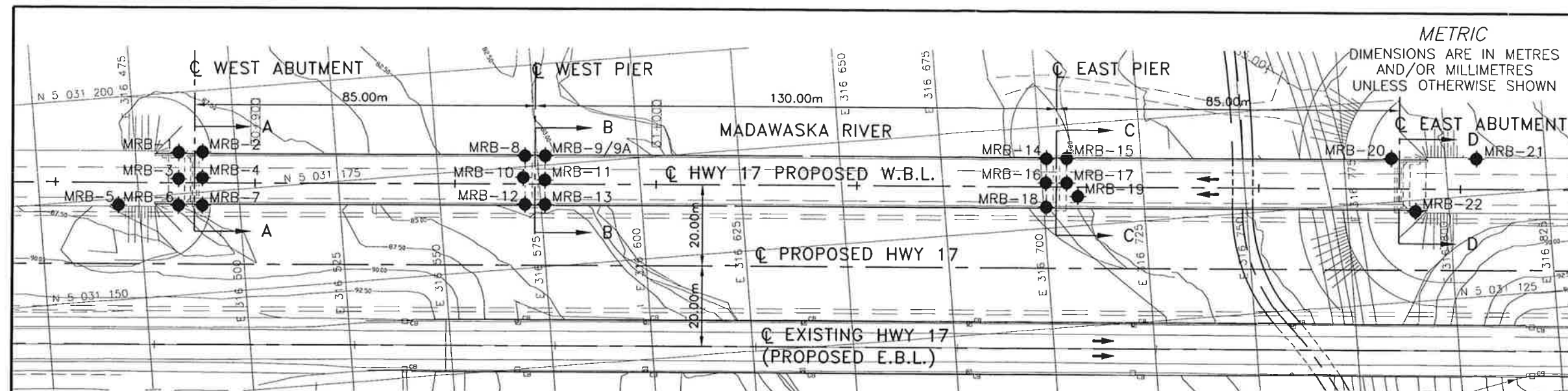
Care shall be taken to avoid large boulders and rock fragments protruding above the average embankment surface within a distance of 3 m beyond the edge of the shoulder for future roadside safety.

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

Madawaska River Bridge
Highway 17 Twinning, Arnprior to Renfrew

Appendix F

Drawings



HWY.17
WP NO. 647-92-00

HIGHWAY 17 TWINNING
MADAWASKA RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

THURBER ENGINEERING LTD.

engineers
architects
planners

SHEET

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

KEYPLAN

LEGEND

- Bore Hole
- ⊕ 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
MRB-1	88.0	5 031 185.2	316 485.8
MRB-2	88.2	5 031 184.9	316 491.8
MRB-3	87.8	5 031 178.8	316 485.3
MRB-4	88.0	5 031 178.5	316 491.3
MRB-5	86.4	5 031 173.5	316 469.8
MRB-6	87.4	5 031 172.2	316 484.8
MRB-7	88.3	5 031 171.8	316 490.8
MRB-8	81.6	5 031 177.7	316 572.0
MRB-9	80.0	5 031 177.3	316 577.0
MRB-9A	80.0	5 031 177.3	316 577.0
MRB-10	81.9	5 031 172.4	316 571.1
MRB-11	79.7	5 031 171.3	316 576.5
MRB-12	82.1	5 031 165.7	316 571.0
MRB-13	81.8	5 031 165.3	316 576.0
MRB-14	79.1	5 031 167.1	316 701.6
MRB-15	79.8	5 031 166.8	316 706.6
MRB-16	79.1	5 031 161.2	316 701.0
MRB-17	79.8	5 031 160.7	316 706.1
MRB-18	79.0	5 031 155.2	316 700.6
MRB-19	78.9	5 031 157.2	316 708.6
MRB-20	83.5	5 031 160.7	316 786.9
MRB-21	86.6	5 031 159.0	316 807.8
MRB-22	83.6	5 031 147.2	316 791.8

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

NO	DATE	BY	DESCRIPTION
1	AUG. 04	SP	FINAL
2	APR. 04	SP	ISSUED AS DRAFT FOR REVIEW

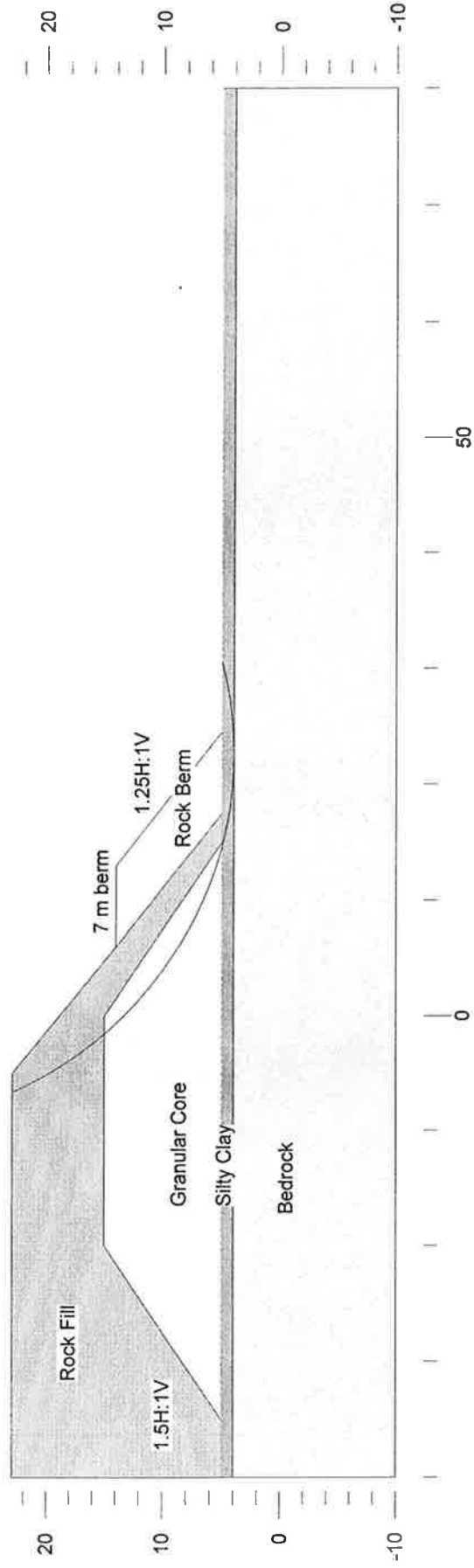
DESIGN	SP	CHK	PKC	CHBDC 2000	LOAD	DATE	AUG.2004
DRAWN	SS	CHK	SP	SITE 29-191/1	STRUCT	DWG.	

Madawaska River Bridge
Highway 17 Twinning, Arnprior to Renfrew

Appendix G

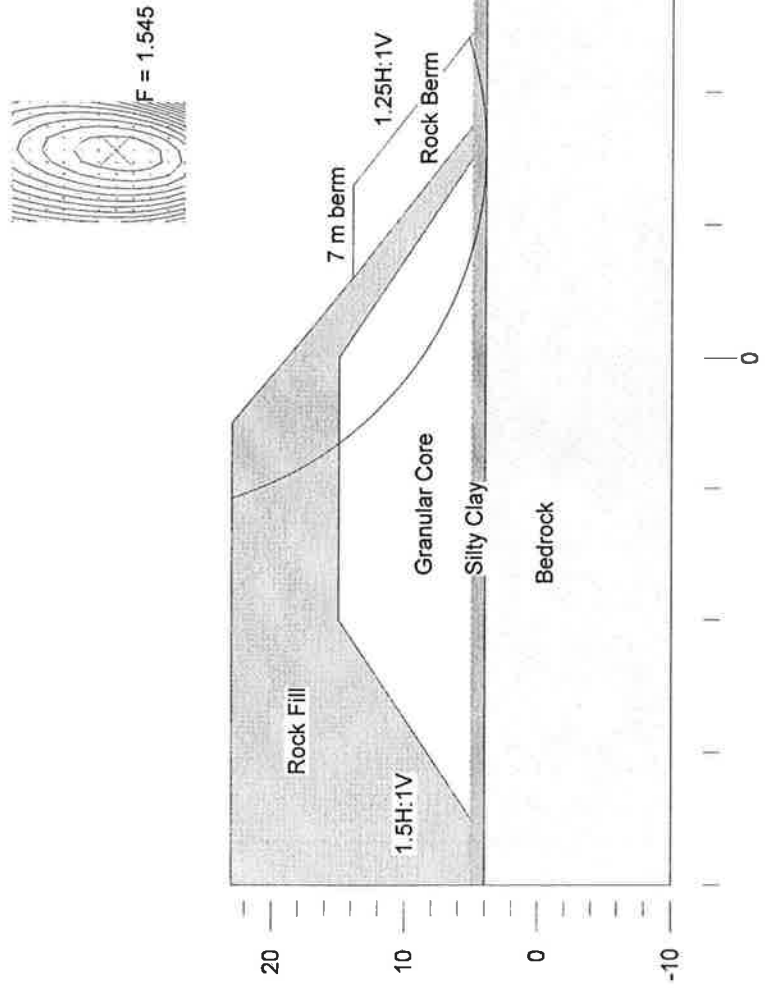
Selected Stability Analyses Results

	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Rock Berm	19	0	42	0	0
Rock Fill	19	0	42	0	0
Granular core	21	0	30	0	0
Silty Clay v.st.	20	0	27	0	1
Bedrock	(Infinitely Strong)				



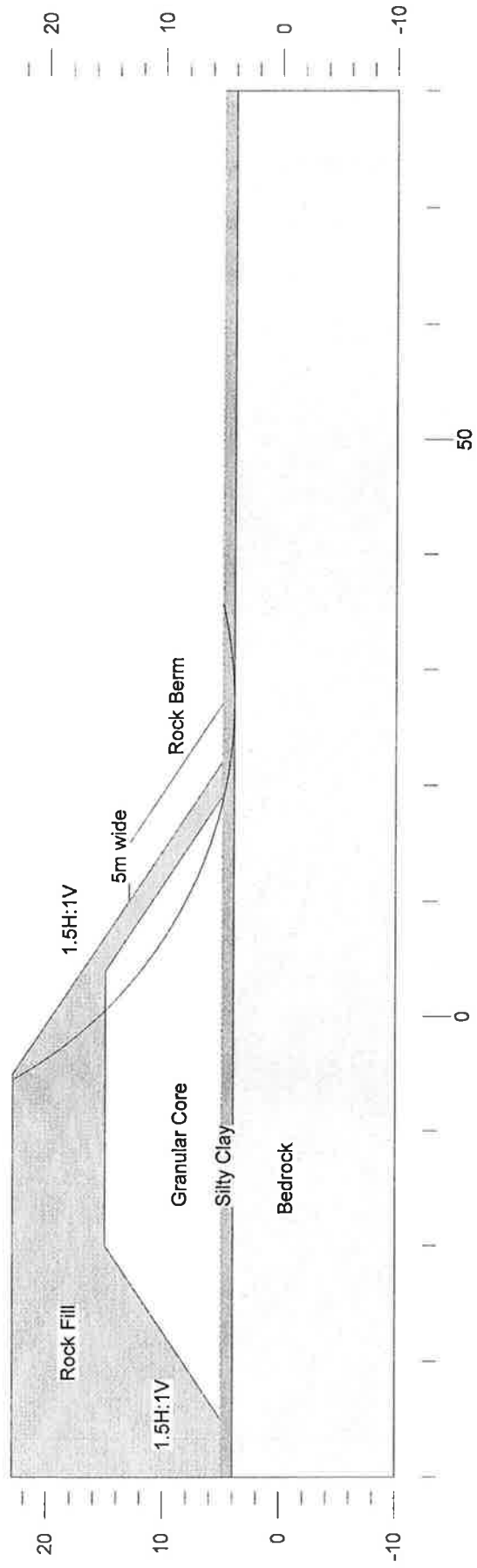
Stability of Embankment Forward Slope - Madawaska River Bridge - East Approach
 FIG G2 Undrained Analysis - 7 m wide mid-height berm with granular core

	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Rock Berm	19	0	42	0	0
Rock Fill	19	0	42	0	0
Granular core	21	0	30	0	0
Silty Clay v. st	20	70	0	0	1
Bedrock	(Infinitely Strong)				



Thurber Engineering Ltd. - Toronto
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 Hwy 17 Twinning
 August 9, 2004
 Stability of Embankment Forward Slope - Madawaska River Bridge - East Approach
 FIG G3 Drained Analysis - Mid height 5 m wide berm with granular core

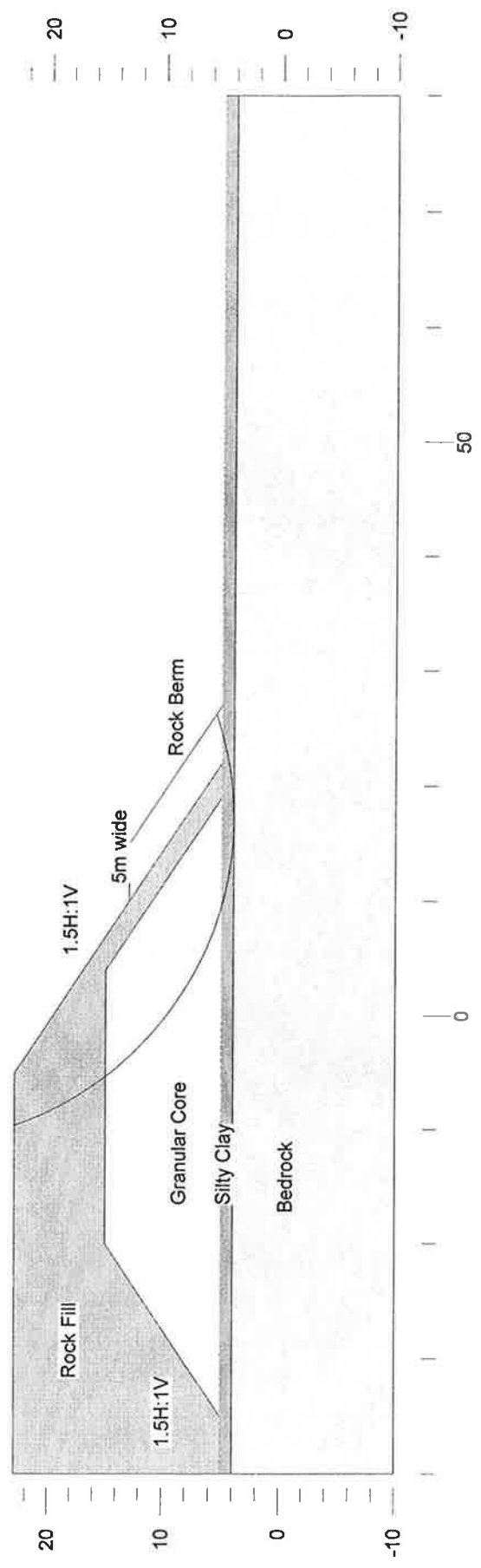
	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Rock Berm	19	0	42	0	0
Rock Fill	19	0	42	0	0
Granular core	21	0	30	0	0
Silty Clay v. st	20	0	27	0	1
Bedrock	(Infinitely Strong)				



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Stability of Embankment Forward Slope - Madawaska River Bridge - East Approach
 FIG G4 Undrained Analysis - Mid height 5 m wide berm with granular core

	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Rock Berm	19	0	42	0	0
Rock Fill	19	0	42	0	0
Granular core	21	0	30	0	0
Silty Clay v. st	20	70	0	0	1
Bedrock	(Infinitely Strong)				



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 Stability of Embankment Forward Slope - Madawaska River Bridge - West Approach
 FIG G5 Drained Analysis - Mid height 2 m wide berm

	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Earth Berm	21	0	30	0	0
Earth Fill	21	0	30	0	0
Bedrock	(Infinitely Strong)				


 $F = 1.380$

