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FOUNDATION INVESTIGATION AND DESIGN REPORT
DIVISION STREET UNDERPASS
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-417
GEOCRES Number: 31F-127

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of the proposed Division Street Overpass structure that will carry Division Street over the proposed Highway 17 westbound lanes. A preliminary foundation investigation was carried out by the Ministry of Transportation (MTO) in 1969 at five possible structure sites situated approximately within the present portion of Highway 17 from Antrim Westerly to Locheil Creek. The factual data from that investigation has been used as a reference during the preparation of this report.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions.

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

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

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

2 SITE DESCRIPTION

The site is located about 50 m east of the existing at grade intersection of Highway 17 and Division Street/Pine Grove Road, Township of McNab, County of Renfrew (approximate mainline Station 27+705 on existing Highway 17). This location is to the southwest of the Town of Arnprior.

The site is flat and there are private dwellings on large open fields to the north and south of the existing Highway. Vegetation is light and consists mainly of grass, some small trees and shrubs.

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 consists 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. There are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechère” graben. To the south and east of Arnprior, the bedrock is predominantly crystalline limestone of the Ordovician Period. To the west of Arnprior, the bedrock consists of limestone interbedded with metamorphosed greywacke of the Precambrian Age.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of October 07 to 10, 2003 and on December 19, 2003. The site investigation consisted of drilling and sampling a total of seventeen boreholes to depths ranging from 1.0 to 7.1 m. The boreholes were numbered DIV-1 to DIV-17. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited and utility clearances were obtained by Thurber prior to drilling. Since clearances for utilities that apparently existed in close proximity of Boreholes DIV-9 and DIV-10 could not be obtained, a hydrovac system was used to advance these two boreholes.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations for fifteen of the seventeen boreholes including DIV-1 to DIV-8 and DIV-11 to DIV-17. 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. Two boreholes within each of the three foundation elements were advanced 2.8 to 3.2 m into bedrock by NQ size rotary coring techniques.

As mentioned earlier, a Hydrovac system supplied and operated by Badger Daylighting Inc. of Ottawa, Ontario was used to advance Boreholes DIV-9 and DIV-10 to avoid damaging any underground utilities. The Hydrovac system consists of injecting pressurised water into the ground via a handheld wand. The resulting slurry is simultaneously vacuumed from the excavation through a large debris hose and temporarily stored in a debris tank. At these two borehole locations the overburden was removed to expose the rock surface, which was then probed using a 25 mm diameter steel bar. These holes were then backfilled with crushed stone.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. One standpipe piezometer was installed in each of Boreholes DIV-2, DIV-7 and DIV-12 (one at each proposed foundation element location) to permit monitoring of the groundwater level. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screens were installed in the boreholes. The sand screen surrounding the pipe was about 1.8 m long. Bentonite holeplug seals were placed just above the sand screen and just below the ground surface in each installation. The remaining space in the boreholes was backfilled with drill cuttings.

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 without a piezometer installation were appropriately backfilled.

The ground surface elevations and plan coordinates at the locations of all boreholes have been established in the field and the survey data forwarded to Thurber by J. D. Barnes Limited.

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 and Atterberg Limit Tests were performed on 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 contained in Appendix B.

Point load tests were performed on rock cores retrieved from Boreholes DIV-2, DIV-6, DIV-7, DIV-11, DIV-12 and DIV-16. These results are shown in Table 1 at the end of this report.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the "Borehole Locations and Soil Strata" drawing in Appendix F. A description of the stratigraphy is given in the following paragraphs.

In general, the site is underlain by a relatively thin deposit of overburden soils overlying bedrock. The overburden soils range in thickness from about 1 m to 3.9 m and the overburden consists of fill, clayey silt to silty clay, and clayey silt till.

5.1 Topsoil

Topsoil was encountered across the site in fifteen boreholes to depths of 75 to 255 mm as shown below.

| Borehole | Topsoil Thickness (mm) |
|-----------------|-------------------------------|
| DIV-1 | 200 |
| DIV-2 | 150 |
| DIV-3 | 100 |

| | |
|--------|-----|
| DIV-4 | 150 |
| DIV-5 | 175 |
| DIV-6 | 250 |
| DIV-7 | 175 |
| DIV-8 | 250 |
| DIV-11 | 100 |
| DIV-12 | 75 |
| DIV-13 | 75 |
| DIV-14 | 100 |
| DIV-15 | 100 |
| DIV-16 | 100 |
| DIV-17 | 125 |

Topsoil thickness may vary between and beyond the boreholes.

5.2 Fill

Fill was encountered on the east side of the site in one borehole only (DIV-14). No fill was noted in any of the other boreholes.

Borehole DIV-14 was likely drilled through the remains of an old road (most likely Division Street before it was realigned to the present alignment). The topsoil in Borehole DIV-14 is underlain by a 1.4 m thick deposit of brown to dark brown clayey silt fill with topsoil inclusions. The fill has a stiff consistency based on SPT 'N' values of 11 to 13. The measured moisture content of samples of the fill ranged from 16% to 24%.

The clayey silt fill material in Borehole DIV-14 is further underlain by silty sand fill containing some gravel. A grain size analysis performed on a representative sample from this deposit (Figure B 1) gave a grain size distribution of 17% gravel, 60% sand and 23% silt and clay size particles. An SPT 'N' value of 40 blows for 0.3 m penetration was recorded within this fill material thereby indicating a relatively dense state.

5.3 Silty Sand

On the west side of the site, a 0.4 m thick deposit of brown to dark brown silty sand was encountered in Borehole DIV-6 below the surficial veneer of topsoil. This granular deposit extends to a depth of 0.7 m below ground surface or to Elevation 110.6 m. This silty sand is in a compact state based on a blow count of 18 blows per 0.3 m penetration. The measured moisture content ranged from 13% to 20%.

5.4 Clayey Silt

On the west side of the site, the topsoil in Boreholes DIV-2, DIV-3 and DIV-4 and the silty sand deposit in Borehole DIV-6 are underlain by a deposit of clayey silt encountered at depths ranging from 0.1 m to 0.7 m (Elevations 111.7 m to 110.6 m). This deposit extends to depths of 0.7 m to 1.5 m below ground surface (Elevations 111.1 m to 109.9 m). The

deposit was found to contain some sand and trace gravel with topsoil inclusions and occasional rootlets. This cohesive deposit is considered to have a firm to hard consistency based on SPT 'N' values that ranged from 7 to 32 blows for 0.3 m penetration. The measured natural moisture content of the clayey silt ranges approximately from 12% to 21%.

5.5 Silty Clay

In a number of boreholes across the site (Boreholes DIV-1, DIV-5, DIV-7, DIV-11, DIV-12, DIV-13, DIV-15, DIV-16 and DIV-17) the topsoil is underlain by a silty clay layer encountered at depths ranging from 0.1 to 0.2 m below ground surface. This deposit extends to depths ranging from 0.7 to 2.6 m or from Elevations 110.6 m to 108 m. Grain size analyses conducted on three samples retrieved from this unit are presented in Figure B2. These results show the following grain size distribution:

| | |
|---------|-----------|
| Gravel: | 0% |
| Sand: | 10% - 13% |
| Silt: | 47% - 52% |
| Clay: | 37% - 43% |

Atterberg Limits tests were also conducted on representative samples from this stratum and the results are presented in Figures B3. These results are summarized below:

| | |
|-------------------|-----------|
| Liquid Limit: | 36% - 42% |
| Plastic Limit: | 18% - 21% |
| Plasticity Index: | 18% - 21% |

These values are characteristic of clayey soils of medium plasticity.

Standard Penetration Tests conducted within this deposit gave 'N' values ranging from 2 to 30 blows per 0.3 m penetration but generally most 'N' values ranged from 7 to 17 blows per 0.3 m penetration. Based on these results, the deposit is considered to be generally firm to very stiff. The measured moisture contents of samples recovered from this unit ranged from 10% to 29%.

Boreholes DIV-9 and DIV-10 were advanced using Hydrovac equipment as mentioned previously. In these boreholes it is inferred that the silty clay deposit exists below the standing water encountered in Borehole DIV-9 and at the ground surface in Borehole DIV-10. It is noted that the measured moisture content of a grab sample recovered from Borehole DIV 10 was of the order of 48%. This high moisture content is probably due to the pressurised water used to advance the borehole.

5.6 Clayey Silt Till

Underlying the thin deposits of surficial soils and the silty clay, a brown clayey silt till layer was encountered across the site in a number of boreholes (DIV-1 to DIV-4, DIV-6, DIV-7, DIV-8, DIV-11 and DIV-15 to DIV-17). This deposit was encountered at depths ranging from 0.3 m to 1.5 m (Elevations 111.2 m to 108.2 m) and it extends to depths of 2m to 3.9 m below ground surface (Elevations 109.9 m to 106.9 m). The thickness of the till layer ranges from 0.7 to 3.3 m.

Grain size analyses conducted on samples retrieved from this unit are presented in Figures B4 and B5. These results show the following grain size distribution:

| | |
|---------|-----------|
| Gravel: | 5% - 23% |
| Sand: | 40% - 53% |
| Silt: | 24% - 35% |
| Clay: | 12% - 21% |

The fine grained portion of the deposit is described as clayey silt (CL-ML) and Atterberg Limit Tests conducted on representative samples gave the following index values:

| | |
|-------------------|-----------|
| Liquid Limit: | 16% - 28% |
| Plastic Limit: | 11% - 17% |
| Plasticity Index: | 5% - 13% |

The till has a stiff to hard consistency based on SPT 'N' values that ranged from 13 to more than 66 blows for 0.3 m penetration. It should be pointed out that in some of the boreholes, the desired penetration of the SPT sample (0.3 m) was not obtained and sampler refusal was attributed to the presence of cobbles, boulders and over size particles in the bottom portion of this deposit (Boreholes DIV-2, DIV-4, DIV-5, DIV-7, DIV-9, DIV-10 and DIV-17). Based on the borehole information, the clayey silt till contains cobbles and boulders. The measured natural moisture contents of samples retrieved from this stratum range approximately from 5% to 25%.

5.7 Bedrock

The overburden soils described above are underlain by crystalline limestone bedrock. In Borehole DIV-11 a 1.0 m thick greywacke unit is interbedded within the limestone. Bedrock was proved by coring in Boreholes DIV-2, DIV-6, DIV-7, DIV-11, DIV-12 and DIV-16. The bedrock surface was inferred from probing with a 25 mm diameter steel rod in Boreholes DIV-9 and DIV-10 (advanced with Hydrovac equipment) and from refusal to auger penetration in the remainder of the boreholes drilled at this site. Table 1 summarizes the depth to bedrock which varies between 1 m and 3.9 m at the site.

| Borehole Number | Depth to Bedrock | Top of Bedrock Elevation |
|-----------------|------------------|--------------------------|
| DIV-1 | 2.1 | 109.9 |
| DIV-2 | 3.3* | 108.6* |
| DIV-3 | 2.6 | 109.2 |
| DIV-4 | 3.5 | 108.0 |
| DIV-5 | 2.6 | 109.1 |
| DIV-6 | 3.9* | 107.4* |
| DIV-7 | 2.5* | 108.1* |
| DIV-8 | 3.6 | 107.8 |
| DIV-9 | 1.0 | 108.6 |
| DIV-10 | 1.8 | 109.3 |
| DIV-11 | 2.7* | 106.9* |
| DIV-12 | 1.5* | 108.8* |
| DIV-13 | 2.3 | 107.9 |
| DIV-14 | 2.3 | 108.3 |
| DIV-15 | 2.0 | 108.4 |
| DIV-16 | 2.2* | 108.0* |
| DIV-17 | 3.0 | 107.4 |

* Proved by coring

The limestone bedrock is described as fresh with occasional slightly weathered joints. Its colour is grey, with visible grey and white horizontal and sub-vertical banding in most cores. The greywacke bedrock encountered within the limestone at Borehole DIV-11 is described as fresh and is dark grey to black with sub-vertical wavy white banding.

Core recovery in the bedrock was generally 95 to 100% and the RQD values typically ranged from 59% (in Borehole DIV-6) to 100%. For the most part however, the RQD values generally ranged between 81% and 100% indicating a good to excellent rock quality. It should be noted that an RQD of 0% was recorded in Borehole DIV-7 during Run 2 due to the presence of a sub-vertical to vertical joint encountered throughout the length of the run.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low, ranging from 0 to 3 except at Boreholes DIV-2 and DIV-6 where FI values of 4 were recorded in the first 0.3 m of core. Most of the joints were generally sub-horizontal and rough and were mostly tight with no infilling or secondary weathering material, although some sub-vertical joints were also present.

The unconfined compressive strength of the rock cores is estimated to exceed 100 MPa indicating that the intact rock is strong to very strong. However, in Borehole DIV-7, the

strength of the rock was assessed to be approximately 46 MPa between 2.5 m to 2.9 m inferring moderately strong rock within this zone. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table 2 attached immediately following the text.

5.8 Water Levels

Standpipe piezometers were installed in Boreholes DIV-2, DIV-7 and DIV-12 and their water levels were measured on three separate visits made after the completion of drilling. These readings are presented in the table below.

| Borehole | Date | Water Levels | |
|----------|-------------------|--------------|---------------|
| | | Depth (m) | Elevation (m) |
| DIV-2 | October 22, 2003 | 1.5 | 110.4 |
| DIV-2 | December 18, 2003 | 1.4 | 110.5 |
| DIV-2 | February 05, 2004 | 2.6 | 109.3 |
| DIV-7 | October 22, 2003 | 0.9 | 109.7 |
| DIV-7 | December 18, 2003 | 1.4 | 109.2 |
| DIV-7 | February 05, 2004 | 1.4 | 109.2 |
| DIV-12 | October 22, 2003 | 1.4 | 108.9 |
| DIV-12 | December 18, 2003 | 1.4 | 108.9 |
| DIV-12 | February 05, 2004 | 1.5 | 108.8 |

Borehole DIV-9 was advanced from the bottom of a ditch containing water. At the time of investigation (December 19, 2003), 0.3 m of standing water was encountered below a thin coat of ice at this borehole location.

Based on these observations, local groundwater levels exist at Elevations 108.8 m to 110.5m. 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 presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

At this location it is understood that Highway 17 will be twinned by constructing two west bound lanes about 40 m centre to centre east of the present highway. Division Street will be realigned to cross over Highway 17 about 50 m south of the present intersection at Sta. 27 + 705 on Highway 17 EBL and at Sta. 27 + 751 on proposed Highway 17 WBL.

Based on the preliminary general arrangement drawings, the proposed structure will consist of a two-span underpass. Each span will be approximately 53 m long and Division Street will intersect the future WBL and EBL of Highway 17 at a skew angle of 45°. The proposed final grade of Division Street is at about Elevation 120 m at both abutments. Consequently, approach fill heights of about 8 m and 10 m will be required at the west and east approaches, respectively.

At the site, the proposed westbound alignment of Highway 17 will be in a shallow fill section approximately 1.5 m to 2 m deep.

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

7 STRUCTURE FOUNDATIONS

The proposed bridge at this site is a two-span underpass structure with a total of three foundation elements: two abutments and one pier.

The stratigraphy encountered at the locations of the three foundation elements consists of 1 m to 4m of overburden soils overlying bedrock. The overburden consists of some fill, clayey silt/silty

clay and silty clay till with cobbles and boulders. Water level exists at elevations ranging from Elevations 108.8 m to 110.5 m.

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

| Location | Borehole Number | Elevations (m) | |
|---------------|-----------------|-------------------------|----------------|
| | | Existing Ground Surface | Top of Bedrock |
| West Abutment | | | |
| NW Corner | DIV-2 | 111.9 | 108.6 |
| NE Corner | DIV-3 | 111.8 | 109.2 ± * |
| Interior | DIV-4 | 111.5 | 108.0 ± * |
| SW Corner | DIV-5 | 111.7 | 109.1 ± * |
| SE Corner | DIV-6 | 111.3 | 107.4 |
| Centre Pier | | | |
| NW Corner | DIV-7 | 110.6 | 108.1 |
| NE Corner | DIV-8 | 111.4 | 107.8 ± * |
| Interior | DIV-9 | 109.6 | 108.6 ± ** |
| SW Corner | DIV-10 | 111.1 | 109.3 ± ** |
| SE Corner | DIV-11 | 109.6 | 106.9 |
| East Abutment | | | |
| NW Corner | DIV-12 | 110.3 | 108.8 |
| NE Corner | DIV-13 | 110.2 | 107.9 ± * |
| Interior | DIV-14 | 110.6 | 108.3 ± * |
| SW Corner | DIV-15 | 110.4 | 108.4 ± * |
| SE Corner | DIV-16 | 110.2 | 108.0 |

* Bedrock inferred from auger refusal.

** Bedrock inferred by probing with a 25 mm diameter steel bar.

7.1 Foundation Alternatives

This section discusses the feasible foundation alternatives, provides geotechnical design parameters and recommends a preferred foundation scheme.

Initial consideration was given to the following foundation types:

- Spread footings
- Driven piles
- Augered caissons (drilled shafts)

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

An integral abutment design was not considered to be a feasible option due to the presence of bedrock at shallow depth and the skew angle of the structure. Semi-integral abutment

design utilizing footings on bedrock may be considered, but the proposed span lengths (53m) are likely to be too long for such a design.

The presence of shallow bedrock also renders it impractical to use driven piles at this site. Accordingly, the driven pile alternative was eliminated from further consideration.

Consideration may be given to using augered caissons founded on bedrock at the abutments. However, an engineered fill platform will need to be constructed to accommodate caisson augering equipment prior to installation. The caissons will also require to be nominally socketted into bedrock to enhance adequate contact between the caisson and the rock over the entire base area. This will require specialized equipment such as core barrels and shot barrels, which will most likely be costlier than other options.

Therefore, the recommended foundation type at this site is spread footings and these could be designed either to bear on engineered fill pads resting on native, stiff to hard clayey silt to clayey silt till, or on bedrock.

7.2 Spread Footings on Bedrock

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of the highway, the structure can be supported on footings bearing on sound bedrock. The excavation for footing construction on bedrock will be in the order of 2 to 4 m deep below the existing ground surface and the excavations may need temporary support and dewatering for footing construction.

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

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

Rock excavation is expensive and unnecessary excavation of bedrock should be avoided where practical. The recommended design top of rock is as follows:

7.2.1 West Abutment

The top of rock varies from Elevations 107.4 m to 109.2 m± across this foundation, as shown in Section 7 of this report.

7.2.2 Centre Pier

At the location of this foundation element, the top of rock varies from Elevations 106.9 m to 109.3 m± as shown in Section 7.

7.2.3 East Abutment

The top of rock varies from Elevations 107.9 m to 108.8 m across this foundation, as shown in Section 7 of this report.

7.3 Bearing Resistance

Footings bearing on sound limestone bedrock encountered at this site may be designed on the basis of a geotechnical resistance of 5,000 kPa at factored ULS for vertical, concentric loads. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

The SLS condition will not govern 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.

7.4 Horizontal Resistance of Footings

The resistance to lateral forces/sliding resistance between the concrete footing and the bedrock surface at the pier locations should be evaluated according to CHBDC, 2000 and a value of 0.85 can be assumed for the ultimate coefficient of friction of concrete poured on rock.

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 only shear resistance.

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

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

7.5 Spread Footings on Engineered Fill

For a perched abutment design at both the east and west abutment locations, spread footings may be founded on an engineered fill pad resting on native, stiff to hard clayey silt to clayey silt till.

If an engineered fill pad is used at this site, all topsoil, and overburden material containing organics should be removed and the fill placed directly on the typically stiff to hard clayey silt or clayey silt till at the abutments. At the east abutment, the base of the engineered fill pad may be founded at or below approximate Elevation 110 m. At the west abutment, the base of the engineered pad may be founded between Elevations 111 m and Elev. 112 m. It is recommended that the thickness of the fill pad is equal to or greater than the footing width, and should not be less than 2 m. Consideration may be given to founding the Granular A pad on bedrock. The elevations of the base of the engineered fill should correspond to the top of bedrock discussed in Section 7.2.

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 generally conforming to the geometry illustrated in Figure D1.

Provided a minimum footing width of 2 m is maintained, a footing bearing on a compacted Granular A pad may be designed for the following geotechnical resistance:

| | | |
|--------------|---|----------|
| Factored ULS | – | 900 kPa |
| SLS | – | 350 kPa. |

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

The SLS value quoted above corresponds to a settlement of up to 25 mm that is expected to be complete by the end of construction.

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

7.6 Frost Cover

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

If footings are founded on engineered fill, or native earth material, frost protection should be provided. 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 more than 2.5 m below the underside of the foundation.

7.7 Augered Caissons

The abutments may be supported by augered caissons (drilled shafts) founded on bedrock. In order to found the caissons below the surficial, more fractured zone of the bedrock and to enhance caisson base contact with sound bedrock, it is recommended that the caissons be designed to be nominally socketted at least 500 mm into bedrock. The sockets should be formed below the low side of a sloping bedrock surface. The recommended design top of bedrock is the same as those presented for spread footings in Section 7.2.

7.7.1 Axial Resistance

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

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

7.7.2 Downdrag

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

Due to the small thickness of compressible cohesive soils (2 m or less) present above bedrock at this site, it is estimated that the magnitude of downdrag force acting on a caisson is relatively insignificant comparing with its end bearing resistance, and will not affect the geotechnical resistance recommended in this report. Downdrag is, therefore, not considered to be a design issue at this site.

7.7.3 Lateral Resistance

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

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

Engineered Sand and Gravel Fill (compact)

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

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

Silty Clay (stiff)

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

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

where

z = depth below abutment base in metres

D = caisson diameter in metres

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

γ = 20 kN/m³

K_p = 3.0 (passive earth pressure coefficient)

S_u = 75 kPa for undrained shear strength of stiff silty clay

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

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

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

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

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

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

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

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

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

Intermediate values may be obtained by interpolation.

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

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

where

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

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

7.7.4 Caisson Installation

Given the presence of bedrock at relatively shallow depths below existing ground surface at this site, an engineered fill platform will need to be constructed to accommodate caisson augering equipment prior to installation.

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

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

The base of the caisson should be drilled at least 500 mm into the bedrock to remove weathered and more fractured rock, and to mitigate the impact of a sloping rock surface. For moderately sloping bedrock surface anticipated at this site, it is recommended that the base of the caisson should be drilled to 500 mm below the low side of the rock surface.

Stepping of the caisson base is allowed in SP 903S01, but is likely not required for this site.

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

8 EXCAVATION AND BACKFILL

8.1 General

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the clayey silt fill, clayey silt and silty clay deposits can be classified as Type 3 soil. The clayey silt till, native silty sand and the silty sand fill deposits at this site are classed as Type 2 soil above the water table and Type 3 soil if below the water table.

8.2 Foundations

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

8.3 Earth Excavation

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

At the west abutment and the central pier locations, excavation for footing construction will extend through the existing cohesive deposits of clayey silt, silty clay and clayey silt till as well as occasional silty sand deposits such as that encountered in Borehole DIV-6. It is anticipated that excavation for foundation construction would be carried out in conjunction with a temporary shoring system due to the relatively close proximity of the existing highway. A similar method of excavation may also be required at the east abutment depending on the final alignment of this underpass structure. The footing excavations will also extend below the groundwater level encountered at about 1.5 m below existing ground surface.

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

A braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring. It should be pointed out that the presence of cobbles and boulders in the clayey silt till could result in hard pile driving conditions. The soldier piles will need to be installed through pre-drilled holes and socketted into bedrock in order to develop the

required fixity. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

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

$$\begin{aligned}\gamma &= 20 \text{ kN/m}^3 \\ \gamma_w &= 10 \text{ kN/m}^3 \\ K_a &= 0.4 \text{ (silty clay, clayey silt \& clayey silt till)} \\ h_w &= 0 \text{ (assuming no hydrostatic pressure build-up behind a} \\ &\text{presumably permeable wall)} \\ H &= \text{depth to base of excavation (rock surface) (m)}\end{aligned}$$

Below the base of the excavation within bedrock, 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. Consequently, 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

$$\begin{aligned}c &= 2000 \text{ kPa (equivalent Mohr-Coulomb cohesion based on} \\ &\text{the Hoek and Brown rock mass classification)} \\ L &= \text{depth of socket in rock, (m)}\end{aligned}$$

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

8.4 Rock Excavation

It is anticipated that requirements for rock excavation will be minimal at this site. The strength of intact rock is strong to very strong. Relatively shallow excavation into rock for footing construction may require appropriate excavation with rock teeth and rock splitting equipment.

Where quantities of rock have to be removed and the excavation is extended into largely unfractured bedrock, it is anticipated that the Contractor may elect to use blasting methods. The design of the blast and removal procedures should be the responsibility of the Contractor. However, it is important that his procedures incorporate methods of reducing damage to the founding surfaces and nearby structures.

Any damage to the founding surfaces on bedrock must be made good prior to constructing the foundation. Open joints and fractures may be encountered at the design founding elevation. These features must be cleaned out and the voids must be filled by grouting prior to constructing the footing.

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

The Contractor's blasting and monitoring plan must not result in damage to any near-by buildings or other structures. The contract documents should alert the contractor of any such structures. During construction it is recommended that the Contract Administrator retain the services of a blasting expert to examine and assess the Contractor's procedures prior to approving them.

9 GROUNDWATER CONTROL

The water level at the site is about 1.5 m below existing ground surface and temporary excavations for footing or engineered fill construction may extend below the groundwater level.

The relatively impervious deposits of silty clay, clayey silt and clayey silt till should not yield significant amounts of water in the short term. At locations where more pervious zones (such as sand seams and layers) exist within the till deposit, trapped water will seep into excavations if cuts are made through these seams and layers.

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

10 APPROACH EMBANKMENTS

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

Anticipated settlements due to compression of the cohesive deposits existing at the founding level have been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6. Settlement due to elastic recompression of the over-consolidated clayey silt, silty clay and clayey silt till deposits is estimated to be in the order of 25 mm and is anticipated to be complete by the end construction. Post construction settlement should be negligible. Some minor settlement will occur within the rock fill, granular or SSM fill. This settlement should be complete by the end of construction. However, if clean inorganic earth fill consisting of clayey materials is used, post construction settlement of the clayey fill itself could be up to the order of 25 mm.

The approach embankments for this structure will be constructed on relative shallow cohesive deposits of silty clay and clayey silt and clayey silt till overlying bedrock. The embankments can be expected to be about 8 m to 10 m high (Elevation 120 m) at the abutments, decreasing in height with increasing distance from the bridge structure. The foundation soils will satisfactorily support

the approach fills at this site assuming that all organic, weak or otherwise unsuitable material will be removed prior to placing the embankment fills.

At the east approach, the required embankment will be up to 10 m in height. All organic, weak or otherwise unsuitable overburden materials should be removed from the footprint of the embankment. At this approach the embankment fill will bear on stiff to very stiff silty clay to clayey silt and hard clayey silt till underlain by bedrock. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations not steeper than 1.25H:1V. Embankments constructed using granular material, select subgrade material and clean, inorganic earth material will have stable side slopes at inclinations no steeper than 2H:1V. Based on slope stability analyses, the factors of safety for short term (total stress, undrained) and long term (effective stress, drained) conditions of the critical forward slope for embankments constructed using granular material were in the order of 1.4 and 1.3, respectively. For rock fill embankments, factors of safety in the order of 1.5 and 1.3 were obtained for short term and long term conditions, respectively.

At the west approach the required embankment height will be up to 8 m in height. All organic, weak or otherwise unsuitable overburden materials should be removed from the footprint of the embankment. At this approach the embankment fill will bear generally on stiff to very stiff clayey silt and hard clayey silt till. Stable fill slopes for different materials are the same as those for the east abutment. Slope stability analyses of the west approach embankment constructed using granular material or rock fill yielded factors of safety in the order of 1.5 for both short and long term conditions.

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

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

The approach embankments will be underlain by native silty clay, clayey silt, clayey silt till or bedrock and the groundwater level is below the base of the embankment. These materials have negligible to no potential for liquefaction. Consequently, the approach embankments will be stable with respect to 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. Given the shallow bedrock and competent native soils above the bedrock, the risk of using RSS walls at this site is considered low.

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

The levelling pad for the RSS wall may be formed directly on the bedrock, mass concrete fill or on a pad of engineered fill. Engineered fill should consist of OPSS Granular A compacted in accordance with the OPSS 501, Method A. The engineered fill pad should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

In addition to the requirements for the levelling pad, the RSS can be founded on well compacted approach fill or on stiff to very stiff clayey silt to silty clay or hard clayey silt till. 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 5000 kPa for walls founded directly on limestone bedrock (SLS is not applicable for foundations on rock)
- Factored geotechnical resistance at ULS of 900 kPa, and geotechnical resistance of 350 kPa at SLS for an engineered Granular A pad resting on bedrock.
- Factored geotechnical resistance at ULS of 300 kPa, and geotechnical resistance of 200 kPa at SLS on native, stiff to hard clayey silt, silty clay or clayey silt till at or below Elevations 110 m and 112 m at the east and west approaches, respectively.
- Ultimate coefficient of friction between cast in-situ concrete levelling pad on Granular A is 0.7.
- Ultimate coefficient of friction between RSS mass and Granular A is 0.55.
- Ultimate coefficient of friction between RSS mass and native clayey silt, silty clay or clayey silt till is 0.45.

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

11.2 Global Stability

The global stability of the RSS wall 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 are likely to be used as wing walls at the abutments. It is envisaged that the RSS will be founded on an engineered fill core resting on native clayey silt till or bedrock.

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

- Engineered fill – Granular A compacted to 100% SPMDD at $\pm 2\%$ optimum moisture content, with a slope of 1H : 1V (angle of internal friction, ϕ , of 35° , cohesion of 0, and unit weight, γ , of 22.8 kN/m^3).
- Rock fill – outer slope of 1.25H : 1V (angle of internal friction, ϕ , of 42° , cohesion of 0, and unit weight, γ , of 19 kN/m^3).
- Groundwater level at 1 m below existing ground surface.
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 50% of the height founded on a Granular A core at the abutments.

Assuming similar geometric configurations as in Section 10 and incorporating an RSS wall, results of the stability analyses indicate that the F.S. would be in the order of 1.4 to 1.5 at the east abutment, provided that the Granular A pad is resting on stiff to hard clayey silt till overlying bedrock. The F.S. will be in the order of 1.5 at the west abutment. These values indicate that global stability can be maintained for the assumed RSS/approach fill configurations.

Based on the above, it may be assumed that RSS walls founded on compacted Granular A pad that is resting on the native stiff to hard clayey silt till or bedrock will be stable against global failure. The actual design configuration must be checked for global stability prior to finalization.

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

For walls founded on bedrock, settlements will be negligible. The settlement of a wall founded on an engineered fill pad will depend on the thickness of the pad, the material used, the foundation soils and the quality of construction. However, settlements are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

12 BACKFILL TO ABUTMENTS

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

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

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

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

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

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

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

13 EARTH PRESSURE

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

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

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

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

where P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see below)

γ = unit weight of retained soil (see below)

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

q = value of any surcharge (kPa)

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

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

| Wall Condition | Earth Pressure Coefficient (K) | | | | | |
|---|--|---|---|---|--|---|
| | OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$ | | OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$ | | Rock Fill $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$ | |
| | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) |
| Active (Unrestrained Wall) | 0.27 | 0.40* | 0.31 | 0.48* | 0.2 | .28* |
| At rest (Restrained Wall) | 0.43 | - | 0.47 | - | .33 | - |
| Passive (Movement Towards Soil Mass) | 3.70 | - | 3.30 | - | 5.0 | - |

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consist of rock fill.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

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

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.10
- Acceleration Related Seismic Zone 4
- Zonal Acceleration Ratio 0.2

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

Site specific seismic hazard data for this area were obtained from Earthquakes Canada. For the design level earthquake (1 in 475 year event), a Peak Horizontal Ground Acceleration at ground surface of 0.184 g and a Peak Horizontal Ground Velocity of 0.091 m/sec should be used for the design of the bridge.

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

14.2 Liquefaction Potential

If the structure foundations bear on bedrock, there is no potential for liquefaction under the foundations. In the case of foundations founded on an engineered fill pad and bearing on the native cohesive soil, there is negligible potential for soil liquefaction.

The immediate approach embankments will bear on relatively shallow cohesive deposits of silty clay, clayey silt and clayey silt till overlying bedrock. Therefore, there is negligible potential for soil liquefaction below the embankments. The embankments themselves will be constructed above groundwater level and are not considered to be in

danger of liquefaction. Some toe failure may occur but is expected to be of limited nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5ϕ . For the design of retaining walls, the following coefficients of horizontal earth pressure may be used:

| Wall Condition | Height of Application From Base as Percentage of Wall Height | Earth Pressure Coefficient (K) for Earthquake Loading | | | | | |
|------------------------|--|---|-------------------------------------|---|-------------------------------------|--|-------------------------------------|
| | | Granular A or Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$ | | OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$ | | Rock Fill $\phi = 42^\circ$; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$ | |
| | | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) |
| Active (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:

- disturbance of the bedrock under the foundations due to blasting or other excavation procedures
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation
- cobbles and boulders may be encountered while excavating through overburden for footing construction.



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



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

Point Load Test Results

TABLE 2
Division Street Underpass
Point Load Test Results

| Depth | | | Is50 | UCS (MPa) | | | | | |
|-------|--------|------|------|--------------|---|-----------------|---------|---------|-----|
| feet | Inches | m | | | | | | | |
| DIV-2 | | | | | | | | | |
| 11 | 1 | 3.38 | 5.62 | 134.83 | } | Total Rock Core | | | |
| 12 | 4 | 3.76 | 4.83 | 115.87 | | Average | Minimum | Maximum | MPa |
| 13 | 6 | 4.11 | 4.35 | 104.28 | | 114 | 55 | 144 | |
| 14 | 1 | 4.29 | 2.28 | 54.78 | | | | | |
| 14 | 9 | 4.50 | 4.61 | 110.60 | | Run # | Average | | |
| 16 | 0 | 4.88 | 5.40 | 129.56 | | 1 | 104.07 | | |
| 17 | 2 | 5.23 | 5.27 | 126.40 | | 2 | 126.14 | | |
| 18 | 8 | 5.69 | 6.01 | 144.31 | | | | | |
| 19 | 8 | 5.99 | 4.35 | 104.28 | | | | | |
| | | | | | | | | | |
| Depth | | | Is50 | UCS (MPa) | | | | | |
| feet | Inches | m | | | | | | | |
| DIV-6 | | | | | | | | | |
| 13 | 9 | 4.19 | 4.83 | 115.87 | } | Total Rock Core | | | |
| 14 | 6 | 4.42 | 4.70 | 112.71 | | Average | Minimum | Maximum | MPa |
| 15 | 9 | 4.80 | 4.92 | 117.98 | | 115 | 87 | 133 | |
| 16 | 10 | 5.13 | 4.21 | 101.12 | | | | | |
| 18 | 5 | 5.61 | 3.64 | 87.43 | | Run # | Average | | |
| 19 | 8 | 5.99 | 5.49 | 131.67 | | 1 | 114.29 | | |
| 21 | 4 | 6.50 | 4.96 | 119.03 | | 2 | 109.55 | | |
| 22 | 3 | 6.78 | 5.00 | 120.08 | | 3 | 123.95 | | |
| 23 | 0 | 7.01 | 5.53 | 132.72 | | | | | |
| | | | | | | | | | |
| Depth | | | Is50 | UCS (MPa) | | | | | |
| feet | Inches | m | | | | | | | |
| DIV-7 | | | | | | | | | |
| 9 | 3 | 2.82 | 1.93 | 46.35 | } | Total Rock Core | | | |
| 12 | 5 | 3.78 | 4.17 | 100.07 | | Average | Minimum | Maximum | MPa |
| 13 | 3 | 4.04 | 5.49 | 131.67 | | 107 | 46 | 135 | |
| 14 | 2 | 4.32 | 4.78 | 114.82 | | Run # | Average | | |
| 15 | 4 | 4.67 | 5.62 | 134.83 | | 1 | 46.35 | | |
| 16 | 2 | 4.93 | 4.65 | 111.66 | | 3 | 115.52 | | |
| 17 | 4 | 5.28 | 4.65 | 111.66 | | 4 | 119.38 | | |

Note: Run #2 - Sub-vertical and vertical joint along the whole run therefore no point load test can be done

Point Load Test Results

TABLE 2 (cont'd)
Division Street
Point Load Test Results

| Depth | | | Is50 | UCS (MPa) |
|--------|--------|------|------|--------------|
| feet | Inches | m | | |
| DIV-11 | | | | |
| 9 | 1 | 2.77 | 4.61 | 110.60 |
| 10 | 0 | 3.05 | 5.57 | 133.78 |
| 11 | 0 | 3.35 | 4.61 | 110.60 |
| 12 | 1 | 3.68 | 7.37 | 176.97 |
| 13 | 0 | 3.96 | 7.72 | 185.39 |
| 14 | 2 | 4.32 | 9.44 | 226.47 |
| 14 | 9 | 4.50 | 1.40 | 33.71 |
| 16 | 2 | 4.93 | 6.14 | 147.47 |
| 18 | 3 | 5.56 | 5.62 | 134.83 |

Total Rock Core
Average Minimum Maximum
140 34 226 MPa

Run # Average
1 122.19
2 174.86
3 105.34

| Depth | | | Is50 | UCS (MPa) |
|--------|--------|------|------|--------------|
| feet | Inches | m | | |
| DIV-12 | | | | |
| 5 | 11 | 1.80 | 4.56 | 109.55 |
| 6 | 11 | 2.11 | 4.61 | 110.60 |
| 7 | 11 | 2.41 | 3.95 | 94.80 |
| 8 | 11 | 2.72 | 4.74 | 113.76 |
| 10 | 3 | 3.12 | 5.44 | 130.62 |
| 11 | 1 | 3.38 | 5.22 | 125.35 |
| 12 | 0 | 3.66 | 5.27 | 126.40 |
| 13 | 0 | 3.96 | 5.27 | 126.40 |
| 14 | 0 | 4.27 | 4.83 | 115.87 |
| 14 | 9 | 4.50 | 5.97 | 143.26 |

Total Rock Core
Average Minimum Maximum
120 95 143 MPa

Run # Average
1 107.18
2 127.98

| Depth | | Is50 | UCS (MPa) | |
|--------|--------|------|--------------|--------|
| feet | Inches | | | |
| DIV-16 | | | | |
| 8 | 0 | 2.44 | 4.74 | 113.76 |
| 8 | 10 | 2.69 | 4.74 | 113.76 |
| 9 | 10 | 3.00 | 5.57 | 133.78 |
| 11 | 6 | 3.51 | 4.92 | 117.98 |
| 12 | 6 | 3.81 | 5.66 | 135.88 |
| 13 | 6 | 4.11 | 4.52 | 108.50 |
| 14 | 6 | 4.42 | 5.49 | 131.67 |
| 15 | 10 | 4.83 | 4.92 | 117.98 |
| 17 | 0 | 5.18 | 5.22 | 125.35 |

Total Rock Core
Average Minimum Maximum
122 108 136 MPa

Run # Average
1 120.43
2 123.51
3 121.66

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



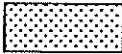


| | |
|------------|--|
| C_{vane} | Shear Strength Determination by Field Insitu Vane |
| C_{pen} | Shear Strength Determination by Pocket Penetrometer |
| C_{lab} | Shear Strength Determination using a Laboratory Vane Apparatus |
| C_U | Undrained Shear Strength determined by Unconfined Compression 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 (MPa) (psi) | Field Estimation of Hardness* | |
| Very thickly bedded | Greater than 2m | Extremely Strong | Greater than 250 | Greater than 36,000 | Specimen can only be chipped with a geological hammer |
| Thickly bedded | 0.6 to 2m | | | | |
| Medium bedded | 0.2 to 0.6m | Very Strong | 100-250 | 15,000 to 36,000 | Requires many blows of geological hammer to break |
| Thinly bedded | 60mm to 0.2m | | | | |
| Very thinly bedded | 20 to 60mm | Strong | 50-100 | 7,500 to 15,000 | Requires more than one blow of geological hammer to break |
| Laminated | 6 to 20mm | | | | |
| Thinly Laminated | Less than 6mm | Medium Strong | 25.0 to 50.0 | 3,500 to 7,500 | Breaks under single blow of geological hammer. |
| | | Weak | 5.0 to 25.0 | 750 to 3,500 | Can be peeled by a pocket knife with difficulty |
| | | Very Weak | 1.0 to 5.0 | 150 to 750 | Can be peeled by a pocket knife, crumbles under firm blows of geological pick. |
| | | Extremely Weak (Rock) | 0.25 to 1.0 | 35 to 150 | Indented by thumbnail |

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

RECORD OF BOREHOLE No DIV-1

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 032 207.7 E 313 665.5 (Division Street) ORIGINATED BY SL
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 10.10.03 - 10.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----------------------------|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | |
| | | | | | | | | ○ UNCONFINED ● QUICK TRIAXIAL | + FIELD VANE × LAB VANE | | | |
| 112.0 | | | | | | | 20 40 60 80 100 | | | | | |
| 111.8 | TOPSOIL (200mm) | | | | | | | | | | | |
| 0.2 | Black Silty CLAY, some sand, with topsoil to 0.7m Stiff to Hard Brown Moist | | 1 | SS | 10 | | | | | | | |
| | | | 2 | SS | 30 | | | | | | | 0 10 47 43 |
| 110.6 | | | | | | | | | | | | |
| 1.4 | Clayey SILT, with sand, trace gravel, occasional cobbles Hard Brown Moist | | 3 | SS | 47 | | | | | | | |
| 109.9 | | | | | | | | | | | | |
| 2.1 | (TILL) END OF BOREHOLE AT 2.13m. AUGER REFUSAL AT 2.13m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 218.5 E 313 676.3 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 10.10.03 - 10.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|-----------------------|----------------------------|-----------------|---|--|------------------------------------|-------------------------------------|-----------------------------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | |
| 111.9 | | | | | | | | 20 40 60 80 100 | | | | | | |
| 111.9 | TOPSOIL (150mm) | | | | | | | ○ UNCONFINED + FIELD VANE | | | | | | |
| 0.2 | Black Clayey SILT, with sand seams, some topsoil, occasional rootlets Stiff to Very Stiff Brown Moist | | 1 | SS | 9 | | | ● QUICK TRIAXIAL × LAB VANE | | 20 40 60 | | | | |
| | | | 2 | SS | 20 | | | | | | | | | |
| 110.4 | | | | | | | | | | | | | | |
| 1.4 | Clayey SILT, with sand, trace gravel, occasional cobbles Very Stiff to Hard Brown Moist (TILL) | | 3 | SS | 18 | | | | | | | | | 5 40 35 21 |
| | | | 4 | SS | 56 | | | | | | | | | |
| | | | 5 | SS | 50/050 | | | | | | | | | |
| 108.6 | | | | | | | | | | | | | | |
| 3.3 | AUGER REFUSAL AT 3.3m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh, very thinly to thinly bedded, grey with dark grey and white horizontal and subvertical banding, very strong Subvertical joint at 3.5m, 3.9m and 4.8m. | | 1 | RUN | 4 1 1 0 0 | | | | | | | | | RUN 1# TCR=96%, SCR=96%, RQD=81%, UCS=104MPa |
| | | | | | 1 0 1 0 0 | | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=126MPa |
| | | | 2 | RUN | | | | | | | | | | |
| 105.8 | | | | | | | | | | | | | | |
| 6.1 | END OF BOREHOLE AT 6.12m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION(m) 22/10/03 110.4 18/12/03 110.5 05/02/04 109.3 | | | | FI | | | | | | | | | |

RECORD OF BOREHOLE No DIV-3

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 218.2 E 313 682.6 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 10.10.03 - 10.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|------------------------------------|-------------------------------------|-----------------------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | |
| 111.8 | | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | |
| 111.6 | TOPSOIL (100mm) | | | | | | | | | | | | | | | |
| 0.1 | Black | | 1 | SS | 7 | | | | | | | | | | | |
| | Clayey SILT, topsoil stained | | | | | | | | | | | | | | | |
| 111.1 | Firm | | | | | | | | | | | | | | | |
| 0.7 | Brown | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | |
| | Clayey SILT, some sand, trace gravel, occasional cobbles | | 2 | SS | 14 | | | | | | | | | | | |
| | Stiff to Very Stiff | | | | | | | | | | | | | | | |
| | Brown | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | |
| | (TILL) | | 3 | SS | 20 | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 109.2 | | | 4 | SS | 50/ 127 | | | | | | | | | | | |
| 2.6 | END OF BOREHOLE AT 2.64m. AUGER REFUSAL AT 2.64m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. | | | | | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-4

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 213.3 E 313 680.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 09.10.03 - 09.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|--|--|------------------------------------|-------------------------------------|-----------------------------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | |
| 111.5 | | | | | | | | 20 40 60 80 100 | | | | | | | |
| 111.0 | TOPSOIL (150mm) | | | | | | | 20 40 60 80 100 | | | | | | | |
| 0.2 | Black | | 1 | SS | 16 | | 111 | | | | | | | | |
| 110.8 | Clayey SILT, some topsoil, trace gravel | | | | | | | | | | | | | | |
| 0.7 | Very Stiff | | | | | | | | | | | | | | |
| | Brown | | 2 | SS | 20 | | 110 | | | | | | | | |
| | Moist | | | | | | | | | | | | | | |
| | Clayey SILT, with sand, trace gravel, occasional cobbles | | | | | | | | | | | | | | |
| | Very Stiff to Hard | | | | | | | | | | | | | | |
| | Brown | | 3 | SS | 40 | | 109 | | | | | | | | |
| | Moist (TILL) | | | | | | | | | | | | | | |
| | limestone fragments at 1.07m | | | | | | | | | | | | | | |
| | frequent cobbles from 1.9m to 2.60m | | | | | | | | | | | | | | |
| 108.0 | | | 4 | SS | 70/ 25 | | 108 | | | | | | | | |
| 3.5 | END OF BOREHOLE AT 3.48m. AUGER REFUSAL AT 3.48m PROBABLY ON BEDROCK OR BOULDERS. | | | | | | | | | | | | | | |

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

RECORD OF BOREHOLE No DIV-5

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 206.2 E 313 675.5 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stern Augers COMPILED BY SS
 DATUM Geodetic DATE 09.10.03 - 09.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | |
| 111.7 | | | | | | | | 20 | 40 | 60 | 80 | 100 | | |
| 111.9 | TOPSOIL (175mm) | | | | | | | | | | | | | |
| 0.2 | Black Silty CLAY , some sand, topsoil stained to 0.7m Very Stiff Brown Moist | | 1 | SS | 17 | | 111 | | | | | | | |
| | | | 2 | SS | 26 | | | | | | | | | |
| | | | 3 | SS | 21 | | 110 | | | | | | | 0 11 52 37 |
| | inferred cobbles from 2.2m to 2.4m. | | | | | | | | | | | | | |
| 109.1 | | | | | | | | | | | | | | |
| 2.6 | END OF BOREHOLE AT 2.59m. AUGER REFUSAL AT 2.59m PROBABLY ON BEDROCK OR BOULDERS. | | | | | | | | | | | | | |

ONTMT4 7450DIV.GPJ 08/03/04

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-6

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 205.6 E 313 686.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 09.10.03 - 09.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | 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 60 80 100 | | | | | | | | | | |
| | | | | | | ○ UNCONFINED | + | FIELD VANE | | | | | | | | | | |
| | | | | | | ● QUICK TRIAXIAL | × | LAB VANE | | | | | | | | | | |
| | | | | | | 20 40 60 80 100 | | | | | 20 40 60 | | | | | | | |
| 111.3 | | | | | | | | | | | | | | | | | | |
| 0.0 | | | | | | | | | | | | | | | | | | |
| 111.1 | TOPSOIL (250mm) | | | | | | | | | | | | | | | | | |
| 0.3 | Black | | 1 | SS | 18 | | | | | | | | | | | | | |
| 110.6 | Silty SAND, and topsoil, trace clay | | | | | | | | | | | | | | | | | |
| 0.7 | Compact | | | | | | | | | | | | | | | | | |
| 109.9 | Brown to Dark Brown | | | | | | | | | | | | | | | | | |
| 1.5 | Moist | | | | | | | | | | | | | | | | | |
| | Clayey SILT, some sand seams, trace gravel | | 2 | SS | 32 | | | | | | | | | | | | | |
| | Hard | | | | | | | | | | | | | | | | | |
| | Brown | | | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | | | |
| | Clayey SILT, with sand, some gravel, occasional cobbles | | 3 | SS | 30 | | | | | | | | | | | | | |
| | Hard | | | | | | | | | | | | | | | | | |
| | Brown | | | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | | | |
| | (TILL) | | 4 | SS | 66 | | | | | | | | | 15 49 24 12 | | | | |
| | | | | | | | | | | | | | | | | | | |
| | Grey/ Brown | | 5 | SS | 38 | | | | | | | | | | | | | |
| 107.4 | | | | | FI | | | | | | | | | | | | | |
| 3.9 | AUGER REFUSAL AT 3.91m. CRYSTALLINE LIMESTONE (BEDROCK) | | | | | | | | | | | | | RUN 1# TCR=95%, SCR=95%, RQD=59%, UCS=114MPa | | | | |
| | Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with white horizontal and subvertical banding, very strong | | 1 | RUN | 3 | | | | | | | | | | | | | |
| | Subvertical joints at 4.6m, 4.7m, 5.2m, 5.3m and 5.4m. | | | | | | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=82%, UCS=109MPa | | | | |
| | | | 2 | RUN | 3 | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | 3 | RUN | 2 | | | | | | | | | RUN 3# TCR=100%, SCR=100%, RQD=80%, UCS=123MPa | | | | |
| 104.3 | | | | | | | | | | | | | | | | | | |
| 7.1 | END OF BOREHOLE AT 7.06m. THE BOREHOLE WAS FILLED WITH DRILL WATER UPON COMPLETION OF ROCK CORING. | | | | | | | | | | | | | | | | | |

ONTMT4 7450DIV.GPJ 09/03/04

RECORD OF BOREHOLE No DIV-7

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 032 213.3 E 313 731.4 (Division Street) ORIGINATED BY SL
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | | |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|------------|------------------------------------|-------------------------------------|-----------------------------------|--|---|-------------------|-----|----------|----|----|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | WATER CONTENT (%) | | | | |
| | | | | | | | | 20 | 40 | 60 | | | | | | 80 | 100 | 20 | 40 | 60 |
| | | | | | | | | ○ UNCONFINED | + | FIELD VANE | | | | | | ● QUICK TRIAXIAL | × | LAB VANE | | |
| 110.6 | | | | | | | | | | | | | | | | | | | | |
| 110.2 | TOPSOIL (175mm) | | | | | | | | | | | | | | | | | | | |
| 0.2 | Black | | 1 | SS | 2 | | | | | | | | | | | | | | | |
| | Silty CLAY, some sand, some rootlets | | | | | | | | | | | | | | | | | | | |
| 109.9 | Soft | | | | | | | | | | | | | | | | | | | |
| 0.7 | Brown | | | | | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | | | | | |
| | Clayey SILT, with sand, some gravel,, occasional cobbles | | 2 | SS | 13 | | | | | | | | | | | | | | | |
| | Stiff | | | | | | | | | | | | | | | | | | | |
| | Brown | | | | | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | | | | | |
| | (TILL) | | 3 | SS | 14 | | | | | | | | | | 14 43 28 15 | | | | | |
| 108.1 | cobble at 2.3m | | | | | | | | | | | | | | | | | | | |
| 2.5 | AUGER REFUSAL AT 2.29m PROBABLY ON BEDROCK OR BOULDERS. CRYSTALLINE LIMESTONE (BEDROCK) | | 1 | RUN | | | | | | | | | | | RUN 1# TCR=98%, SCR=98%, RQD=72%, UCS=46MPa | | | | | |
| | Fresh, slightly weathered at joints, thinly to medium bedded, grey with dark grey and white horizontal and subvertical banding, moderately strong to 3.0m, very strong below 3.0m | | 2 | RUN | | | | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=0% | | | | | |
| | Subvertical to vertical joints from 3.0m to 3.8m. | | 3 | RUN | | | | | | | | | | | RUN 3# TCR=100%, SCR=100%, RQD=94%, UCS=115MPa | | | | | |
| | | | 4 | RUN | | | | | | | | | | | RUN 4# TCR=100%, SCR=100%, RQD=100%, UCS=119MPa | | | | | |
| 105.0 | | | | | | | | | | | | | | | | | | | | |
| 5.5 | END OF BOREHOLE AT 5.54m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION(m) 22/10/03 109.7 18/12/03 109.2 05/02/04 109.2 | | | | | | | | | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-8

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 215.0 E 313 737.2 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 07.10.03 - 07.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | | |
| 111.4 | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL, with rootlets (250mm) | | | | | | | | | | | | | |
| 111.2 | Black | | 1 | SS | 18 | | 111 | | | | | | | |
| 0.3 | Clayey SILT, with sand, trace gravel, occasional cobbles Very Stiff Brown Moist (TILL) | | 2 | SS | 20 | | 110 | | | | | | | |
| | | | 3 | SS | 22 | | 109 | | | | | | | |
| | | | 4 | SS | 18 | | 108 | | | | | | | |
| | | | 5 | SS | 68/ 279 | | 107 | | | | | | | |
| 107.9 | | | | | | | | | | | | | | |
| 3.6 | END OF BOREHOLE AT 3.56m. AUGER REFUSAL AT 3.56m PROBABLY ON BEDROCK OR BOULDERS. | | | | | | | | | | | | | |

RECORD OF BOREHOLE No DIV-9

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 208.9 E 313 735.9 (Division Street) ORIGINATED BY JL
 HWY HWY 17 BOREHOLE TYPE High Pressure Water Jetting and Vacuuming COMPILED BY SS
 DATUM Geodetic DATE 19.12.03 - 19.12.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 109.6 | | | | | | | | | | | | | | | | | |
| 0.0 | WATER | | | | | | | | | | | | | | | | |
| 109.3 | | | | | | | | | | | | | | | | | |
| 0.3 | Inferred silty CLAY , trace to some sand, trace rootlets Brown | | | | | | 109 | | | | | | | | | | |
| 108.7 | inferred cobbles at 0.7m to 0.9m | | 1 | GS | | | | | | | | | | | | | |
| 1.0 | END OF BOREHOLE AT 0.96m. BOTTOM OF BOREHOLE PROBED WITH A STEEL BAR, BEDROCK OR BOULDER INFERRED. REFER TO BOREHOLES DIV-7, DIV-8, DIV-11 FOR INFERRED SOIL STRATIGRAPHY. | | | | | | | | | | | | | | | | |

+ 3 . × 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-10

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 202.8 E 313 734.6 (Division Street) ORIGINATED BY JL
 HWY HWY 17 BOREHOLE TYPE High Pressure Water Jetting and Vacuuming COMPILED BY SS
 DATUM Geodetic DATE 19.12.03 - 19.12.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT Y 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 | | |
| 111.1 | | | | | | | | SHEAR STRENGTH kPa | | | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE | | | | | | |
| | | | | | | | | ● QUICK TRIAXIAL × LAB VANE | | | | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 100 | | |
| | | | | | | | | WATER CONTENT (%) | | | | | | |
| | | | | | | | | 20 | 40 | 60 | | | | |
| 0.0 | Inferred silty CLAY, trace to some sand, trace rootlets Brown | | | | | | 111 | | | | | | | |
| | | | 1 | GS | | | | | | | | | | |
| | | | | | | | 110 | | | | | | | |
| | inferred cobbles from 1.2m to 1.8m | | | | | | | | | | | | | |
| 109.3 | | | | | | | | | | | | | | |
| 1.8 | END OF BOREHOLE AT 1.83m. BOTTOM OF BOREHOLE PROBED WITH A STEEL BAR, BEDROCK OR BOULDER INFERRED. REFER TO BOREHOLES DIV-7, DIV-8, DIV-11 FOR INFERRED SOIL STRATIGRAPHY. | | | | | | | | | | | | | |

+³, ×³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-11

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 204.5 E 313 740.3 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|--|--|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | |
| 109.6 | | | | | | | | | | | | |
| 108.6 | TOPSOIL (100mm) | | | | | | | | | | | |
| 0.1 | Black | | 1 | SS | 13 | | | | | | | |
| | Silty CLAY, with rootlets, with sand seams | | | | | | | | | | | |
| | Stiff | | | | | | | | | | | |
| | Brown | | 2 | SS | 14 | | | | | | | |
| | Moist | | | | | | | | | | | |
| 108.2 | | | | | | | | | | | | |
| 1.5 | Clayey SILT, with sand, some gravel, some sand seams, occasional cobbles | | 3 | SS | 29 | | | | | | | |
| | Very Stiff to Hard | | | | | | | | | | | |
| | Brown | | | | | | | | | | | |
| | Moist | | | | | | | | | | | |
| | (TILL) | | 4 | SS | 62/254 | | | | | | | |
| 106.9 | | | | | | | | | | | | 14 46 27 13 |
| 2.7 | AUGER REFUSAL AT 2.69m. | | 1 | RUN | 1 | | | | | | | RUN 1# |
| | CRYSTALLINE LIMESTONE (BEDROCK) | | | | 0 | | | | | | | TCR=100%, |
| 106.3 | Fresh, slightly weathered at joints, | | | | | | | | | | | SCR=100%, |
| 3.3 | thinly bedded, grey with white subvertical banding, very strong. | | | | 2 | | | | | | | RQD=100%, |
| | GRAYWACKE | | | | 0 | | | | | | | UCS=122MPa |
| | Fresh, thinly bedded, dark grey to black with subvertical wavy white banding, very strong | | 2 | RUN | 2 | | | | | | | RUN 2# |
| 105.3 | Subvertical joint at 4.1m | | | | 1 | | | | | | | TCR=100%, |
| 4.3 | CRYSTALLINE LIMESTONE | | | | 1 | | | | | | | SCR=100%, |
| | Fresh, very strong | | | | | | | | | | | RQD=100%, |
| | Subvertical joint from 5.0m to 5.4m | | 3 | RUN | 0 | | | | | | | UCS=174MPa |
| | | | | | 1 | | | | | | | |
| 104.0 | | | | | 0 | | | | | | | |
| 5.6 | END OF BOREHOLE AT 5.61m. | | | | FI | | | | | | | |
| | THE BOREHOLE WAS FILLED WITH DRILL WATER UPON COMPLETION OF ROCK CORING. | | | | | | | | | | | |

RECORD OF BOREHOLE No DIV-12

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 212.2 E 313 785.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|-----------------------|----------------------------|-----------------|---|----|----|----|------------------------------------|-------------------------------------|-----------------------------------|---------------------|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | |
| 110.3 | | | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | GR SA SI CL |
| 110.8 | TOPSOIL (75mm) | | | | | | | | | | | | | | | |
| 0.1 | Black Silty CLAY, with sand seams, some rootlets and organics Stiff Brown to Dark Brown Moist | | 1 | SS | 11 | | | | | | | | | | | |
| | | | 2 | SS | 14 | | | | | | | | | | | |
| 108.8 | | | | | FI | | | | | | | | | | | |
| 1.5 | AUGER REFUSAL AT 1.49m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh, very thinly to thinly bedded, grey with dark grey and white subvertical banding, very strong | | 1 | RUN | 0 0 0 0 0 | | | | | | | | | | | RUN 1# TCR=99%, SCR=99%, RQD=99%, UCS=107MPa |
| | | | 2 | RUN | 2 1 0 0 0 | | | | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=98%, UCS=127MPa |
| 105.8 | | | | | 0 | | | | | | | | | | | |
| 4.6 | END OF BOREHOLE AT 4.55m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION(m) 22/10/03 108.9 18/12/03 108.9 05/02/04 108.8 | | | | | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-13

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 211.7 E 313 793.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|--|---|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | |
| 110.2 | | | | | | | | 20 | 40 | 60 | 80 | 100 | | |
| 110.0 | | | | | | | | 20 | 40 | 60 | 80 | 100 | | |
| 0.1 | TOPSOIL (75mm) Black Silty CLAY, some sand seams, occasional rootlets Stiff to Very Stiff Brown Moist | | 1 | SS | 8 | | 110 | | | | | | | |
| | | | 2 | SS | 16 | | 109 | | | | | | | |
| | | | 3 | SS | 17 | | | | | | | | | |
| 108.0 | | | | | | | 108 | | | | | | | |
| 2.3 | END OF BOREHOLE AT 2.29m. AUGER REFUSAL AT 2.3m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY ON COMPLETION. | | | | | | | | | | | | | |

RECORD OF BOREHOLE No DIV-14

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 204.5 E 313 791.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|--|--|--|--|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | | |
| | | | | | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L | | | | | | |
| | | | | | | | | WATER CONTENT (%) | | | | | | |
| | | | | | | | | | | | | | | |
| 110.6 | | | | | | | | | | | | | | |
| 110.5 | | | | | | | | | | | | | | |
| 0.1 | TOPSOIL (100mm) Black Clayey SILT, with topsoil Stiff Brown to Dark Brown Moist (FILL) | | 1 | SS | 13 | | 110 | | | | | | | |
| | | | 2 | SS | 11 | | | | | | | | | |
| 109.2 | | | | | | | | | | | | | | |
| 1.5 | Silty SAND, some gravel, occasional cobbles Dense Brown Moist (FILL) | | 3 | SS | 40 | | 109 | | | | | | 17 60 23 (SI+CL) | |
| 108.3 | | | | | | | | | | | | | | |
| 2.3 | END OF BOREHOLE AT 2.34m. AUGER REFUSAL AT 2.34m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY ON COMPLETION. | | | | | | | | | | | | | |

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-15

1 OF 1

METRIC

G.W.P. 647-82-00 LOCATION N 5 032 199.6 E 313 789.1 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 09.10.03 - 09.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | | |
| 110.4 | | | | | | | | | | | | | | |
| 110.8 | TOPSOIL (100mm) | | | | | | | | | | | | | |
| 0.1 | Black | | 1 | SS | 8 | | 110 | | | | | | | |
| | Silty CLAY, with topsoil, trace rootlets | | | | | | | | | | | | | |
| 109.7 | Firm | | | | | | | | | | | | | |
| 0.7 | Brown to Dark Brown | | | | | | | | | | | | | |
| | Moist (OL) | | 2 | SS | 32 | | | | | | | | | 5 49 31 14 |
| | Clayey SILT, with sand, trace gravel, occasional cobbles | | | | | | | | | | | | | |
| | Hard | | | | | | 109 | | | | | | | |
| | Brown | | | | | | | | | | | | | |
| | Moist | | 3 | SS | 66 | | | | | | | | | |
| 108.4 | (TILL) | | | | | | | | | | | | | |
| 2.0 | END OF BOREHOLE AT 2.01m. AUGER REFUSAL AT 2.01m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. | | | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DIV-16

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 199.7 E 313 797.5 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 09.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | |
|---------------|---|------------|---------|------|-------------|----------------------------|-----------------|---|--------------|----------|---------------------------------|-------------------------------------|-----------------------------------|--|--|-------------------|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | WATER CONTENT (%) |
| | | | | | | | | ○ UNCONFINED | + FIELD VANE | × | | | | | | |
| | | | | | | | | ● QUICK TRIAXIAL | × | LAB VANE | | | | | | |
| 110.2 | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 110.6 | TOPSOIL (100mm) | | | | | | | | | | | | | | | |
| 0.1 | Black | | 1 | SS | 7 | | | | | | | | | | | |
| | Silty CLAY, with topsoil | | | | | | | | | | | | | | | |
| 109.5 | Firm | | | | | | | | | | | | | | | |
| 0.7 | Brown to Dark Brown | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | |
| | Clayey SILT, with sand, trace gravel, occasional cobbles | | 2 | SS | 37 | | | | | | | | | | | |
| | Hard | | | | | | | | | | | | | | | |
| | Brown | | | | | | | | | | | | | | | |
| | Moist | | | | | | | | | | | | | | | |
| | (TILL) | | 3 | SS | 38 | | | | | | | | | | 5 53 29 14 | |
| 107.9 | | | | | FI | | | | | | | | | | | |
| 2.2 | AUGER REFUSAL AT 2.21m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with dark grey and white subvertical banding, very strong Subvertical joints at 2.3m, and 3.3m | | 1 | RUN | 2 0 0 | | | | | | | | | | RUN 1# TCR=100%, SCR=100%, RQD=89%, UCS=120MPa | |
| | | | | | 4 0 0 | | | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=93%, UCS=123MPa | |
| | | | 2 | RUN | 0 0 0 | | | | | | | | | | | |
| | | | | | 0 | | | | | | | | | | | |
| | | | | | 0 | | | | | | | | | | | |
| | | | 3 | RUN | 0 1 | | | | | | | | | | RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=121MPa | |
| 104.9 | | | | | | | | | | | | | | | | |
| 5.3 | END OF BOREHOLE AT 2.01m. THE BOREHOLE WAS FILLED WITH DRILL WATER UPON COMPLETION OF ROCK CORING. | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No DIV-17

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 032 204.5 E 313 811.4 (Division Street) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 08.10.03 - 08.10.03 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | | |
| 110.4 | | | | | | | | | | | | | | |
| 110.0 | TOPSOIL (125mm) | | | | | | | | | | | | | |
| 0.1 | Black Silty CLAY, some topsoil, with sand seams Very Stiff Brown to Dark Brown Moist | | 1 | SS | 16 | | 110 | | | | | | | |
| | | | 2 | SS | 15 | | | | | | | | | |
| 108.9 | | | | | | | 109 | | | | | | | |
| 1.5 | Clayey SILT, with sand, and gravel, occasional cobbles Hard Brown Moist (TILL) cobbles from 2.3m to 2.4m | | 3 | SS | 30 | | | | | | | | | |
| | | | 1 | GS | | | 108 | | | | | | | |
| 107.4 | | | | | | | | | | | | | | |
| 3.0 | END OF BOREHOLE AT 3m. AUGER REFUSAL AT 3m PROBABLY ON BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. | | | | | | | | | | | | | |

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

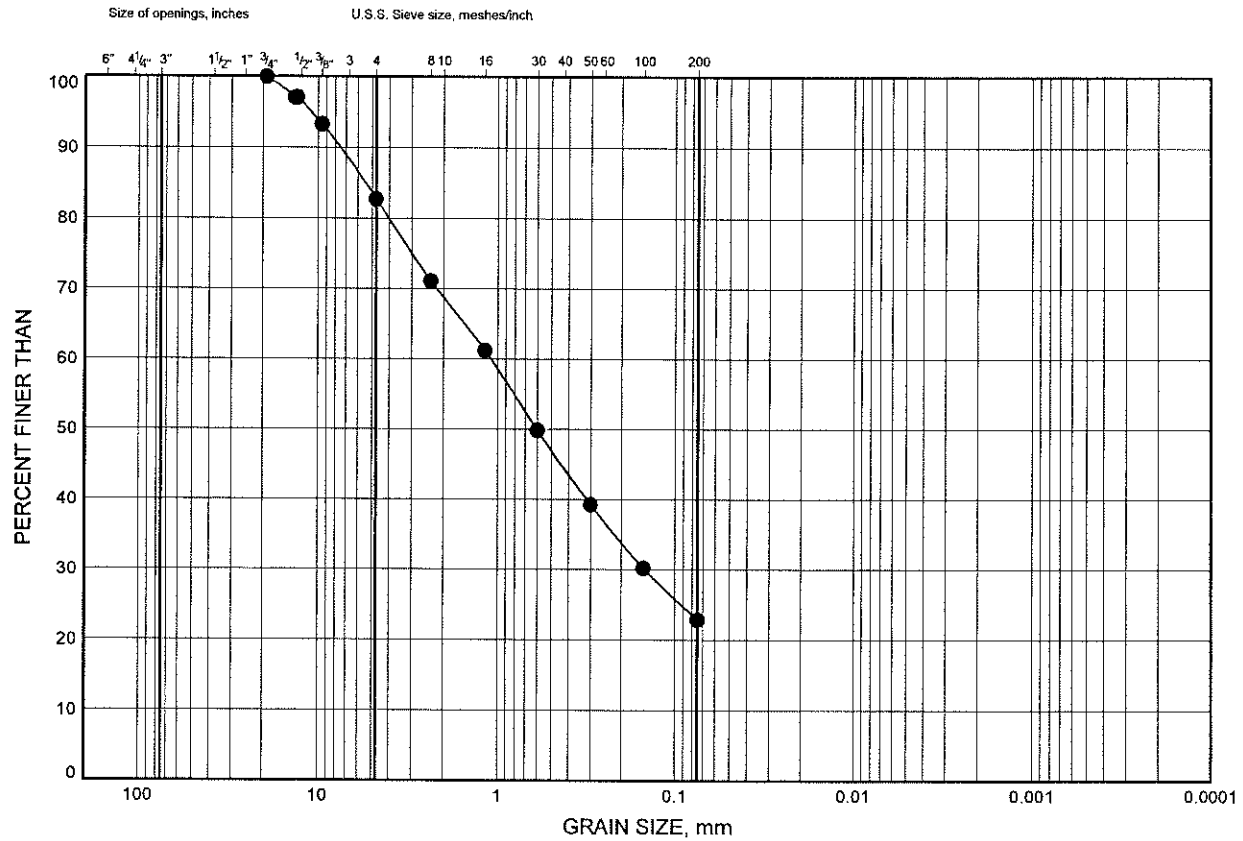
Appendix B

Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

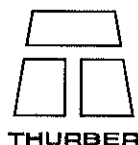
SILTY SAND (FILL)



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

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|--------|-----------|-----------|
| ● | DIV-14 | 1.83 | 108.78 |

Date February 2004
Project 647-92-00

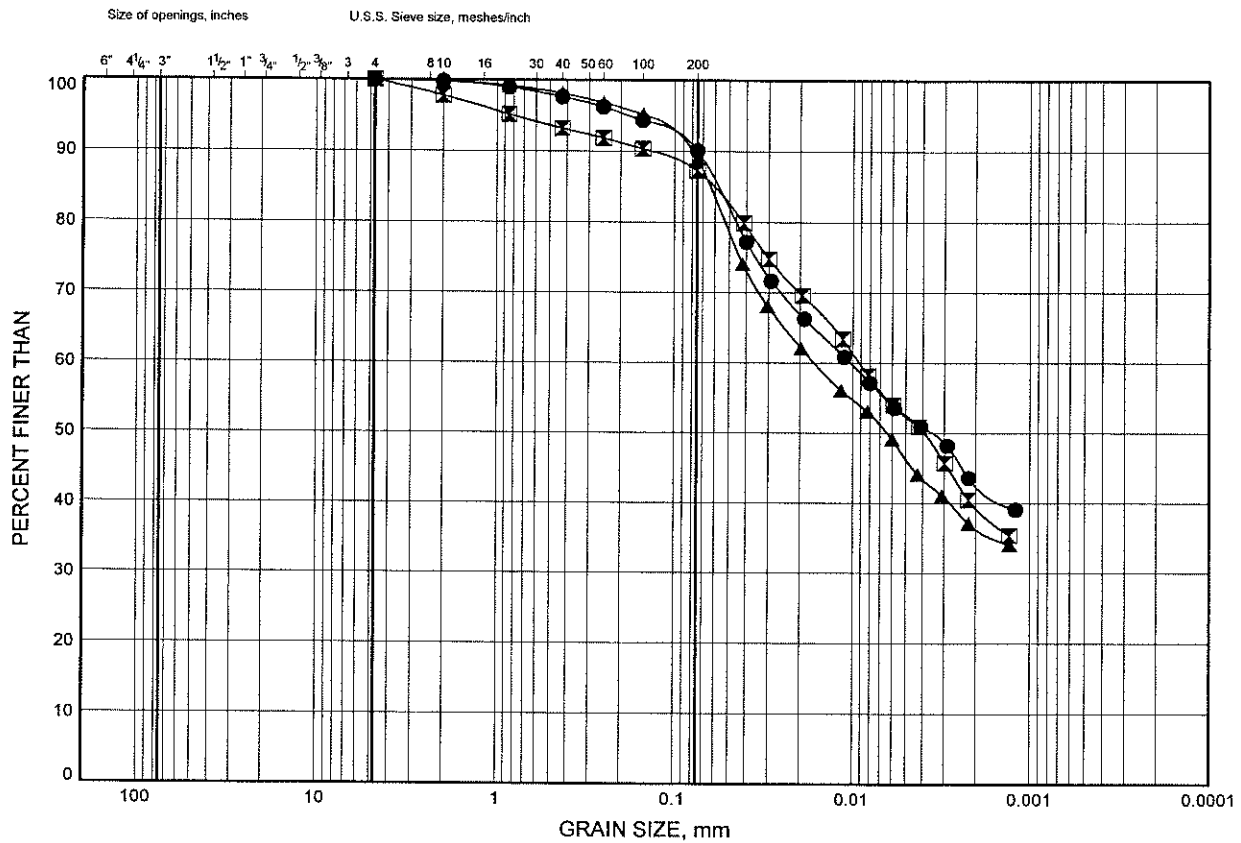


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

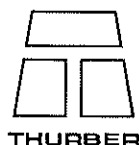
SILTY CLAY



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

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|--------|-----------|-----------|
| ● | DIV-1 | 1.07 | 110.97 |
| ⊠ | DIV-13 | 1.83 | 108.41 |
| ▲ | DIV-5 | 1.83 | 109.87 |

Date February 2004
Project 647-92-00

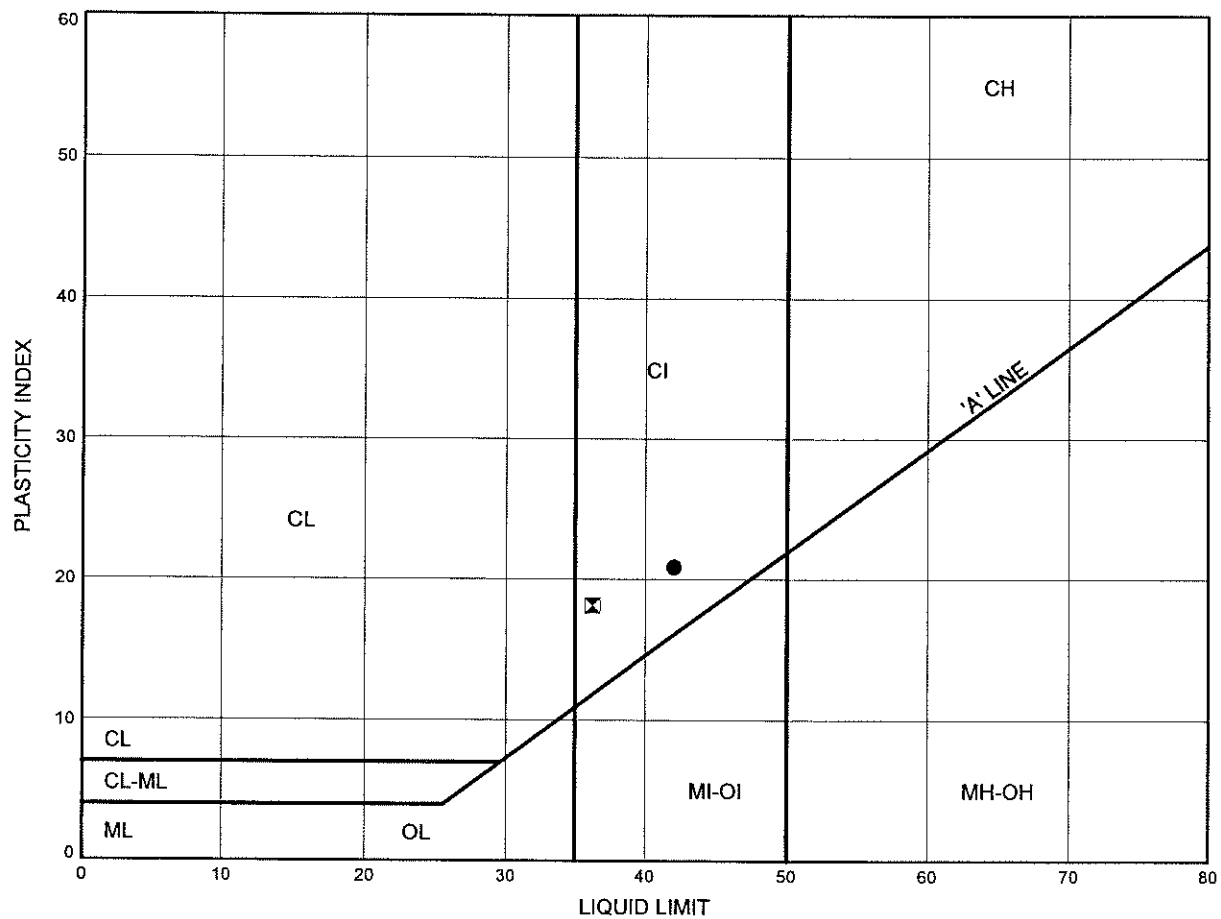


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

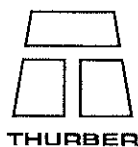
FIGURE B3

SILTY CLAY



| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|--------|-----------|-----------|
| ● | DIV-1 | 1.07 | 110.97 |
| ⊠ | DIV-13 | 1.83 | 108.41 |

Date February 2004
 Project 647-92-00

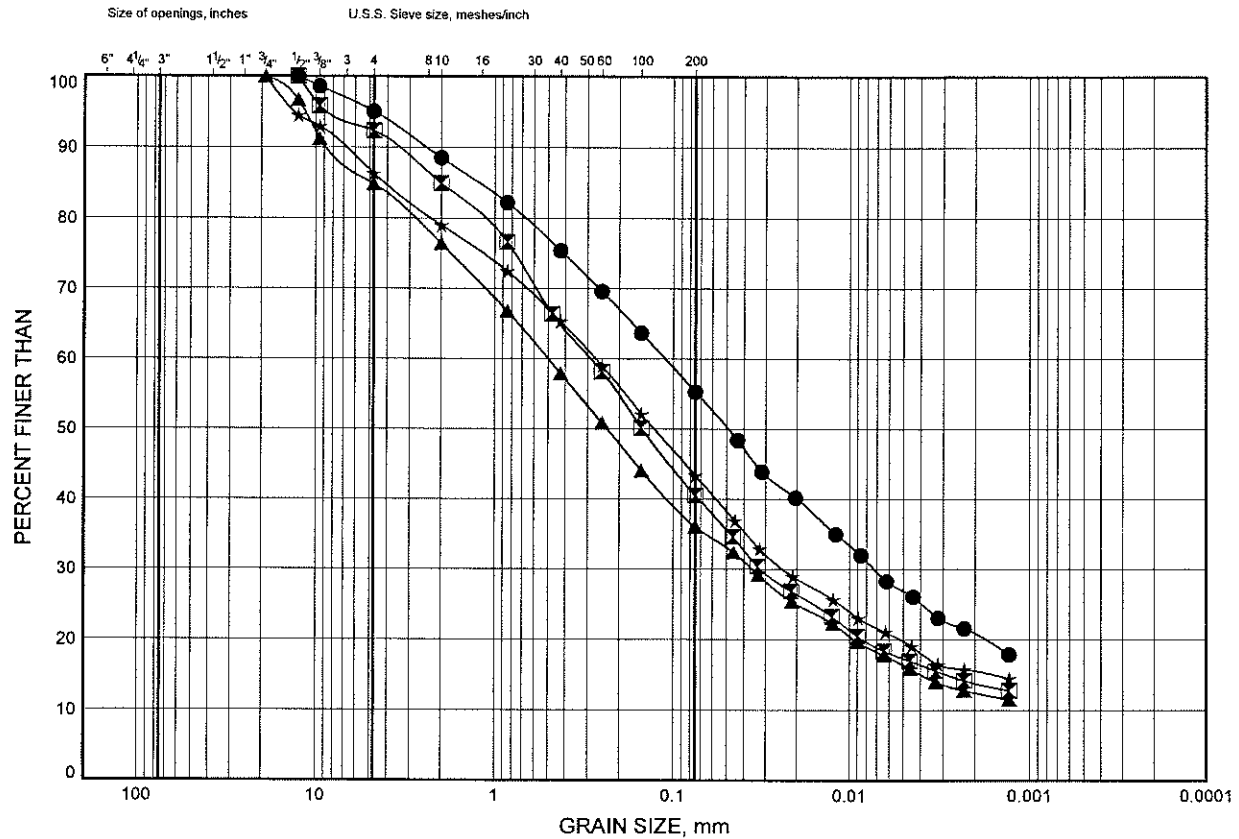


Prep'd SS
 Chkd SKP

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B4

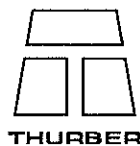
CLAYEY SILT TILL



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

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|-------|-----------|-----------|
| ● | DIV-2 | 1.83 | 110.04 |
| ⊠ | DIV-4 | 1.07 | 110.41 |
| ▲ | DIV-6 | 2.59 | 108.73 |
| ★ | DIV-7 | 1.83 | 108.72 |

Date February 2004
Project 647-92-00

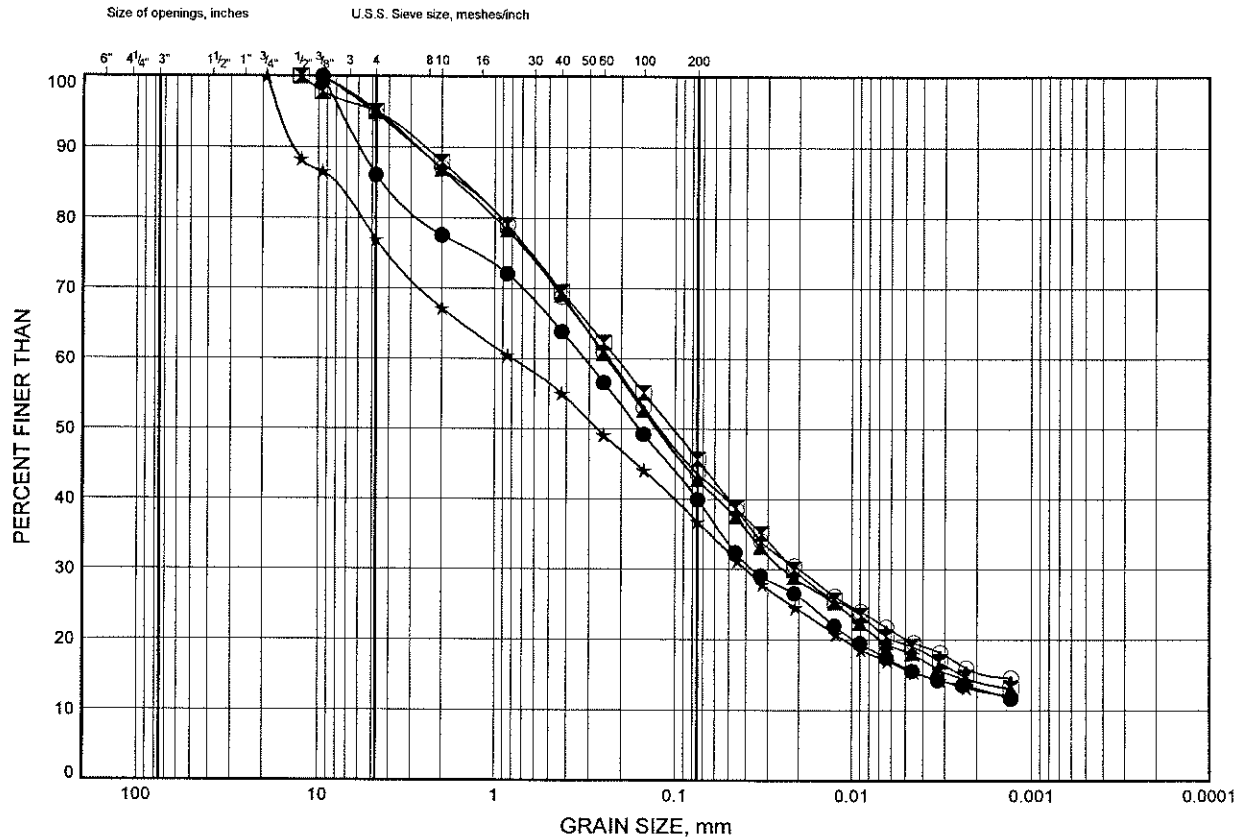


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B5

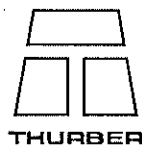
CLAYEY SILT TILL



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

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|--------|-----------|-----------|
| ● | DIV-11 | 2.44 | 107.16 |
| ⊠ | DIV-15 | 1.07 | 109.33 |
| ▲ | DIV-16 | 1.83 | 108.32 |
| ★ | DIV-17 | 1.83 | 108.56 |
| ⊙ | DIV-8 | 2.59 | 108.83 |

Date February 2004
Project 647-92-00

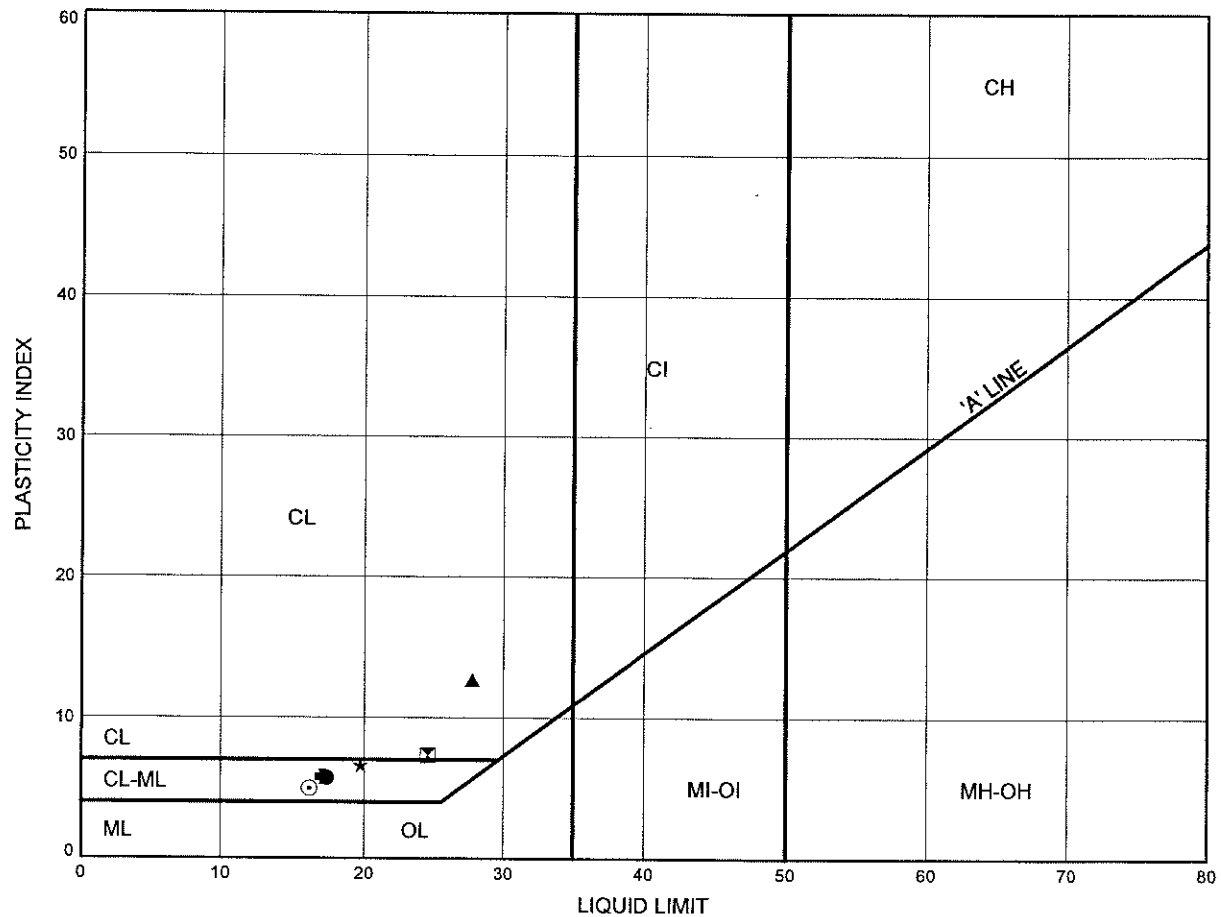


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

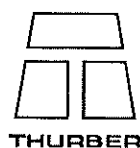
FIGURE B6

CLAYEY SILT TILL



| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|--------|-----------|-----------|
| ● | DIV-11 | 2.44 | 107.16 |
| ⊠ | DIV-17 | 1.83 | 108.56 |
| ▲ | DIV-2 | 1.83 | 110.04 |
| ★ | DIV-4 | 1.07 | 110.41 |
| ⊙ | DIV-7 | 1.83 | 108.72 |
| ⊛ | DIV-8 | 2.59 | 108.83 |

Date February 2004
 Project 647-92-00



Prep'd SS
 Chkd. SKP

Appendix C

Foundation Comparison

Division Street Underpass
Highway 17 Twinning, Amprior to Renfrew

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

| Foundation Element | Driven Piles | Footing on Bedrock | Footing on Engineered Fill | Caisson |
|--------------------|---|--|--|---|
| West Abutment | <p>Advantages: N/A</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Proximity of bedrock surface to the underside of the girders and shallow overburden effectively precludes the use of driven piles | <p>Advantages:</p> <ul style="list-style-type: none"> i. Bedrock comparatively close to the underside of the girders. ii. High values of geotechnical resistance are available on the bedrock <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Stepped footing may be required ii. High cost of excavation, if any is required iii. Mass concrete fill required to create a level founding surface. iv. Comparatively longer abutment stem. | <p>Advantages:</p> <ul style="list-style-type: none"> i. Possibility of shortening the abutment height <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than bedrock ii. Footing has to be located further back to accommodate forward slope of fill | <p>Advantages:</p> <ul style="list-style-type: none"> i. High bearing capacity on bedrock. ii. Possibility of shortening the abutment height. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Some rock coring is required to seat caisson into bedrock. ii. Engineered fill pad is required as a platform to accommodate caisson installation equipment. |

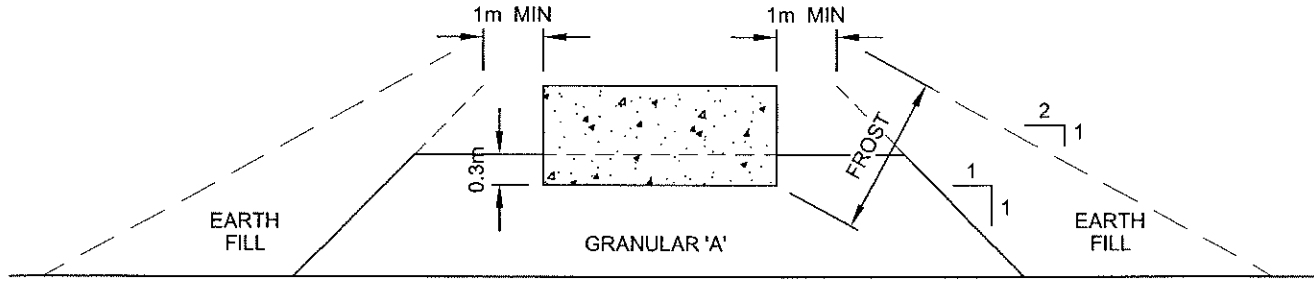
Division Street Underpass
Highway 17 Twinning, Arnprior to Renfrew

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

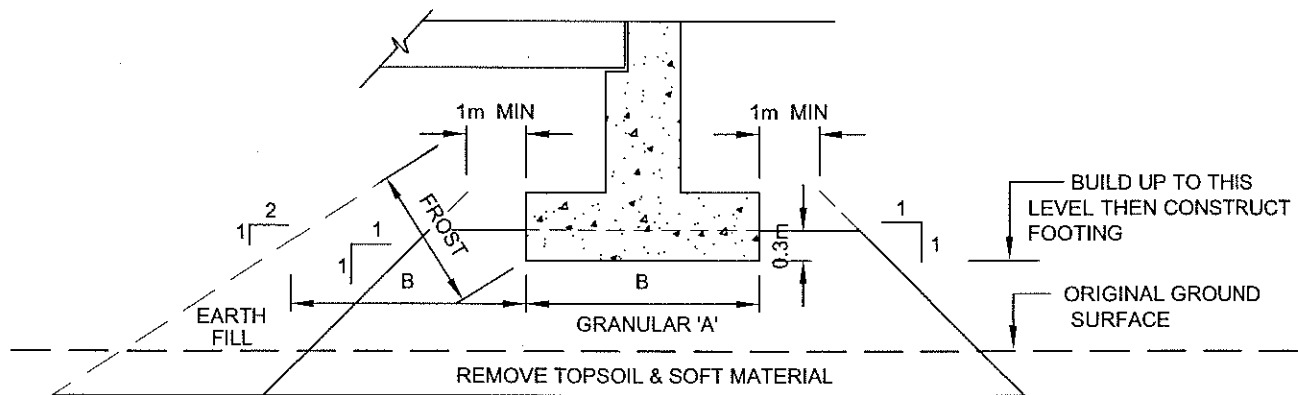
| Foundation Element | Driven Piles | Footing on Bedrock | Footing on Engineered Fill | Caisson |
|----------------------|---|---|---|--|
| Centre Pier | <p>Advantages: N/A</p> <p>Disadvantages: i. Proximity of bedrock surface to the underside of the girders and shallow overburden effectively precludes the use of driven piles</p> | <p>Advantages: i. Bedrock close to the surface in the median. ii. High values of geotechnical resistance are available on the bedrock</p> <p>Disadvantages: i. Stepped footing may be required ii. High cost of excavation, if any is required</p> | <p>Advantages: i. None identified</p> <p>Disadvantages: i. Lower geotechnical resistance than bedrock.</p> | <p>Advantages: None identified</p> <p>Disadvantages: i. Shallow bedrock depth below ground surface precludes use of caissons.</p> |
| East Abutment | <p>Advantages: N/A</p> <p>Disadvantages: i. Proximity of bedrock surface to the underside of the girders and shallow overburden effectively precludes the use of driven piles</p> | <p>Advantages: i. High values of geotechnical resistance are available on the bedrock.</p> <p>Disadvantages: i. Stepped footing may be required ii. High cost of excavation, if any is required. iii. Comparatively longer abutment stem. iv. Mass concrete fill required to create a level founding surface.</p> | <p>Advantages: i. Possibility of shortening the abutment height</p> <p>Disadvantages: i. Lower geotechnical resistance than bedrock ii. Footing has to be located further back to accommodate forward slope of fill</p> | <p>Advantages: i. High bearing capacity on bedrock. ii. Possibility of shortening the abutment height</p> <p>Disadvantages: i. Some rock coring is required to seat caisson into bedrock. ii. Engineered fill pad is required as a platform to accommodate caisson installation equipment.</p> |

Appendix D

Figures



CROSS-SECTION



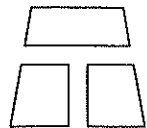
LONGITUDINAL SECTION

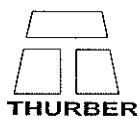
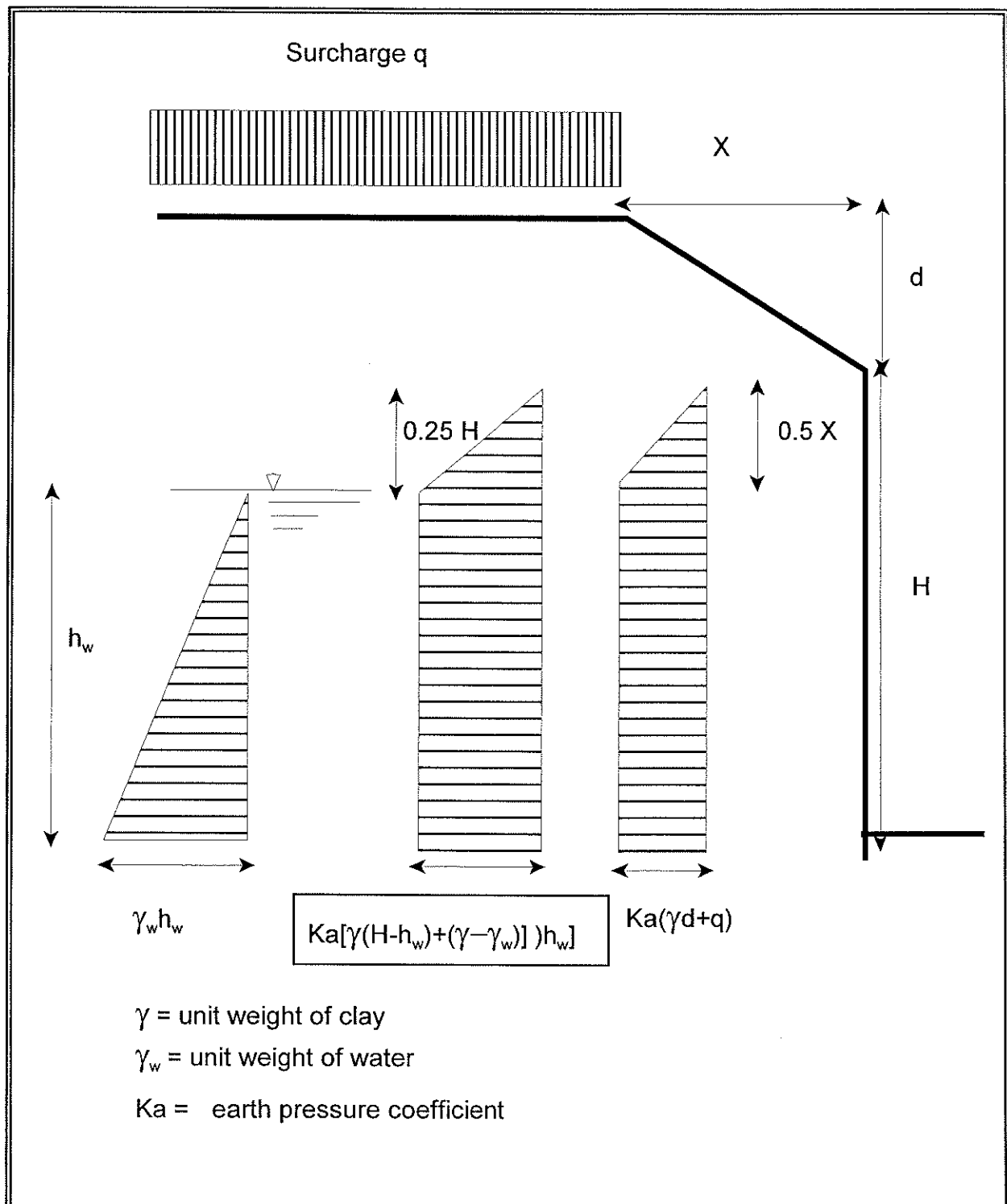
NOT TO SCALE

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.

TED35146.DWG

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



**LATERAL PRESSURE DISTRIBUTION FOR
BRACED SHORING DESIGN**

FIGURE D2

Appendix E

Special Provisions

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

Special Provision No. 902S01

September 2003

Excavation and Backfilling-Structures

902.02 REFERENCES

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

OPSS 510

902.03 DEFINITIONS

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

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

902.04 SUBMISSION AND DESIGN REQUIREMENTS

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

902.04.01 Site Survey

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

902.04.02 Working Drawings

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

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

902.04.03 Submission of Certificate of Conformance

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

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

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

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

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

902.05.04 Protection System

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

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

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

902.07.02 Excavation

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

902.07.02.01 General

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

902.07.02.02 Excavation for Foundation

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

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

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

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

902.07.02.03 Excavation for Backfill and Frost Tapers

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

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

902.07.02.04 Preservation of Channel

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

902.07.02.05 Removals

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

902.07.03 Unwatering Structure Excavation

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

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

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

902.07.04 Backfilling

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

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

Backfilling shall be according to OPSS 501.

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

902.09 Measurement for Payment

902.09.01 Structures

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

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

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

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

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

902.10 Basis of Payment

902.10.01 Excavation and Backfill

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

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

WARRANT: Always with these tender items.

AMENDMENT TO OPSS 120, AUGUST, 1994

Special Provision

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

Construction Special Provision for Rock Excavation Utilizing Blasting

120.01 SCOPE

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

120.02 REFERENCES

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

Canadian Standards Association:

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

Ministry of Transportation Publications:

Ontario Traffic Manual Book 7

Federal Government Publication:

Explosives Act (Canada)

Department of Fisheries and Oceans Publication:

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

120.03 DEFINITIONS

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

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

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

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

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

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

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

120.04 SUBMISSION AND DESIGN REQUIREMENTS

120.04.01 General

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

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

120.04.02 Blasting Design and Monitoring

120.04.02.01 Blasting Consultant

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

120.04.02.02 Blasting Design

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

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

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

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

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

120.04.02.03 Blasting Monitoring

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

The blasting monitoring equipment shall meet the following requirements:

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

120.04.03 Blasting Permits, Approvals and/or Agreements

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

120.04.04 Pre-Blast Survey

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

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

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

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

120.04.05 Trial Blasting

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

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

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

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

120.04.06 Post Blast Survey and Reporting

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

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

120.04.07 Certificate of Conformance

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

120.05 MATERIAL

120.05.01 Explosives

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

120.05.02 Handling and Storage

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

120.06 EQUIPMENT

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

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

120.07 CONSTRUCTION

120.07.01 General

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

120.07.02 Safety Precautions

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

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

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

120.07.03 Notice

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

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

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

120.07.04 Vibration Monitoring

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

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

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

Structures

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

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

Fresh Concrete

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

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

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

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

120.07.05 Utilities

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

120.07.06 Excessive Vibration Readings – Work Stoppage

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

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

AMENDMENT TO OPSS 206, DECEMBER 1993

Special Provision

November 25, 2002

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

206.01 SCOPE

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

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

206.04 SUBMISSION AND DESIGN REQUIREMENTS

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

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

206.06 EQUIPMENT

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

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

206.07 CONSTRUCTION

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

206.07.01.03 Compaction

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

206.07.01.03.01 Compaction of Earth Embankments

Compaction of earth materials shall conform to OPSS 501.

206.07.01.03.02 Compaction of Rock Embankments

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

206.07.05 Rock Excavation, Grading

206.07.05.01 General

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

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

206.07.08 Rock Embankments

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

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

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

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

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

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

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

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

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

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

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

Appendix F

Drawing

