

Geocres No:  
31E-203

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
STRONG TWP 10/11 (ADAMS RD) OVERPASS, NBL  
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER  
ONTARIO  
G.W.P. 759-93-01, W.P. 745-93-01, SITE 44-417**

**Geocres Number: 31E-203**

**Report to**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation at the site of the Strong Twp 10/11 (Adams Rd) Overpass, to carry re-aligned Highway 11 NBL over the Strong Twp 10/11 Road near Sundridge, Ontario. A previous, preliminary investigation had been carried out at the site by Golder Associates Ltd. (Golder) and the factual data from that investigation has been incorporated into 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, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed through considering a combination of the data from the previous Golder investigation and the data obtained in the course of the present investigation.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

**2 SITE DESCRIPTION**

The site is located at the intersection of the new alignment of Highway 11 NBL and Strong Twp 10/11 (Adams Rd), Strong Township Concession 10/11, Parry Sound District. This location is on the existing Strong Twp 10/11 less than 1 km west of the Sundridge municipal limits.

The site is located in an area of rolling terrain characterized by bedrock outcrops and shallow glacial drift. The bedrock is typically Precambrian granitic gneiss of the Canadian Shield and the shallow overburden soils are typically non-cohesive outwash or basal till deposits.

Drainage in the area is typically poor and there are extensive, though generally shallow, deposits of organic soils in low-lying areas. The immediate bridge site is not affected by these organic deposits.

The area is generally heavily wooded and development is sparse, though a senior citizens residence is located a short distance west of the site.

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this site were carried out between December 17, 2003 and February 11, 2004. The site investigation consisted of drilling and sampling a total of ten boreholes numbered Borehole 417-11 to Borehole 417-20. A further set of ten boreholes were drilled for the adjacent SBL structure and the logs of these boreholes are included in Appendix A for reference. In addition to the boreholes drilled by Thurber, four boreholes drilled for each structure by Golder as part of the preliminary design assignment were also used in the analysis. Golder's boreholes are numbered Borehole 10-1 through Borehole 10-4 for the SBL and 10-5 through 10-8 for the NBL and are included in Appendix C. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing presented in Appendix F.

The positions of the boreholes relative to the SBL structure site are as shown in Table 3.1.

**Table 3.1 – Borehole Locations Relative to Structure**

<b>Location on Structure</b>	<b>Boreholes Considered in Design</b>
South Approach	BH 10-5*
South Abutment	BH 417-11, 417-12, 413-13, 417-14, 417-15, 10-6*
North Abutment	BH 417-16, 417-17, 417-18, 417-19, 417-20, 10-7*
North Approach	BH 10-8*

\* Boreholes drilled by Golder in 2000

Since the results of the preliminary investigation showed shallow bedrock, the borehole pattern was selected to place six sampled boreholes in each proposed abutment footprint plus one sampled borehole in the area of each approach fill. The boreholes penetrated 2.7 to 3.7 m of earth at the south approach and south abutment and 3.3 m to 4.9 m at the north approach and north abutment. Three boreholes within each foundation footprint were advanced 2.1 to 3.9 m into bedrock by coring and the remainder were terminated at effective auger refusal on assumed bedrock.

Prior to any drilling being carried out, the borehole locations were marked in the field by surveyors from Marshall Macklin Monaghan Ltd. and utility clearances were obtained by Thurber.

Malone's Soil Samples Co. Ltd. of Etobicoke, Ontario supplied a CME 55 drill rig mounted on a Bombardier tracked carrier and conducted most of the drilling, sampling and in-situ testing operations. The final round of site investigation and field testing was carried out by a George Downing Estate Drilling Ltd. using a CME 75 drill rig mounted on a Nodwell tracked carrier.

Hollow stem and solid stem auger drilling techniques were used, as appropriate, to advance the boreholes through the overburden and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ diamond coring techniques were used to

advance the selected three boreholes within each of the foundation elements to depths of 2.1 to 3.9 m into bedrock.

Standpipe piezometers, consisting of 19 mm PVC pipe with slotted tips, were installed in selected boreholes to monitor the groundwater levels. The locations and completion details for the piezometers are shown in Table 3.2.

**Table 3.2 – Piezometer Details**

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation	Completion Details
BH 417-15	5.6/358.9	Piezometer with 1.5 m tip installed at 5.6 m, bottom of borehole at 5.6 m. Native sand collapsed to 2.7 m, bentonite seal from 2.7 to 1.8 m, cave to 0.3 and bentonite seal installed at 0.3 m to surface.
BH 417-20	8.2/356.0	Piezometer with 1.5 m tip installed at 8.2 m, bottom of borehole at 8.2 m. Native sand collapsed to 0.3 and bentonite seal installed at 0.3 m to surface.
BH 10-7	7.2 m $\pm$	Piezometer installed by Golder

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 the recovered samples and processed the samples for transport to Thurber's Oakville office.

On completion of drilling and sampling, boreholes not containing piezometers were backfilled using drill cuttings.

#### **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 (sieve test). Since no cohesive soils were encountered, no other laboratory testing was deemed necessary. Nine samples out of a total of thirty-two samples were selected for these tests and the results are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

Laboratory test results contained in the preliminary report by Golder were also used in the analysis and are included in Appendix C.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets in Appendix A and to the Record of Borehole sheets prepared by Golder and included in Appendix C. Details of the encountered soil and rock

stratigraphy are presented in these appendices and on the “Borehole Locations and Soil Strata” Drawing presented in Appendix F. A description of the stratigraphy is given in the following paragraphs.

In general terms, the site was found to be underlain by peat and a relatively thin deposits of sand and sandy silt overlying Pre-Cambrian bedrock.

### 5.1 Peat (Topsoil)

Dark brown, moist, fine fibrous peat was encountered in some of the boreholes in depths of 250 to 760 mm as shown in Table 5.1.

**Table 5.1 – Peat Thicknesses**

Borehole	Peat Thickness (mm)	Borehole	Peat Thickness (mm)
417-11	0	417-18	300
417-12	300	417-19	500
417-13	0	417-20	300
417-14	0	10-5	300
417-15	400	10-6	300
417-16	300	10-7	760
417-17	400	10-8	180

Occasional boulders occur in or immediately below the peat and on the surface.

### 5.2 Road Fill

Although not sampled in the course of this investigation, sand and gravel fill should be expected in the existing Adams Road pavement structure. In view of the geographic location and past road building practices, wood corduroy may occur below the fill. The gravelly sand identified in Golder’s Boreholes 10-2 and 10-3 may represent the fill present in the existing shoulders.

### 5.3 Sand

A layer of sand was encountered below the peat, or from the surface where peat was absent. The sand is typical of a thin veneer of outwash material deposited on top of the bedrock at the end of the last glaciation. The composition of the sand is variable across the site.

The description of the soil is sand, trace to some silt. Near the surface, some samples contained occasional rootlets and occasionally an organic odour and decaying wood were detected to 2.2 m depth in Borehole 10-7.. The sand is described as being in a very loose to dense state, based on SPT values ranging from 3 to 34 blows for 0.3 m of penetration.

However, most results indicate a compact condition. A few very high SPT values were encountered near the base of the layer but these are considered to be caused by contact with the bedrock, a boulder or rock shatter immediately overlying the bedrock rather than being indicative of a very dense soil matrix. Cobbles and boulders may be found in the overburden at this site.

The sand was brown and generally wet, with natural moisture contents ranging from 15 to 21%, with a few higher values apparently due to organic content near the surface. Sand noted in Boreholes 10-7 and 10-8 was organic and blackish-brown in colour. The wet conditions result from the low lying nature of the site and the poor drainage conditions presented by the bedrock.

The thickness of the layer of sand was found to be approximately 1.8 to 3.1 m at the south approach and approximately 2.3 to 4.0 m thick at the north abutment. The sand was underlain by bedrock at Boreholes 417-13, 417-14, 417-16, 417-17 and 417-20.

The elevation of the underside of the sand ranged from a mean of 362.1 at the south abutment to a mean of 361.2 at the north abutment.

Grain size distribution curves for selected samples are shown in Figure B1 in Appendix B.

#### **5.4 Sandy Silt**

The sand is underlain by sandy silt at Boreholes 417-11, 417-12, 417-15, 417-18, 417-19, 10-5, 10-6, 10-7 and 10-8.

The silty sand was brown and wet, with natural moisture contents ranging from 16 to 19%. SPT values recorded in the sandy silt stratum ranged from 9 to 37 blows for 0.3 m of penetration, indicating loose to dense conditions. A few very high SPT values were encountered near the base of the layer but these are considered to be caused by contact with the bedrock, a boulder or rock shatter immediately overlying the bedrock rather than being indicative of a very dense soil matrix. Cobbles and boulders may be found in the overburden at this site.

The elevation of the underside of the sandy silt ranged from a mean of 361.2 at the south abutment to a mean of 359.9 at the north abutment.

#### **5.5 Bedrock**

The soils described above were found to be underlain by Pre-Cambrian Canadian Shield bedrock. The bedrock was proved by coring in Boreholes 417-11, 417-15, 417-16, 417-20, 10-6 and 10-7. The bedrock surface was inferred from refusal to auger penetration in the remainder of the boreholes drilled at this site.



Both abutments were found to be underlain by both granite and gneiss bedrock. Both are hard, competent rocks that will provide good founding conditions and are described below simply as one unit of bedrock.

Where granite was encountered, the rock is massive, pink and has black speckling. The gneiss is also massive, pink, but with thin black banding.

Core recovery in the bedrock was generally 80 to 100% and the RQD values ranged from 55% (near the surface) to 100% and for the most part lay between 80% and 100%, indicating good to excellent rock quality. Most of the lower RQD values were encountered at the north abutment.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally found to range from 0 to 5.

The strength of the intact rock is assessed to exceed 170 to 230 MPa on the basis of point load testing. The rock is classed as very strong to extremely strong.

The elevations of the top of bedrock are shown in Table 5.2.

**Table 5.2 – Elevation of Top of Bedrock**

<b>Borehole</b>	<b>Elevation of Bedrock</b>	<b>Borehole</b>	<b>Elevation of Bedrock</b>
417-11	361.9	417-18	359.7
417-12	361.7	417-19	359.9
417-13	361.7	417-20	360.0
417-14	361.4	10-5	361.3
417-15	360.8	10-6	360.3
417-16	361.3	10-7	359.6
417-17	361.1	10-8	361.4

## **5.6 Water Levels**

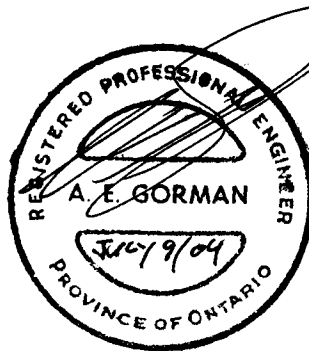
The groundwater level in the sand is at or close to the surface and a small artesian head was recorded in the bedrock in by Golder in Borehole 10-2 at the SBL structure. The observed groundwater levels in piezometers are as shown in Table 5.2.

Table 5.2 – Groundwater Levels

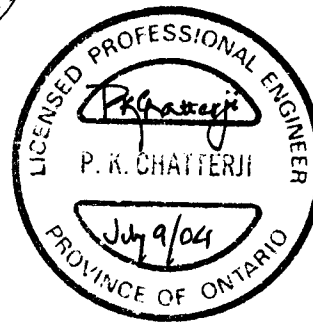
Date	Depth/Elevation of Groundwater		
	BH 417-15	BH 417-20	BH 10-7
February 14, 2000	-	-	1.1/363.4
March 8, 2000	-	-	+1.0/365.5
March 26, 2000	-	-	+1.0/365.5
February 12, 2004	-	+0.5/364.8	-
March 11, 2004	Buried in snow	0.0/364.3	-

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

Direction of Fieldwork and Engineering Analysis  
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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 INTRODUCTION**

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

The proposed structure will consist of a single-span overpass with a span in the order of 36 m, and abutments square to the centreline of the structure.

It is understood that the Strong Twp Road 10/11 (Adams Road) will remain close to the present grade and that Highway 11 will cross on an embankment. The existing ground elevations, proposed Highway 11 grades and resulting embankments heights at the abutment locations are shown in Table 6.1.

**Table 6.1 – Grades and Embankment Height**

<b>Location</b>	<b>Existing Ground Elevation</b>	<b>Proposed Highway 11 Grade</b>	<b>Embankment Height</b>
South Abutment	364.7	374.3	9.6 m
North Abutment	364.5	374.8	10.3 m

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

**7 STRUCTURE FOUNDATIONS**

The proposed bridge for this site will consist of a single-span overpass structure with two abutment foundations.

The elevations at which bedrock was encountered at the two foundation elements are shown in Table 7.1.



**Table 7.1 – Original Ground and Bedrock Elevations**

Location	BH Number	Elevations	
		Original Ground	Top of Bedrock
<b>South Abutment</b>			
NW Corner	417-12	365.1	361.7
NE Corner	417-15	364.5	360.8
Interior West	417-13	364.6	361.7
Interior East	10-6	364.0	360.3
SW Corner	417-11	365.0	361.9
SE Corner	417-14	364.5	361.4
<b>MEAN</b>		<b>364.6</b>	<b>361.3</b>
<b>North Abutment</b>			
NW Corner	417-17	364.6	361.1
NE Corner	417-20	364.3	360.0
Interior West	417-18	364.5	359.7
Interior East	10-7	364.5	359.6
SW Corner	417-16	364.6	361.3
SE Corner	417-19	364.4	359.9
<b>MEAN</b>		<b>364.5</b>	<b>360.3</b>

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
- Caissons (drilled shaft piles)

A comparison of the foundation alternatives, in tabular form, based on advantages and disadvantages of each is included in Appendix D.

### 7.1 Driven piles

The stratigraphy encountered at the site consists of comparatively thin overburden deposits overlying bedrock. With the Highway 11 grade lying approximately 10 m above original ground level, the underside of an abutment stem would lie approximately 2 to 3 m above the original ground level.

The configuration of the structure and the bedrock elevations determined at the boreholes result in piles at the south abutment being in the order of 6 to 8 m long and piles at the

north abutment being 7 to 10 m long. These pile lengths are considered to be sufficient for the design of integral abutments.

In the event that a pile reaches refusal on bedrock at an elevation that results in a pile length less than 5 m, the Contractor should proceed as follows:

1. Drill into the bedrock for sufficient depth to provide a minimum pile length of 5.5 m
2. Place the pile in the rock socket and place 300 mm concrete to pin the bottom of the pile.
3. Backfill around the pile with sand as specified for integral abutments.
4. Continue with the normal construction sequence.

#### 7.1.1 Geotechnical Resistance

The axial geotechnical resistances at ULS for four selected pile sections when driven to bedrock are as follows:

- 2,000 kN for HP 310 X 110
- 2,400 kN for HP 310 X 132
- 2,750 kN for HP 310 X 152
- 2,400 kN for HP 360 X 132

All the foregoing recommended values are for vertical, concentric geotechnical resistance.

The structural resistance of the pile should be checked by the structural designer.

#### 7.1.2 Pile Tips

All piles must be reinforced with driving shoes in accordance with SS 103-12.

#### 7.1.3 Pile Installation

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

#### 7.1.4 Pile Driving

The appropriate note for the foundation drawing is Note 5, i.e. "Piles to be driven to bedrock".

#### 7.1.5 Downdrag

The piles at this site will be placed in compacted granular fill. Accordingly, downdrag is not an issue.

### 7.1.6 Lateral Resistance of Piles

The lateral resistance of the piles may be calculated using a value for the modulus of horizontal subgrade reaction ( $k_s$ ) given by:

$$k_s = n_h z / D$$

where:

$z$  = depth below ground surface in metres

$D$  = pile width in metres

$n_h$  = (kN/m<sup>3</sup>) a factor from Table 7.2

**Table 7.2 – Parameters for Lateral Pile Resistance**

Location	Elevation	$n_h$ (kN/m <sup>3</sup> )
North Abutment	Fill	6,000
	OGL to 361 or bedrock	1,500
South Abutment	Fill	6,000
	OGL to 362 or bedrock	1,500

The modulus of subgrade reaction may have to be reduced, based on the pile spacing.

The following reduction factors should be used for a pile group oriented *perpendicular* to the direction of loading.

Pile spacing	Reduction Factor
4b	1.00
1b	0.5

The following reduction factors should be used for a pile group oriented *parallel* to the direction of loading.

Pile spacing	Reduction Factor
8b	1.00
6b	0.7
4b	0.4
3b	0.25

--- where "b" is the breadth of the pile, spacing is centre to centre

Intermediate values may be obtained by linear interpolation.

In the case of conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

## **7.2 Spread Footings on Bedrock**

The presence of bedrock at a comparatively shallow depth makes support of the structure on footings bearing on the bedrock feasible from a geotechnical standpoint.

The bearing elevations for footings on bedrock are the elevations of the top of bedrock given in Table 7.1. In practical terms, the underside of the concrete footing should be designed to lie approximately 200 mm above the local top of rock, the difference in elevation made up using mass concrete fill. Typically, the footing may be stepped down across the width of the structure to accommodate changes in the elevation of the top of bedrock. This approach will help reduce the risk that the contractor will have to excavate bedrock under a footing.

The top surface of the bedrock should be stripped of all overburden and should be cleaned. All shattered and loosened rock fragments should be removed from the footprint of the footing or mass concrete fill.

Footings bearing on the sound bedrock may be designed on the basis of a maximum vertical, concentric geotechnical resistance of 10,000 kPa at factored ULS. The SLS condition will not govern for footings founded on bedrock.

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 proportions of the footing may be governed by factors other than the vertical bearing resistance. Such factors may include, though not necessarily be limited to sliding resistance and overturning moment.

## **7.3 Spread Footings on Native Soil**

The native soils at the south abutment are considered suitable for the support of lightly loaded spread footings designed for vertical concentric, geotechnical resistances of 300 kPa factored ULS and 200 kPa at SLS.

The native soils at the north abutment are less dense and the use of spread footings at this location is not recommended, especially since the native founding surface will be below the water table.

Overall, considering the available resistance at the south and unsuitability of the north abutment. The use of spread footings bearing on native soil is not recommended at this site.

#### 7.4 Spread Footings on Engineered Fill

Where founding on bedrock is not practical from a structural standpoint and a higher founding elevation is required, the structure can be supported on spread footings bearing on engineered fill. At this site, the engineered fill may bear on either the overburden or the bedrock, provided the footprint of the footing is prepared as described below.

All topsoil, peat, existing fill, loose soil or other deleterious materials must be stripped from the footprint of the foundation. If stripping results in an inflow of water that disturbs the earth subgrade intended to receive engineered fill, the following options are available to the contractor:

- Implement a groundwater control scheme to prevent disturbance of the subgrade and to allow engineered fill to be placed in dry conditions
- Strip all earth overburden and expose bedrock, groundwater control then need only be sufficient to remove accumulated water prior to placing engineered fill

Artesian pressures in the bedrock are not large and any seepage from the bedrock will be dissipated in the overburden and is not expected to cause any problems in the long term.

The geometry of the engineered fill below the underside of the footing must be controlled within the following limits:

- The minimum thickness over exposed bedrock must be 1.0 m
- The minimum thickness over prepared, competent earth subgrade must be at least 2.0 m
- The underside of the engineered fill must lie below Elevation 364.0 at the south abutment and 362.0 at the north abutment

The earth subgrade must be recompacted prior to placing the engineered fill.

The engineered fill must consist of OPSS Granular “A” or Granular “B” Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at  $\pm 2\%$  of optimum moisture content and generally conforming to the geometry illustrated in Figure 1.

Provided a minimum footing width of 2 m is maintained, a footing bearing on the engineered fill may be designed for a concentric, vertical geotechnical resistance of 900 kPa at factored ULS and a resistance at SLS of 350 kPa.

These resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.



For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. Differential settlements are not expected to exceed 20 mm in across the width of the structure.

### **7.5 Caisson Foundations**

Caissons bearing on sound bedrock could be designed for high geotechnical resistances.

However, in view of the comparatively shallow depth of bedrock and potential difficulties in obtaining a seal between the liner and bedrock, caissons will be difficult to install and will be uneconomical when compared to spread footing.

The caisson alternative is not recommended and has not been developed further.

### **7.6 Integral Abutment Considerations**

The site is considered suitable for an integral abutment design.

Semi-integral abutment design utilizing footings on engineered fill is also considered feasible.

### **7.7 Recommended Foundation System**

An integral abutment design supported on piles to bedrock is recommended.

### **7.8 Horizontal Resistance of Footings**

#### **7.8.1 Footing on Bedrock**

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors should be included in the design.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock or grout is exceeded. Using a lower bound value of 20 MPa for the strength of the rock or grout, an ultimate horizontal resistance of 1.5 MN may be assumed 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.

### **7.8.2 Footing on Engineered Fill**

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.7. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

### **7.9 Frost Cover**

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

For footings founded on engineered fill, frost protection should be provided. This may take the form of 1.9 m of earth cover over the footing base (founding elevation). The thickness of soil cover may be reduced by the partial substitution of an appropriate insulation material such as extruded polystyrene.

Typically, 25 mm of rigid, extruded polystyrene insulation is equivalent to 600 mm of earth cover. Suitable protection must be provided to the insulation where it is used.

Rock fill is not equivalent to earth fill in terms of thermal resistance. For design purposes, assume that the depth of frost penetration through rock fill will be 5 m.

## **8 EXCAVATION AND BACKFILL**

### **8.1 General**

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native sand at this site is classed as Type 3 soil above the water table and Type 4 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 or slabs of detached bedrock that may have to be removed..

### **8.4 Rock Excavation**

It is not anticipated that rock excavation will be required at this site.

However, if quantities of rock have to be removed, 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. Such methods may include, though not necessarily be limited to, line drilling, pre-splitting and cushion blasting.

A NSSP should be included governing the use of explosives. The text of the NSSP is included in Appendix E.

The Contractor's blasting and monitoring plan must take account of near-by buildings or plant that might be affected by the blasting to ensure that appropriate peak particle velocities are specified for the protection of the buildings.

## **9 GROUNDWATER CONTROL**

Near surface groundwater in the sand layer or overlying peat will be trapped locally in depressions in the rock surface and will seep into excavations made through the sand. Artesian pressure was noted in the bedrock at this site and at the adjacent structure site and may cause seepage into an excavation.

The design of foundations bearing on bedrock will not be influenced by the groundwater but the Contractor must make provision to control the groundwater seepage and use sump pumps to pump any accumulation of water from the footing base prior to placing concrete.

The groundwater conditions observed at site will present difficulties for any attempt to compact the existing overburden. If the design requires compaction of the overburden, the Contractor should put in place an adequate dewatering system. Due to the shallow depth of bedrock, one feasible system may be perimeter ditches and sumps.

## **10 APPROACH EMBANKMENTS**

The approach embankments within 20 m of this structure will be constructed on shallow deposits of non-cohesive soil overlying bedrock. It is recommended that the topsoil/peat be stripped from within the footprint of the approach fill.

The existing overburden, will satisfactorily support the approach fills at this site, which are not expected to exceed 10 m in height. The foundation materials will provide adequate protection against global stability failure of earth fill and rock fill embankments constructed with side slopes of 2H:1V and 1.25H:1V, respectively.

Some minor settlement, in the order of 30 to 40 mm, will occur in the sand as the new embankment fill is placed. This settlement should be complete by the end of construction and long-term consolidation-type settlement is not an issue at this site.

In terms of settlement within the mass of the embankment fill, the magnitude and time to substantial completion will be dependent on the fill material used. Guidelines as to the expected order of magnitude are shown in Table 10.1.

**Table 10.1 – Settlement of Embankment Fill**

<b>Fill Material</b>	<b>Magnitude of Post-Construction Settlement</b>	<b>Approximate Time Frame for Settlement</b>
Rock Fill	Negligible	Immediate
Granular fill	0.3% to 0.5% of the fill height	One to two years
SSM/Earth fill	0.5% to 0.7% of fill height	Two to four years
Cohesive earth fill	1% to 2% of fill height	Several years, depending on nature of fill and method of placement.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry but also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations no steeper than 1.25H:1V. Embankments constructed using granular material, select subgrade material and most earth materials will have stable side slopes at inclinations no steeper than 2H:1V.

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 earth fill embankments are higher than 8 m, berms must be incorporated at a height of 8 m below the subgrade level. The berms must:

- extend for the length through which the embankment height exceeds 8 m
- be 2 m wide
- have 2% positive drainage to shed run-off water.

Where rock fill embankments are higher than 6 m, berms must be incorporated at a height of 6 m below the subgrade level. The berms must:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

## **11 RETAINED SOIL SYSTEMS**

Retained soil system (RSS) walls may be considered at this site in conjunction with engineered fill pads. Due to the nature and variability of the native soils, construction of a RSS system directly on the native soil is not recommended.

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.

The levelling pad for the RSS wall may be formed directly on the exposed bedrock, mass concrete fill or on a pad of engineered fill. Engineered fill should be designed in the same manner as the fill pads to support foundations as described elsewhere in this report, except that a minimum thickness of 600 mm is required. The geotechnical resistance of the bedrock or engineered fill is as stated elsewhere in this report. The elevations for the construction of engineered fill and for bedrock are provided elsewhere in this report.

The foundation under the RSS wall will be bedrock or engineered fill overlying bedrock. In either case, the global stability of the wall will be satisfactory.

The internal stability of the RSS 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 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 semi-integral abutments, backfill to the abutment should be granular material. Integral abutment design is not recommended at this site.

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 in order to reduce the loss of granular material into the rock fill.

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.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

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

### 13 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. The pressures should be computed in accordance with 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)

$\gamma$  = unit weight of retained soil (see table below)

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

$q$  = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 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 values are shown in Table 13.1.

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.

**Table 13.1 – Earth Pressure Coefficient (K)**

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 (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

\* For wing walls.

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 Canadian Highway Bridge Design Code.

## 14 SEISMIC CONSIDERATIONS

For design purposes, the site is treated as lying in Seismic Zone 2.

### 14.1 Seismic Design Parameters

The following seismic parameters should be used for design::

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1
- Peak Horizontal Acceleration 0.11

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 1.0 should be used in seismic design.

## 14.2 Liquefaction Potential

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method<sup>1</sup>.

Using this method, it was determined that the foundation soils at the abutments are not in danger of liquefaction under earthquake loading when confined under the embankment loading.

Additionally, the recommended foundation system will involve removing loose, near-surface soil and replacing it with compacted Granular A or Granular B Type II, which are not considered to be prone to liquefaction.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction but would be subject to failure in the event that the foundation soil liquefied.

There is a possibility at this site that the native soil could liquefy under the toe of the embankment, particularly at the north approach. Further work is recommended to study possible ground improvement options but that work is beyond the present terms of reference.

## 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 values of ( $K_{AE}$ ) and ( $K_{PE}$ ), the following geotechnical parameters were used:

$\phi$	= 35° for OPSS Granular A or Granular B Type II
$\phi$	= 32° for OPSS Granular B Type I
$\phi$	= 42° for rock fill
$\delta$	= 50% of $\phi$

Where  $\phi$  = the angle of internal friction of the backfill and  $\delta$  = the angle of friction between the wall and the backfill.

---

<sup>1</sup> Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 101, No. SM9, September, pp. 1249 – 1273.



The seismic earth pressure coefficients to be used in design at this site are shown in Table 14.1 at the end of the text.

## 15 CONSTRUCTION CONCERNS

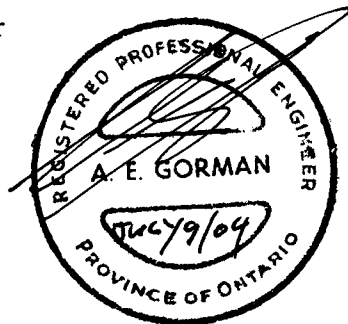
During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- Disturbance of the native overburden by excavating or attempting to re-compact without adequate groundwater control
- Difficulty in compacting engineered fill due to inadequate site preparation or inadequate control of the groundwater

Direction of Fieldwork and Engineering Analysis  
Alastair E. Gorman, P.Eng.,  
Senior Geotechnical Engineer

Report reviewed by:  
P.K. Chatterji, P.Eng.,  
Review Principal

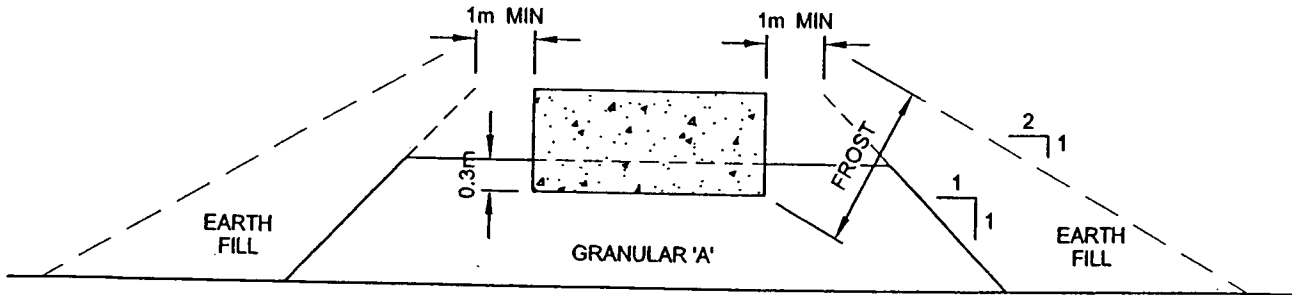


**Table 14.1**  
**Earth pressure Coefficients for Seismic Design**

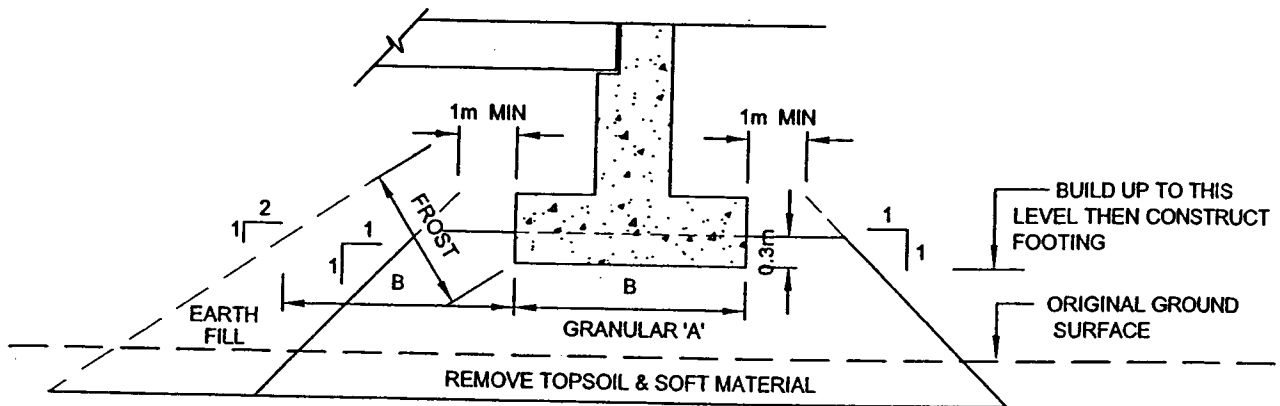
Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active ( $K_{AE}$ )*	0.30	0.45	0.33	0.54	0.23	0.31
Passive ( $K_{PE}$ )*	6.3	6.3	5.4	5.4	12.0	12.0
At Rest ( $K_{OE}$ )**	0.59		0.63		0.33	

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods



## CROSS-SECTION

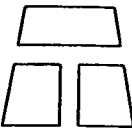


## LONGITUDINAL SECTION

NOT TO SCALE

### NOTES:

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

ENGINEER	AEG	<div style="text-align: center;">   <b>THURBER</b> </div>
DRAWN	SS	
DATE	April , 2004	
APPROVED	PKC	
SCALE	NTS	
<div style="text-align: center;"> <b>ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE</b> </div>		DWG. NO. <b>FIGURE 1</b>

**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 <sup>(1)</sup> "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


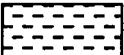



C <sub>vane</sub>	Shear Strength Determination by Field Insitu Vane
C <sub>pen</sub>	Shear Strength Determination by Pocket Penetrometer
C <sub>lab</sub>	Shear Strength Determination using a Laboratory Vane Apparatus
C <sub>u</sub>	Undrained Shear Strength determined by Unconfined Compression Test

- (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

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
<b>Fresh (FR)</b>	No visible signs of weathering.		
<b>Fresh Jointed (FJ)</b>	Weathering limited to the surface of major discontinuities.		CLAYSTONE
<b>Slightly Weathered (SW)</b>	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
<b>Moderately Weathered (MW)</b>	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
<b>Highly Weathered (HW)</b>	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
<b>Completely Weathered (CW)</b>	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>		
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa)      (psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250      Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m			
Medium bedded	0.2 to 0.6m	Very Strong	100-250      15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m			
Very thinly bedded	20 to 60mm	Strong	50-100      7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm			
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0      3,500 to 7,500	Breaks under single blow of geological hammer.
		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

<u>TERMS</u>	
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 417-11

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070324.8 E 311290.0 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 08.01.04 - 08.01.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
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											
365.0	SAND, fine grained, trace silt, trace to some iron oxide staining, trace rootlets, organic odour Dense to Compact Brown Moist (SP)		1	SS	19											0 91 9 (SI+CL)		
0.0																		
			2	SS	32													
			3	SS	28													
362.8	Sandy SILT, some clay, occasional iron oxide staining Dense Brown Wet (ML-nonplastic)															0 24 65 11		
2.2																		
			4	SS	36													
362.0	GNEISS, BEDROCK Fresh, slightly weathered at joints, laminated to very thinly bedded, pink with black subvertical banding, planar and rough joint surface, extremely strong Vertical joint from 3.2m to 3.3m Subvertical joint from 3.3m to 3.4m															RUN 1# TCR=100%, SCR=91%, RQD=91%, UCS=225.8MPa		
3.1																		
			1	RUN														






# RECORD OF BOREHOLE No 417-12

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070328.6 E 311289.5 ORIGINATED BY SL  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 18.12.03 - 18.12.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
365.1																		
0.0	TOPSOIL (250mm)						365											
364.9																		
0.3	SAND, fine to medium grained, trace to some silt Compact Brown Moist to Wet		1	SS	29		364											
			2	SS	20													
							363											
			3	SS	21													
362.4																		
2.7	Sandy SILT, trace clay Compact to Very Dense Brown Moist to Wet		4	SS	50/ .127		362											
361.7																		
3.4	END OF BOREHOLE AT 3.4m. AUGER REFUSAL AT 3.4m. BOREHOLE OPEN TO 1.98m. WATER LEVEL IN OPEN BOREHOLE AT 1.14m DEPTH UPON COMPLETION.																	

ONTMT4 417ADAMS.GPJ 21/04/04

# RECORD OF BOREHOLE No 417-13

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070326.5 E 311295.6 ORIGINATED BY GA  
HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
DATUM Geodetic DATE 07.01.04 - 07.01.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								20 40 60 80 100							
364.6	SAND, fine to medium grained, trace to some iron oxide staining, occasional rootlets, trace to some silt Compact Brown Wet (SP)   														

# RECORD OF BOREHOLE No 417-14

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070324.4 E 311303.9 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 07.01.04 - 07.01.04 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
364.5 0.0	SAND, fine to medium grained, trace to some iron oxide staining, trace to some silt, occasional rootlets Compact Brown Wet (SP)		1	SS	11		364						
			2	SS	24		363						
	Dense		3	SS	34		362						
	occasional silt		4	SS	21								1 85 15 (SI+CL)
361.4 3.1	END OF BOREHOLE AT 3.05m. AUGER REFUSAL AT 3.05m ON PROBABLE BEDROCK. BOREHOLE OPEN TO 2.9m. WATER LEVEL IN OPEN BOREHOLE AT 0.76m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.												

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15  
10  
5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 417-15

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070326.8 E 311303.7 ORIGINATED BY SL  
HWY 11 BOREHOLE TYPE Solid Stem Augers, NQ Core COMPILED BY SS  
DATUM Geodetic DATE 17.12.03 - 17.12.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)			
								20 40 60 80 100						W <sub>P</sub> W      W <sub>L</sub>		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE								
364.5																
0.0	TOPSOIL															
364.2																
0.4	SAND, trace silt Compact Brown Moist		1	SS	25											
			2	SS	22											
362.3																
2.2	Sandy SILT, trace clay, trace sand Compact Brown Moist (ML-nonplastic)		3	SS	20											
			4	SS	13											
360.9																
3.7	GNEISS, BEDROCK Fresh, Slightly weathered at joints, laminated to very thinly bedded, orange and pink with subhorizontal black banding, planar and rough joint surface, very strong to extramly strong Subvertical joint from 4.9m to 5.0m		1	RUN												
			2	RUN												
358.7			3	RUN												
5.8	END OF BOREHOLE AT 5.84m. AUGER REFUSAL AT 3.66m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.83m slotted screen.															

+<sup>3</sup> × 3: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 417-16

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070358.6 E 311294.4 ORIGINATED BY DP  
HWY 11 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
DATUM Geodetic DATE 11.02.04 - 11.02.04 CHECKED BY AEG



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20	40	60	80	100		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE						
								20	40	60	80	100		
								WATER CONTENT (%)						
								PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT						
								W <sub>P</sub> W      W <sub>L</sub>						
364.6														
0.0	TOPSOIL, sandy loam, some organics													
364.3	Brown													
0.3	SAND, fine to very fine grained, slightly stained by organics, faint organics odor.													
	Loose		1	SS	8		364							
	Brown													
	Wet													
			2	SS	6		363							
	trace to some silt		3	SS	28		362							
	Compact													
	Moist		4	SS	53/									
361.4														
3.3	GRANITE, BEDROCK				.052									
	Fresh, pink with black speckles, planar and rough joint surface, extremely strong		1	RUN			361							
			2	RUN			360							
			3	RUN			359							
358.2														
6.5	END OF BOREHOLE AT 6.45m. WATER LEVER AT SURFACE. BOREHOLE CAVE TO 1.52m.													

# RECORD OF BOREHOLE No 417-17

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070365.7 E 311291.7 ORIGINATED BY SL  
 HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 19.12.03 - 19.12.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  γ  kN/m <sup>3</sup>	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						
								20 40 60 80 100						
				○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) W <sub>P</sub> — W — W <sub>L</sub>						
364.6														
0.0	TOPSOIL													
364.2														
0.4	SAND, topsoil stained, trace to some silt Loose to Compact Dark Brown Wet		1	SS	3		364							
			2	SS	10		363							
			3	SS	13		362							1 86 13 (SI+CL)
			4	SS	56/ 229									
361.1														
3.5	END OF BOREHOLE AT 3.48m. AUGER REFUSAL AT 3.48m. PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN TO 2.79m. WATER LEVEL IN OPEN BOREHOLE AT 2.79m DEPTH UPON COMPLETION.													

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 417-18

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070362.6 E 311297.6 ORIGINATED BY SL  
 HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 19.12.03 - 19.12.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100										

364.5															
0.0	TOPSOIL														
364.2															
0.3	SAND, fine to medium grained Compact Brown Wet		1	SS	11		364								
	Loose		2	SS	9		363								
361.8			3	SS	10		362								
2.7	Sandy SILT, trace gravel, occasional cobbles Compact Grey Wet		4	SS	10		361							0 39 56 5	
359.7			5	SS	50/		360								
4.8	END OF BOREHOLE AT 4.78m. AUGER REFUSAL AT 4.78m. PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN TO 2.13m. WATER LEVEL IN OPEN BOREHOLE AT 0.20m DEPTH UPON COMPLETION.				.150										

+ 3, x 3: Numbers refer to  
Sensitivity




20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 417-19

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070360.5 E 311303.7 ORIGINATED BY SL  
HWY 11 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
DATUM Geodetic DATE 19.12.03 - 19.12.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE							
364.4							20	40	60	80	100							
0.0	TOPSOIL																	
364.0																		
0.5	SAND, fine to medium grained, topsoil stained, trace silt Loose to Compact Dark Brown Wet		1	SS	5													
			2	SS	12													
			3	SS	13													
361.6																		
2.8	Sandy SILT Loose Grey Wet		4	SS	9													
359.9																		
4.5	END OF BOREHOLE AT 4.52m. AUGER REFUSAL AT 4.53m. PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN TO 2.64m. WATER LEVEL IN OPEN BOREHOLE AT 0.64m DEPTH UPON COMPLETION.																	

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 417-20

1 OF 1

METRIC

W.P. 759-93-00 LOCATION 417 Adams Road N 5070364.4 E 311305.1 ORIGINATED BY DP  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 11.02.04 - 11.02.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100			W <sub>p</sub>	W	W <sub>L</sub>		
								SHEAR STRENGTH kPa							WATER CONTENT (%)				
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE								
364.3																			
0.0	TOPSOIL, sandy loam, some organics																		
364.0	Brown																		
0.3	SAND, fine to very fine grained, stained by organics, faint organics odor above 2.21m																		
	Very Loose to Loose		1	SS	1														
	Brown																		
	Wet																		
	trace silt																		
359.9			5	SS	56/														
					.229														
4.3	GNEISS, BEDROCK																		
	Fresh, slightly weathered at joints, laminated to very thinly bedded, pink with black subvertical banding, occasional iron oxide staining at joints, planar and rough joint surface, extremely strong		1	RUN															
	Sand seam from 4.6m to 4.7m.																		
	Cobbles from 4.7m to 4.8m.																		
	Subhorizontal joint at 4.9m.																		
	Becoming massive at 5.1m.																		

ONTMT4 417ADAMS.GPJ 21/04/04

**Appendix B**

**Laboratory Test Results**

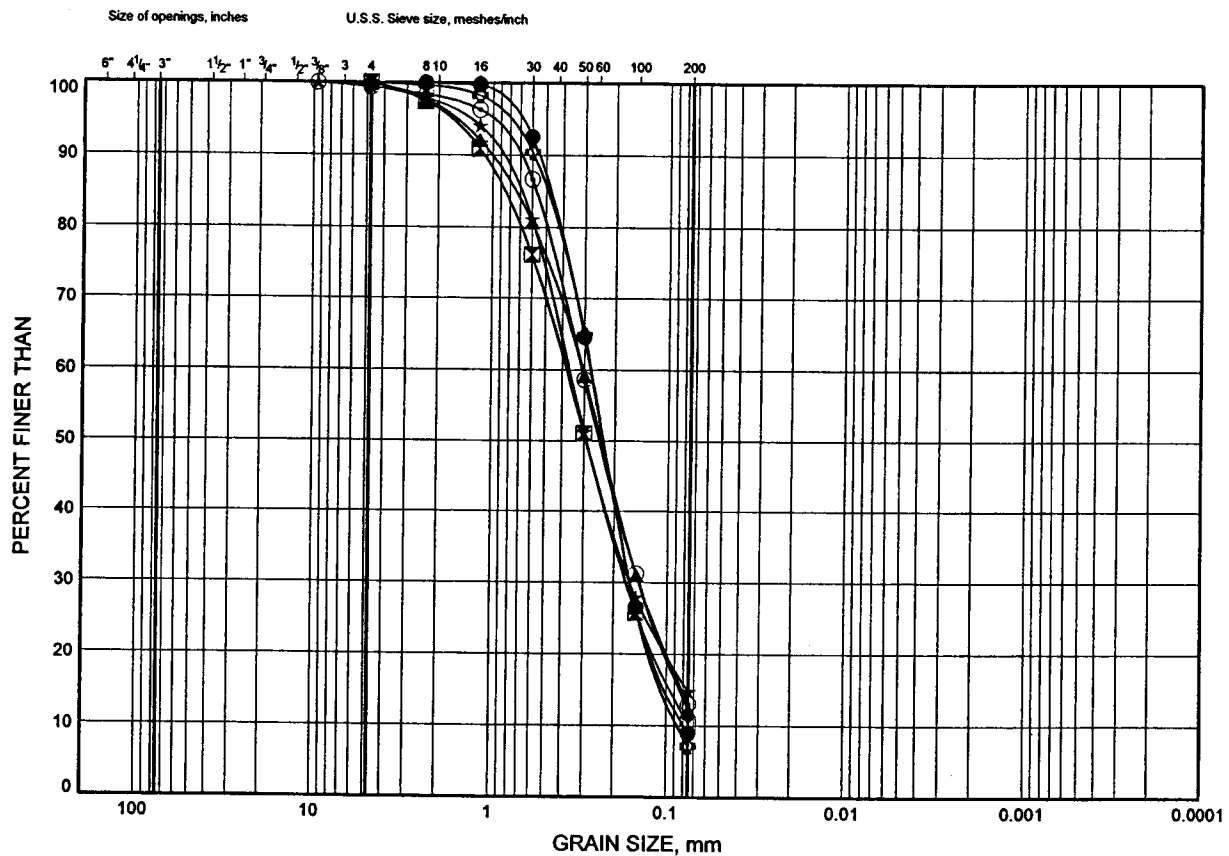


# Hwy 11 Four Laning

## GRAIN SIZE DISTRIBUTION

FIGURE B1-N

### SILTY SAND

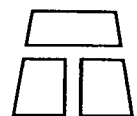


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	417-11	1.07	363.94
⊠	417-12	2.51	362.59
▲	417-13	1.83	362.81
★	417-14	2.59	361.88
⊙	417-17	2.51	362.10
⊕	417-19	1.83	362.58

Date April 2004

Project 759-93-00



THURBER

Prep'd SS

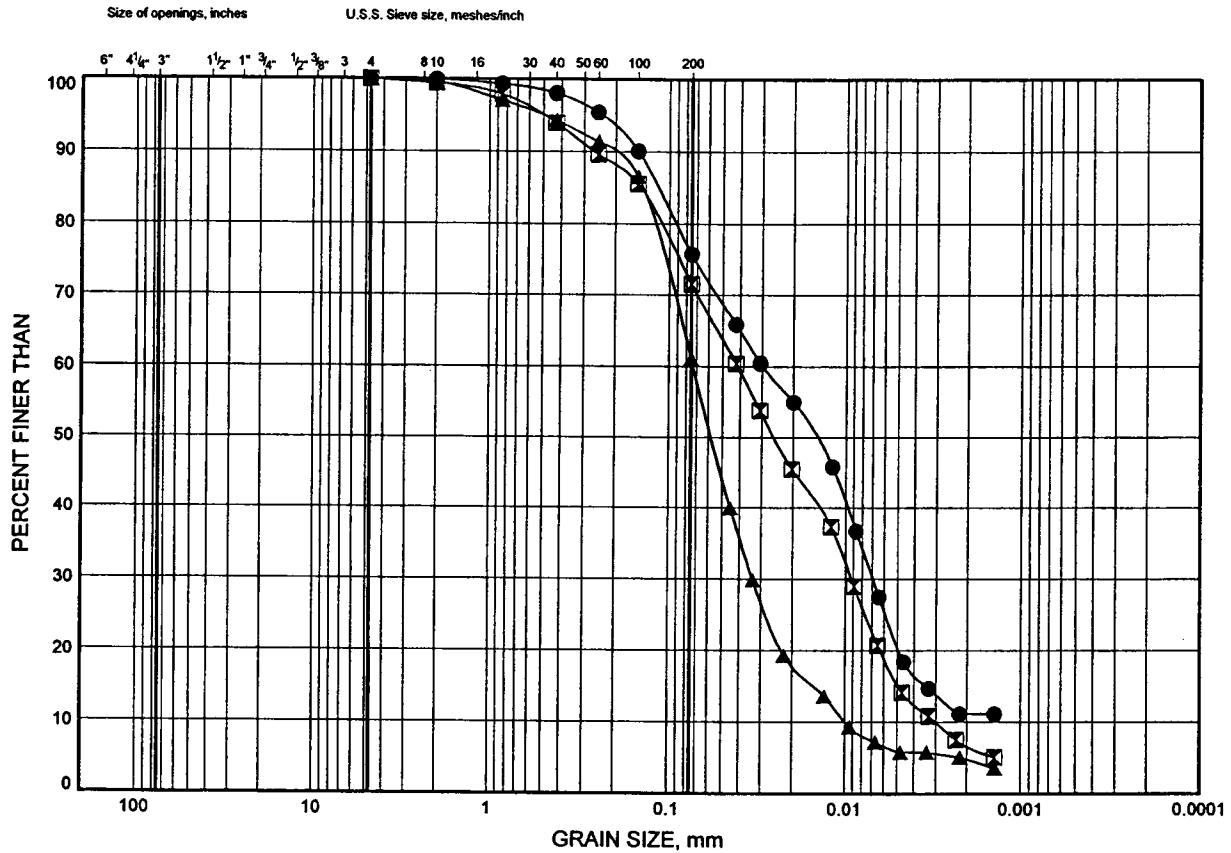
Chkd. AEG

# Hwy 11 Four Laning

## GRAIN SIZE DISTRIBUTION

FIGURE B2-N

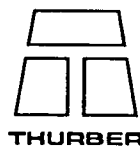
### SANDY SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	417-11	2.59	362.42
⊠	417-15	2.44	362.07
▲	417-18	3.35	361.15

Date April 2004  
Project 759-93-00



Prep'd SS  
Chkd. AEG

## **Appendix C**

### **Factual Data from Golder's Report**

PROJECT <u>991-1193</u>		<b>RECORD OF BOREHOLE No 10-1</b>		1 OF 1		<b>METRIC</b>	
W.P. <u>335-98-00</u>		LOCATION <u>N 5070290, E 311257</u>		ORIGINATED BY <u>SS</u>			
DIST <u>54</u> HWY <u>11</u>		BOREHOLE TYPE <u>108mm I.D. HOLLOW STEM AUGERS</u>		COMPILED BY <u>DKB</u>			
DATUM <u>GEODETIC</u>		DATE <u>Feb. 16/00</u>		CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI C			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		

365.00	GROUND SURFACE																
0.00 364.70 0.30	Fibrous Peat Black Wet Sand, trace silt and gravel Compact Brown Moist to wet		1	SS	13	▽	364										
			2	SS	29		363										
362.71 2.29	END OF BOREHOLE Refusal to further auger penetration; probable bedrock.  Note: Water level measured in open borehole at 0.8m depth (El. 364.2m) upon completion of drilling.  Northing and Easting co-ordinate accurate to nearest metre.																

ON MOT 991-1193.GPJ ON MOT.GDT 24/000

PROJECT 991-1193  
 W.P. 3-5-98-00  
 DIST 54 HWY 11  
 DATUM GEODETIC

LOCATION N 5070310: E 311257  
 BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS  
 DATE Feb. 16/00


1 OF 1 METRIC  
 ORIGINATED BY SB  
 COMPILED BY DKB  
 CHECKED BY ASP

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa			WATER CONTENT (%)							
									20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
365.50	GROUND SURFACE															
0.00	Gravelly Sand, trace to some silt Compact Brown Wet															
364.31			1	SS	21											
1.19	Slightly weathered to fresh, grayish-pink with black speckles, moderately to widely jointed, lightly foliated, coarse grained, strong to very strong GRANITE.															
	Bedrock cored from 1.19m to 4.50m depth.															
	For bedrock coring details refer to Record of Drillhole 10-2															
361.00																
4.50	END OF HOLE															
	Note: 1. Water level measured in piezometer at 0.3m above ground surface (El. 365.8m) upon completion of installation. 2. Water level measured in piezometer at 0.4m above ground surface (El. 365.9m) on March 8, 2000. 3. Water level measured in piezometer at 0.4m above ground surface (El. 365.9m) on March 26, 2000.  Northing and Easting co-ordinate accurate to nearest metre.															

+ 3, X 3: Numbers refer to Sensitivity  
 ○ 3% STRAIN AT FAILURE

ON MOT 991-1193 GPJ ON MOT GOT 24/00

PROJECT <u>991-1193</u>		<b>RECORD OF BOREHOLE No 10-3</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>335-98-00</u>		LOCATION <u>N 5070350; E 311259.14</u>		ORIGINATED BY <u>SB</u>	
DIST <u>54</u> HWY <u>11</u>		BOREHOLE TYPE <u>108mm I.D. HOLLOW STEM AUGERS</u>		COMPILED BY <u>DKB</u>	
DATUM <u>GEODETIC</u>		DATE <u>Feb. 15/00</u>		CHECKED BY <u>ASP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT  Y  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100		10 20 30				
365.50 0.00	GROUND SURFACE  Gravelly Sand, trace to some silt Compact Brown Wet													
363.90 1.60	Slightly weathered to fresh, grayish-pink with black speckles, moderately jointed, foliated (60°), medium to coarse grained, strong to very strong ANORTHOSITE GNEISS.  Bedrock cored from 1.60m to 4.06m depth.  For bedrock coring details refer to Record of Drillhole 10-3		1	SS	28									
361.44 4.06	END OF HOLE  Northing co-ordinate accurate to nearest metre.													

ON MOT 991-1193.GPJ ON MOT.GDT 24/4/00



PROJECT: 991-1193

## RECORD OF DRILLHOLE: 10-3

SHEET 1 OF 1

LOCATION: N 5070350; E 311259.14

DRILLING DATE: Feb.15/00

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR FRACTURE CL CLEAVAGE SH SHEAR VN VEIN	F FAULT J JOINT P POLISHED S SLICKENSIDED	SM SMOOTH R ROUGH ST STEPPED PL PLANAR	FL FLEXURED UE UNEVEN W WAVY C CURVED	BC BROKEN CORE MB MECH. BREAK B BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		363.80										
2		Slightly weathered, grayish pink with black speckles, moderately jointed, foliated (60°), medium to coarse grained, strong to very strong ANORTHOSITE GNEISS.		1.80										
3					1		100							
4					2		100							
		END OF HOLE		361.44 4.86										
5														
6														
7														
8														
9														
10														
11														

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: PD

DRILLHOLE 1193 ROCK GPJ GLDR CAN GDT 24/400 PS

PROJECT 991-1193		RECORD OF BOREHOLE No 10-4				1 OF 1		METRIC	
W.P. 335-98-00		LOCATION N 5070365 E 311264				ORIGINATED BY SB			
DIST 54 HWY 11		BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS				COMPILED BY DKB			
DATUM GEODETTIC		DATE Feb. 15/00				CHECKED BY ASP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
365.50	GROUND SURFACE																
0.00	Fibrous Peat Black Wet																
0.30	Sand, trace to some silt, trace organics Loose Blackish brown to brown Wet		1	SS	7												
			2	SS	8												
363.29	Sandy Silt, trace clay Compact Brown Wet		3	SS	10												
			4	SS	12												
361.99	Sand and Gravel, trace silt Very dense Brown Wet		5	SS	23/15												
3.51																	
361.54																	
3.96																	
	END OF BOREHOLE Refusal to further auger penetration; probable bedrock.  Note: Water level in open borehole at ground surface upon completion of drilling.  Northing and Easting co-ordinate accurate to nearest metre.																

ON MOT 991-1193.GPJ ON MOT.GDT 24/4/00

PROJECT <u>991-1193</u>			<b>RECORD OF BOREHOLE No 10-5</b>			1 OF 1			<b>METRIC</b>		
W.P. <u>335-98-00</u>			LOCATION <u>N 5070304; E 311297</u>			ORIGINATED BY <u>SB</u>					
DIST <u>54</u> HWY <u>11</u>			BOREHOLE TYPE <u>108mm I.D. HOLLOW STEM AUGERS</u>			COMPILED BY <u>DKB</u>					
DATUM <u>GEODETIC</u>			DATE <u>Feb. 16/00</u>			CHECKED BY <u>ASP</u>					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
364.00	GROUND SURFACE																
0.00	Topsoil																
363.70																	
0.30	Sand, trace silt and gravel Dense to compact Brown Moist to wet		1	SS	30												
			2	SS	21												
361.79																	
2.21	Sandy Silt, trace clay Dense Brown		3	SS	37												
361.26																	
2.74	Wet <b>END OF BOREHOLE</b> Refusal to further auger penetration; probable bedrock  Note: Water level measured in open borehole at 1.2m depth (El. 362.8m) upon completion of drilling.  Northing and Easting co-ordinates accurate to nearest metre.																

ON MOT 991-1193.GPJ ON MOT.GDT 24/4/00

PROJECT 991-1193			RECORD OF BOREHOLE No 10-6			1 OF 1			METRIC				
W.P. 335-98-00			LOCATION N 5070324; E 311298			ORIGINATED BY SB							
DIST 54 HWY 11			BOREHOLE TYPE 109mm I.D. HOLLOW STEM AUGERS			COMPILED BY DKB							
DATUM GEODETTIC			DATE Feb 16/00			CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	W <sub>p</sub>	W	W <sub>L</sub>	UNIT WEIGHT γ	GR SA SI CL
364.00	GROUND SURFACE							20 40 60 80 100					
0.00	Topsoil							20 40 60 80 100					
0.30	Sand, trace silt and gravel Compact Brown Moist to wet		1	SS	15	7	363						
			2	SS	24		362						
361.79	Sandy Silt, trace clay Compact Brown Wet		3	SS	20		361						
			4	SS	14		360						
360.29	Slightly weathered to fresh, grey white with black blotches, moderately jointed, slightly foliated, medium to coarse grained, strong to very strong GRANITIC GNEISS.						359						
3.71	Bedrock cored from 3.71m to 6.71m depth.  For bedrock coring details refer to Record of Drillhole 10-6						358						
357.29	END OF HOLE												
6.71	Note: Water level measured in open borehole at 1.1m depth (El. 362.9m) upon completion of drilling.  Northing and Easting co-ordinate accurate to nearest metre.												

ON MOT 991-1193.GPJ ON MOT.GDT 24/00

PROJECT: 991-1193

## RECORD OF DRILLHOLE: 10-6

SHEET 1 OF 1

LOCATION: N 5070324; E 311298

DRILLING DATE: Feb.16/00

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE		F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
							CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK			
							SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING			
							VN-VEN		S-SUCKENSIDED		PL-PLANAR		C-CURVED					
RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY % DRIVING												
TOTAL CORE %	SOLID CORE %			DIP W.I.L. CORE AXIS	TYPE AND SURFACE DESCRIPTION	1	2	3	4									
		GROUND SURFACE	350.29															
4		Slightly weathered to fresh, gray-white with black blotches, moderately jointed, slightly foliated, medium to coarse grained, strong to very strong GRANITIC GNEISS.	3.71															
5	NO RC			1	0.3	100												
6				2	0.2	100												
7				3	0.1	100												
		END OF HOLE	357.29															
7			6.71															
8																		
9																		
10																		
11																		
12																		
13																		

DEPTH SCALE

1:50

Golder  
Associates

LOGGED: SB

CHECKED: PD

DRILLHOLE 1193 ROCK GPJ GLDR CAN GDT 2/14/00 PS

PROJECT <u>991-1193</u>		<b>RECORD OF BOREHOLE No 10-7</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>335-98-00</u>		LOCATION <u>N 5070364.94; E 311299.70</u>		ORIGINATED BY <u>SB</u>	
DIST <u>54</u> HWY <u>11</u>		BOREHOLE TYPE <u>108mm I.D. HOLLOW STEM AUGERS</u>		COMPILED BY <u>DKB</u>	
DATUM <u>GEODETTIC</u>		DATE <u>Feb. 14/00</u>		CHECKED BY <u>ASP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	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										
364.50	GROUND SURFACE																	
0.00	Fibrous Peat Black Wet																	
363.74																		
0.76	Sand, trace silt, trace organics/decaying wood matter to 2.2m depth Very loose to compact Blackish brown to brown Wet		1	SS	2													
			2	SS	11													
			3	SS	8													
361.53																		
2.97	Sandy Silt, trace clay Loose to compact Brown Wet		4	SS	5													
			5	SS	14													
360.00																		
4.50	Sand and Gravel, trace silt Very dense Brown Wet		6	SS	21/10													
359.62																		
4.88	Slightly weathered, pinkish grey-white with black blotches, moderately jointed, lightly foliated, coarse to very coarse grained, strong GRANITIC GNEISS.																	
	Bedrock cored from 4.88m to 7.62m depth.																	
	For bedrock coring details refer to Record of Drillhole 10-7																	
356.88																		
7.62	END OF HOLE																	
	Note: 1. Water level measured in piezometer at 1.1m depth (El. 363.4m) upon completion of installation. 2. Water level measured in piezometer at 1.0m above ground surface (El. 365.5m) on March 8 and 26, 2000.																	

ON MOT 991-1193.GPJ ON MOT.GDT 24/4/00

PROJECT: 991-1193

## RECORD OF DRILLHOLE: 10-7

SHEET 1 OF 1

LOCATION: N 5070364.94; E 311299.70

DRILLING DATE: Feb.15/00

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETER POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION					
															FR-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE
															CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK
		GROUND SURFACE		359.82															
5		Slightly weathered, pinkish grey-white with black blotches, moderately jointed, lightly foliated, coarse to very coarse grained, strong GRANITIC GNEISS. Becomes Pegmatitic, very coarse grained 5.3-5.4m.		4.84															
6	1																		
7	2																		
8		END OF HOLE		356.84															
9				7.62															
10																			
11																			
12																			
13																			
14																			

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: PD

DRILLHOLE 1193ROCK.GPJ GLDR CAN.GDT 24/00 PS

PROJECT 991-1193

# RECORD OF BOREHOLE No 10-8

1 OF 1

METRIC

W.P. 335-98-00

LOCATION N 5070384, E 311300

ORIGINATED BY SB

DIST 54 HWY 11

BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS

COMPILED BY DKB

DATUM GEOODETIC

DATE Feb. 15/00

CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
365.50	GROUND SURFACE													
368.82 0.18	Fibrous Peat Black Wet Sand, trace silt, trace organics to 2.2m depth Very loose to loose Blackish brown to brown Wet		1	SS	WH	7	365							
			2	SS	8		364							
			3	SS	8		363							
362.53 2.97	Sandy SR, trace clay Compact to dense Brown Wet		4	SS	17		362							
361.35 4.15	END OF BOREHOLE Refusal to further auger penetration; probable bedrock  Note: Water level measured in open borehole at 1.3m depth (El. 364.2m) upon completion of drilling.  Northing and Easting co-ordinate accurate to nearest metre.		5	SS	45/20									

+3, X3: Numbers refer to  
Sensitivity

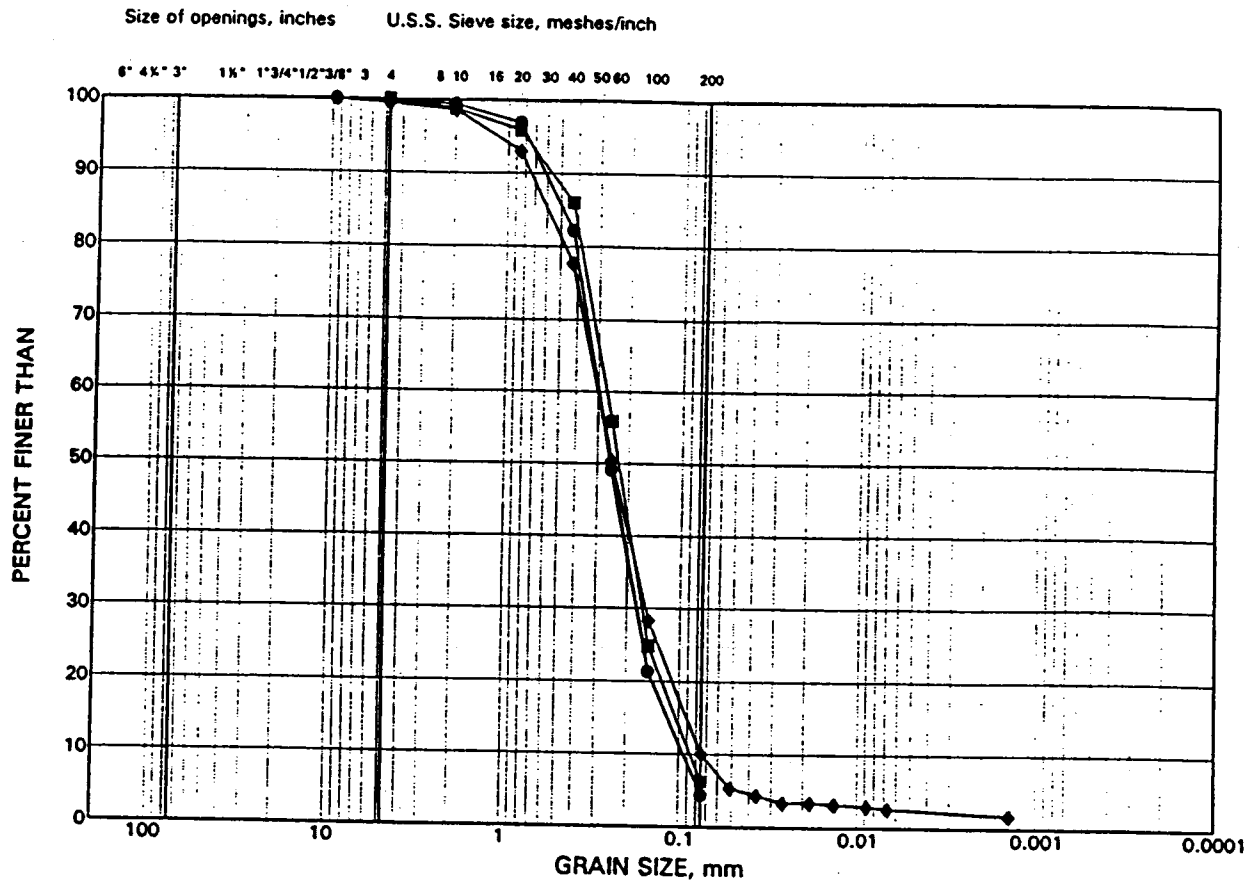
○ 3% STRAIN AT FAILURE



# GRAIN SIZE DISTRIBUTION

Sand, trace silt and gravel

FIGURE 13



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

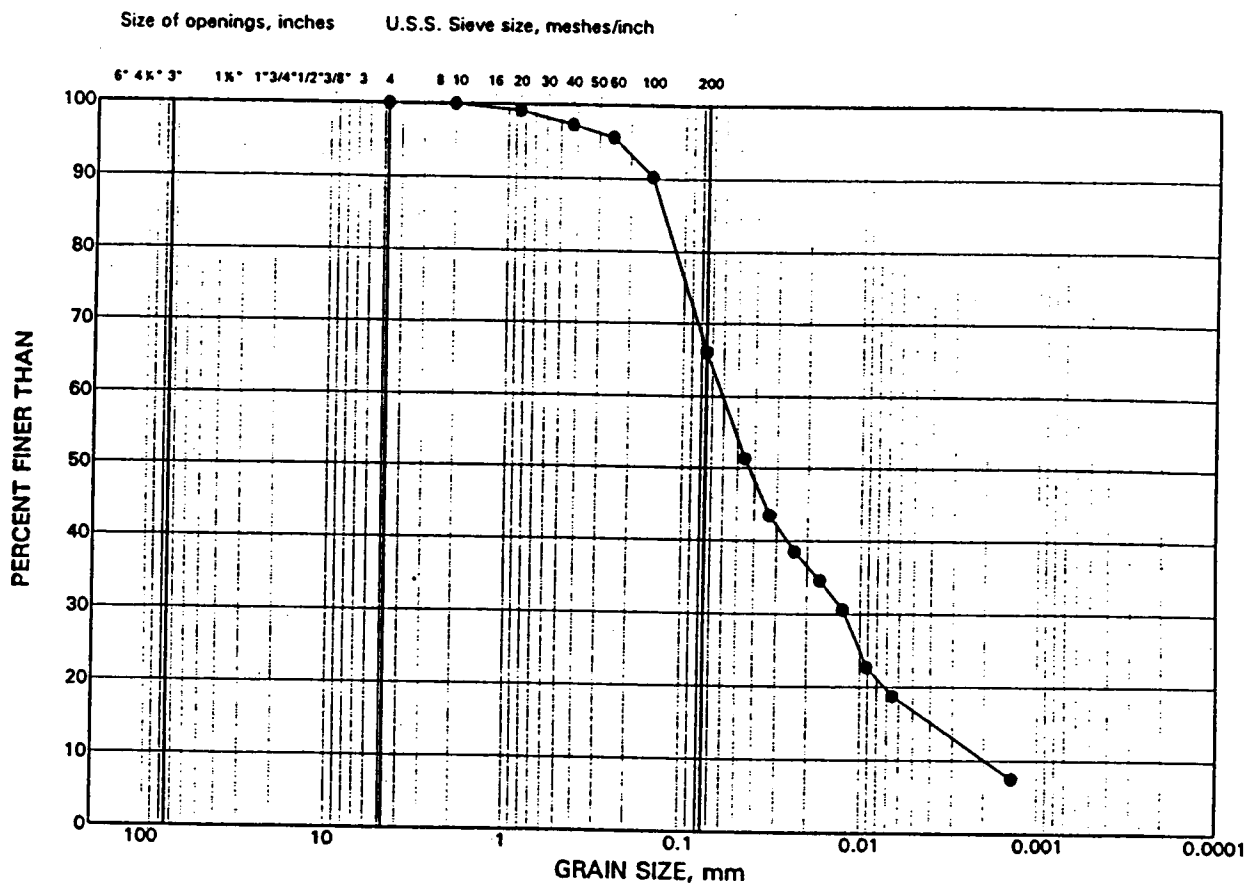
SYMBOL	BOREHOLE	SAMPLE ELEVATION(m)
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●	10-7	3      361.6
■	10-8	3      362.6
◆	10-4	2      363.4

# GRAIN SIZE DISTRIBUTION

Sandy Silt, trace clay

FIGURE 14



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION
--------	----------	--------	-----------

•	10-6	4	360.3
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## **Appendix D**

### **Foundation Comparison**

**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Footing on Native Soil	Caisson
<b>South Abutment</b>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance.</li> <li>ii. Commonly used system for highway bridge foundations.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Proximity of bedrock surface to the underside of the girders mitigates against the use of driven piles unless bedrock is excavated to accommodate an integral abutment.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Bedrock comparatively close to the underside of the girders.</li> <li>ii. Short abutment stem.</li> <li>iii. High values of geotechnical resistance are available on the bedrock</li> <li>iv. Allows footing to be placed close to edge of the rock cut.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Stepped footing may be required</li> <li>ii. High cost of excavation, if any is required</li> <li>iii. Mass concrete fill required to create a level founding surface.</li> </ul> <p>Groundwater control will be required.</p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher geotechnical resistance than native soil can provide.</li> <li>ii. Permits founding at elevations above original ground level.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower geotechnical resistance than bedrock</li> <li>ii. Cost of constructing engineered fill</li> <li>iii. Groundwater control will be required at the north abutment.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less expensive to construct.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Soil conditions at this site are not favourable.</li> </ul> <p>NOT RECOMMENDED</p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High bearing resistance</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Difficulties installing caissons through saturated cohesionless soils.</li> <li>ii. Proximity of bedrock surface and short length of caissons not likely to justify costs.</li> </ul>

Strong Twp 10/11 (Adams Rd) Overpass, NBL

<p><b>North Abutment</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance.</li> <li>ii. Commonly used system for highway bridge foundations.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Proximity of bedrock surface to the underside of the girders mitigates against the use of driven piles unless bedrock is excavated to accommodate an integral abutment.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Bedrock comparatively close to the underside of the girders.</li> <li>ii. Short abutment stem.</li> <li>iii. High values of geotechnical resistance are available on the bedrock</li> <li>iv. Allows footing to be placed close to edge of the rock cut.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Stepped footing may be required</li> <li>ii. High cost of excavation, if any is required</li> <li>iii. Mass concrete fill required to create a level founding surface.</li> <li>iv. Groundwater control will be required.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher geotechnical resistance than native soil can provide.</li> <li>ii. Permits founding at elevations above original ground level.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower geotechnical resistance than bedrock</li> <li>ii. Cost of constructing engineered fill</li> <li>iii. Groundwater control will be required at the north abutment.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less expensive to construct.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>ii. Soil conditions at this site are not favourable.</li> </ul> <p>NOT RECOMMENDED</p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High bearing resistance</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Difficulties installing caissons through saturated cohesionless soils.</li> <li>ii. Proximity of bedrock surface and short length of caissons not likely to justify costs.</li> </ul>
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**Appendix E**

**Special Provisions**

## **AMENDMENT TO OPSS 120, AUGUST, 1994**

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### **Special Provision**

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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

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Special Provision

November 25, 2002

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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.



Suggested text to modify OPSS 501 for RSS construction.

501.08.02 Method A shall be replaced by the following:

501.08.02 Method A

Granular materials shall be compacted to 100% of the maximum dry density and all earth materials shall be compacted to 100% of the maximum dry density.

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

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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.

SUPPLY EQUIPMENT FOR DRIVING PILES - Item No.  
SUPPLY EQUIPMENT FOR INSTALLING CAISSON PILES - Item No.  
SUPPLY EQUIPMENT FOR INSTALLING DISPLACEMENT CAISSON PILES - Item No.  
SHEET PILES - Item No.  
H-PILES - Item No.  
TUBE PILES - Item No.  
WOOD PILES - Item No.  
PRECAST CONCRETE PILES - Item No.  
CAISSON PILES - Item No.  
DISPLACEMENT CAISSON PILES - Item No.  
DRIVING SHOES - Item No.  
ROCK POINTS. - Item No.  
RETAPPING PILES – Item No.

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Special Provision No. 903S01

October, 2002

#### 15.1.1.1 Piling

OPSS 903, December 1983, is deleted and replaced with the following:

#### **903.01 SCOPE**

This specification covers the requirements for the supply and installation of deep foundation units comprised of wood, steel, concrete or a combination of these materials.

#### **903.02 REFERENCES**

This specification refers to the following standards, specifications or publications:

#### **Ontario Provincial Standard Specifications, General:**

OPSS 180 Management and Disposal of Excess Materials

#### **Ontario Provincial Standard Specifications, Construction:**

OPSS 904 Concrete  
OPSS 905 Steel Reinforcement  
OPSS 909 Prestressed Concrete - Precast  
OPSS 911 Coating Structural Steel Construction

#### **Ontario Provincial Standard Specifications, Material:**

OPSS 1302 Water  
OPSS 1350 Concrete (Materials and Production)  
OPSS 1440 Steel Reinforcement for Concrete

#### **Canadian Standards Association Standards:**

CAN/CSA 3-G40.20/G40.21-M92 - General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Sheets

CAN3-056-M79 - Round Wood Piles

CSA 080 Series-M97 - Wood Preservation

W47.1-92 - Certification of Companies for Fusion Welding of Steel Structures

W48.1 - M1991 - Carbon Steel Covered Electrodes for Shielded Metal Arc Welding

W59 - M1989 - Welded Steel Construction (Metal Arc Welding)

**American Society for Testing and Materials Standards:**

ASTM A 252-93 Welded and Seamless Steel Pipe Piles

ASTM A 328/ A 328M-93A Steel Sheet Piling

**American Petroleum Institute:**

API 13A-86 Oil Well Drilling Fluid Materials

API 13B Standard Procedures for Field Testing Drilling Fluids

**903.03 DEFINITIONS**

For the purposes of this specification, the following definitions apply:

**Anvil:** means the component of a diesel hammer that acts as an impact block for the ram

**Bedrock:** means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic or sedimentary in origin which may or may not be weathered. The actual surface of the bedrock, weathered or unweathered, exists immediately below the overburden.

**Casing:** means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground with caisson piles that is structurally required and can be used to render a stable excavation hole.

**Caisson Pile:** means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

**Cap Block:** means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.



**Deep Foundation Unit:** means a structural member, driven or otherwise installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

**Displacement Caisson Pile:** means a pile formed in the ground by driving a casing or liner by means of a concrete plug or an expendable metal plate and replacing the displaced soil with plain or reinforced concrete.

**Driving Shoe:** means a reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate into a hard stratum.

**Driving to a Set:** means driving the pile to a penetration that satisfies pile driving criteria correlated to a required pile resistance

**Follower:** means a removable extension which transmits the hammer blows to the head of the pile.

**Helmet:** means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

**Jetting:** means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

**Liner:** means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

**Pile:** means a relatively slender structural element which is installed, wholly or partly in the ground by driving, drilling, auguring, jetting or other means.

**Pile Cap:** means a footing or some other structural component used to transfer the load to the piles as well as maintaining them in position.

**Pile Cushion:** means a pad of resilient material placed between the helmet and the top of a reinforced concrete or timber pile to minimize damage to the head during driving.

**Pile Group:** means the piles supporting a pile cap.

**Pumped Concrete:** means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

**Quality Verification Engineer(QVE):** means an Engineer who has a minimum of five (5) years experience in the field of installation of piling 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 to issue Certificate(s) of Conformance.

**Retapping:** means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration.

**Rock Points:** means a specially designed steel tip, fitted to piles to enable them to be driven into hard, sound sloped bedrock.

**Sheet Pile:** means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage.

**Slurry:** means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

**Stamped:** means drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer.

**Tremie:** means a hopper with a vertical pipe leading out of the bottom of it, used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete is always above water level.

#### **903.04 SUBMISSION AND DESIGN REQUIREMENTS**

All submissions shall bear the seal and signature of an Engineer experienced in this field. This Engineer, under this section, will not be permitted to carry out the work of the Quality Verification Engineer.

The Contractor shall submit to the Quality Verification Engineer for review and stamping, the equipment and installation procedure and the procedure for monitoring installation.

##### **903.04.01 Site Survey**

Prior to commencing the work, the Contractor shall submit to the CA, 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.

##### **903.04.02 Materials**

###### **903.04.02.01 Mill Certificates**

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

#### 903.04.02.02 Concrete

***Concrete and concrete work shall conform to OPSS 1350 and OPSS 904. The Contractor shall submit a suitable, site specific concrete mix design that meets the requirements of the hardened concrete specified. The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non segregating concrete that does not exhibit rapid slump loss. The concrete mix design shall be submitted to the Contract Administrator for information purposes only, one(1) week prior to construction.***

#### 903.04.02.03 Slurry

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

The type, source, physical and chemical properties of the bentonite or polymer.  
Slurry mix proportions and procedure.  
Quality Control Plan to control properties of slurry mix.  
Method of disposal.

#### 903.04.03 Installation

##### 903.04.03.01 Driven Piles

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

Type of equipment and hammer details including Contractors stated potential energy(rated energy) of the hammer, operating efficiency, weight of ram, anvil and helmet.

Procedure including sequence for pile installation.  
Procedure for monitoring installation

#### **903.04.03.02 Caisson Piles**

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

Shop drawings that describe and illustrate equipment, materials.  
Procedure for caisson excavation and construction.  
Procedure for monitoring installation and caisson inspection.

#### **903.04.03.03 Displacement Caisson Piles**

The Contractor shall submit, for information purposes only, one(1) week prior to construction:

1. Equipment to be used for installation.
2. Procedure for installation
3. Procedure for monitoring installation.

#### **903.04.03.04 Certificate of Conformance**

Upon completion of the deep foundation work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The certificate shall state that the work has been carried out in general conformance with the contract documents, specifications and stamped working drawings.

### **903.05 MATERIAL**

#### **903.05.01 Wood Piles**

Wood piles shall be according to CSA CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CSA 080.

Wood piles shall not be spliced.

#### **903.05.02 Steel Piles**

##### **903.05.02.01 Steel H Piles**

Steel H piles shall be according to CSA G40.20/G40.21 and shall be 350 W grade.

#### **903.05.02.02 Steel Tube Piles**

Steel tube piles shall be according to ASTM A252 minimum Grade 2.

#### **903.05.02.03 Steel Sheet Piles**

Steel sheet piles shall be according to ASTM A328. Steel sheet piles shall not be spliced.

#### **903.05.02.04 Straightness Tolerance for Steel Piles**

All steel piles shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.

#### **903.05.03 Driving Shoes and Rock Points**

Rock points and driving shoes shall be as specified. Driving shoes shall transfer the driving stresses to the pile over the full cross-sectional area of the pile.

Where the contract shows details of "Splice and Driving Shoe Details for Steel 'H' Piles, the Contractor may substitute the Titus "H" Bearing Pile Point, Standard model, in place of the driving shoe details shown.

Where the contract shows details of "Oslo Points for HP310 H-Piles" the Contractor may substitute the Titus "H" Bearing Pile Point, Rock Injector model in place of the pile point details shown.

Welding of Titus Points shall conform to the manufacturer's specifications.

Where the Contractor elects to use any of the above substitutions, the cost shall be deemed to be included in the contract price for the appropriate item.

#### **903.05.04 Casing for Caissons**

Casings shall be according to ASTM A252 Grade 2. If welded they shall be welded by the electric arc method according to CSA W59.

The wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.

### **903.05.05 Steel Reinforcement**

Steel reinforcement shall be according to OPSS 1440.

### **903.05.06 Concrete**

#### **903.05.06.01 General**

Concrete shall be according to OPSS 1350.

#### **903.05.06.02 Tube Piles**

Concrete shall have a slump of 150 to 180 mm.

#### **903.05.08.03 Caisson Piles**

Concrete shall have a slump of 150 to 180 mm. When approved by the Contract Administrator in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal, and to prevent setting of concrete until after the liner is withdrawn.

### **903.05.07 Slurry**

#### **903.05.07.01 Solids**

Bentonite and polymers shall be according to API 13A.

#### **903.05.07.02 Slurry Composition**

Slurry shall be according to API 13B

### **903.05.08 Helmets and Striker Plates**

The head of piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

## **903.06 EQUIPMENT**

The hammers shall be capable of driving the piles and liners/casings to the prescribed depth or to the specified resistance without damage to portions that are not cut off.

## **903.07 CONSTRUCTION**

### **903.07.01 Subsurface Conditions**

A Foundation Investigation Report that describes the subsurface conditions for the project is available, as specified elsewhere in the Contract. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

Any interpretation of data or opinions expressed in the report are not warranted. Regarding the data presented in the report, although the raw measured data presented is warranted, the Contractor must satisfy itself as to the sufficiency of the information presented for the intended construction purpose and obtain any updating or additional information as required to facilitate the deep foundation works.

### **903.07.02 Transportation, Handling, Storage**

Piles, casings and reinforcing steel cages shall be transported, stored and handled in such a manner that damage and distortion is prevented and that the strength and integrity are maintained.

### **903.07.03 Driven Piles**

#### **903.07.03.01 Pile Driving Requirements and Restrictions**

Piles shall be installed at the locations indicated and to the set or depth specified without being damaged.

Damage to adjacent structures, utilities and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 7.5 m of concrete which has been in place for less than 72 hours.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated on the Contract Drawings.

Piles shall not be forced into their proper position by the use of excessive manipulation. Pile damage due to excessive driving shall be avoided.

## **903.07.03.02                      Splicing**

### **903.07.03.02.01      General**

Splices within 6 m of the pile cut-off shall be certified by the Quality Verification Engineer as being equal to the full strength of the pile. Any damaged material shall be cut-off prior to splicing. The certificate shall be sealed and signed by the Quality Verification Engineer and shall be submitted to the Contract Administrator.

### **903.07.03.02.02      H Piles, Tube Piles and Sheet Piles**

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H piles and steel tube piles may be spliced providing the pieces being spliced are not less than 3 m long. Splices in marine structures shall be located below the low water level unless otherwise encased in concrete.

Sheet piles shall not be spliced without approval by the Contract Administrator.

### **903.07.03.02.03      Precast Piles**

Precast piles shall only be spliced when specified and the splices shall only be made with approved mechanical splicing devices.

## **903.07.03.03                      Concrete in Steel Tube Piles**

Concrete in steel tube piles shall be placed according to the OPSS 904 requirements.

## **903.07.03.04                      Cutting Off Piles**

### **903.07.03.04.01      General**

Driven piles shall be cut to the elevation as specified in the contract.

The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.

### **903.07.03.04.02      Wood Piles**



Where wood piles are broomed, splintered or otherwise damaged below the cutoff elevation, the pile shall be considered defective and shall be replaced.

#### **903.07.03.05 Protective Coating for Steel H and Steel Tube Piles**

Exposed steel H and steel tube piles shall have a protective coating applied from an elevation 600 mm below the low water level or finished ground surface up to the top of the exposed steel.

The steel surfaces shall be cleaned according to SSPC-SP10 prior to application of a coal tar epoxy system which shall be according to OPSS 911.

#### **903.07.03.06 Reinforcing Steel**

Reinforcing Steel shall be installed according to OPSS 905.

The reinforcing steel cage shall be fabricated in one piece.

Welding of reinforcing steel and use of splices shall not be done unless specified in the contract.

#### **903.07.04 Caisson Piles**

##### **903.07.04.01 Installation - General**

Caissons shall be constructed as specified in the contract.

The final bearing elevation shall be as specified in the contract or shall be an elevation determined by the Contract Administrator. When permanent casings are not specified the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.

##### **903.07.04.02 Excavation**

Sidewall stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.

Excavation methods shall be such that the sides and bottoms of the hole are straight and free of loose material.

Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped with each step not greater than  $\frac{1}{4}$  the diameter of the bearing area.

#### **903.07.04.03 Unwatering**

Where unwatering is required, the Contractor shall effect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material, or prevent subsidence or ground loss that may adversely affect the work of adjacent structures.

#### **903.07.04.04 Backfilling Liners Left in Place**

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

#### **903.07.04.06 Concrete**

##### **903.07.04.06.01 General**

Concrete shall be placed in the caisson according to OPSS 904. Concrete shall be placed immediately following acceptance of the caisson hole by the QVE.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

Arching of concrete during casing withdrawal shall be prevented.

The QVE shall provide inspection throughout the concreting operation.

##### **903.07.04.06.02 Concrete Placed in the Dry**

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson, or where columns are cast integral with the caisson, to the elevation of the bottom of the column reinforcing cage. The concrete shall be vibrated for the last 1.5 m of the pour.

#### **903.07.04.06.03      Concrete Placed Under Water or Under Slurry**

Tremie or pumped concrete shall be carried out in one continuous operation. The Contractor shall carry out the tremie or pumping operation to ensure a continuous flow of concrete that prevents the inflow of water or slurry.

#### **903.07.04.07                      Reinforcing Steel**

The reinforcing steel cage shall be checked to ensure conformance to the approved shop drawings prior to installation and during concrete placement.

#### **903.07.05                              Displacement Caisson Piles**

##### **903.07.05.01                      General**

Work shall be carried out in accordance with displacement caisson pile suppliers installation procedures. A permanent liner shall be used when specified.

The pile shall not be extended below the specified pile tip elevation without approval in writing from the Contract Administrator.

#### **903.07.06                              Tolerances**

##### **903.07.06.01                      Driven Piles**

cut off  $\pm$  25 mm

deviation from vertical not more than 1 in 50, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1 in 75 in the direction of the span

the deviation from the specified inclination for battered piles shall not exceed 1 in 25

the centre of the pile at the junction with the pile cap shall be within 150 mm of that specified (measured horizontally) except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm(measured horizontally) in the direction of the span.

##### **903.07.06.02                      Caissons**

Cut off elevation  $\pm$ 25 mm

Horizontal location at cut-off not more than 5% of shaft diameter nor 75 mm

Vertical alignment not more than 2% of the caisson length from vertical for

vertical caissons, nor 2% of the caisson length from the specified inclination for battered caissons

## **903.08                      QUALITY CONTROL**

### **903.08.01                      Monitoring Driven Piles**

#### **903.08.01.01                      General**

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile. All driving records shall be certified by the Quality Verification Engineer and submitted to the Contract Administrator.

#### **903.08.01.02                      Driving to a Set**

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group.

***The other piles shall be controlled by the pile penetration rate in blows per mm that correlates to the set.***

When new conditions such as change in hammer size, change in pile size or change in soil material occur, new sets shall be determined.

#### **903.08.01.03      Driving to Bedrock**

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

#### **903.08.01.04      Hammer Performance**

When requested by the Contract Administrator, the Contractor shall verify the hammer performance using the Pile Driving Analyzer or other approved equivalent. The Contractor shall provide all instrumentation, related access and assistance for the testing and monitoring as directed by the Contract Administrator.

***Hammer performance shall be verified to ensure that the actual potential energy is not less than 90% of the stated potential energy.***

#### **903.08.01.05      Retapping Tests on Piles**

In each pile group, 10% of the piles (actual number of piles to be rounded off to higher number) but no fewer than two piles shall be retapped no sooner than 24 hours *after installation of the individual pile* to confirm the bearing resistance has been sustained.

Retapping of piles driven to bedrock is not required.

#### **903.08.01.06      Retapping/Redriving Piles**

Where the retapping tests indicate the bearing resistance has not been sustained, all piles in the group shall be retapped.

Where the retapping reveals that the bearing resistance of the piles has not been achieved, the piles shall be redriven to the specified resistance. Where piles have risen, the piles shall be redriven to the original depth.

#### **903.08.02 Inspection of Caisson Holes**

The caisson holes shall be inspected and approved by the QVE.

### **903.09 MEASUREMENT FOR PAYMENT**

#### **903.09.01 H Piles, Tube Piles, Wood Piles and Precast Concrete Piles**

Measurement is in metres of the piling left in place after cut-off.

#### **903.09.02 Sheet Piles**

Measurement is in square metres based on the driving lines specified and the length of piling left in place after cut-off.

#### **903.09.03 Driving Shoes and Rock Points**

Measurement is for each driving shoe and rock point specified and used.

#### **903.09.04 Caissons and Displacement Caisson Piles**

Measurement is in metres of the depth along the centre line between the approved bearing surface at the bottom and the specified elevation at the top.

#### **903.09.05 Retapping Piles**

Measurement is lump sum for retapping the piles above and beyond the minimum 10% but no fewer than two piles requirement for the pile group.

For measurement purposes a count will be made of the number of piles retapped above and beyond the minimum 10% but no fewer than two piles requirement and the number of piles in the pile group and a ratio will be determined.

Where retapping is not required above and beyond the minimum, no measurement for payment will be made for this item.

### **903.10 BASIS FOR PAYMENT**

#### **903.10.01 Supply Equipment for Installing Driven Piles - Item Supply Equipment for Installing Caisson Piles - Item**

### **Supply Equipment for Installing Displacement Caisson Piles - Item**

Payment at the contract price for the above items shall be full compensation for all labour, testing, equipment and material required to do the work.

It will be assumed, for payment purposes, that 50% of the work under this item has been completed when the satisfactory performance of the equipment has been demonstrated to the Contractor Administrator by the installation of one(1) pile. The remaining 50% will be paid on the satisfactory completion of the installation.

When the hammer performance is requested to be verified, all costs associated with this work will be included in the contract price when the energy delivered is less than 90% of the stated potential energy(rated energy) specified in the submission.

When the energy is greater than 90% of the stated potential energy(rated energy) stated in the required submission, the cost will be paid as extra work.

**903.10.02 H-Piles – Item**  
Tube Piles – Item  
**Precast Concrete Piles - Item**  
Wood Piles - Item  
Displacement Caisson Pile - Item  
**Caisson Piles - Item**  
**Driving Shoes - Item**  
**Rock Points - Item**  
Sheet Piles - Item

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material to do the work

Payment for redriving piles shall be at the contract price for the applicable item(s) above.

### **903.10.03 Retapping Piles – Item**

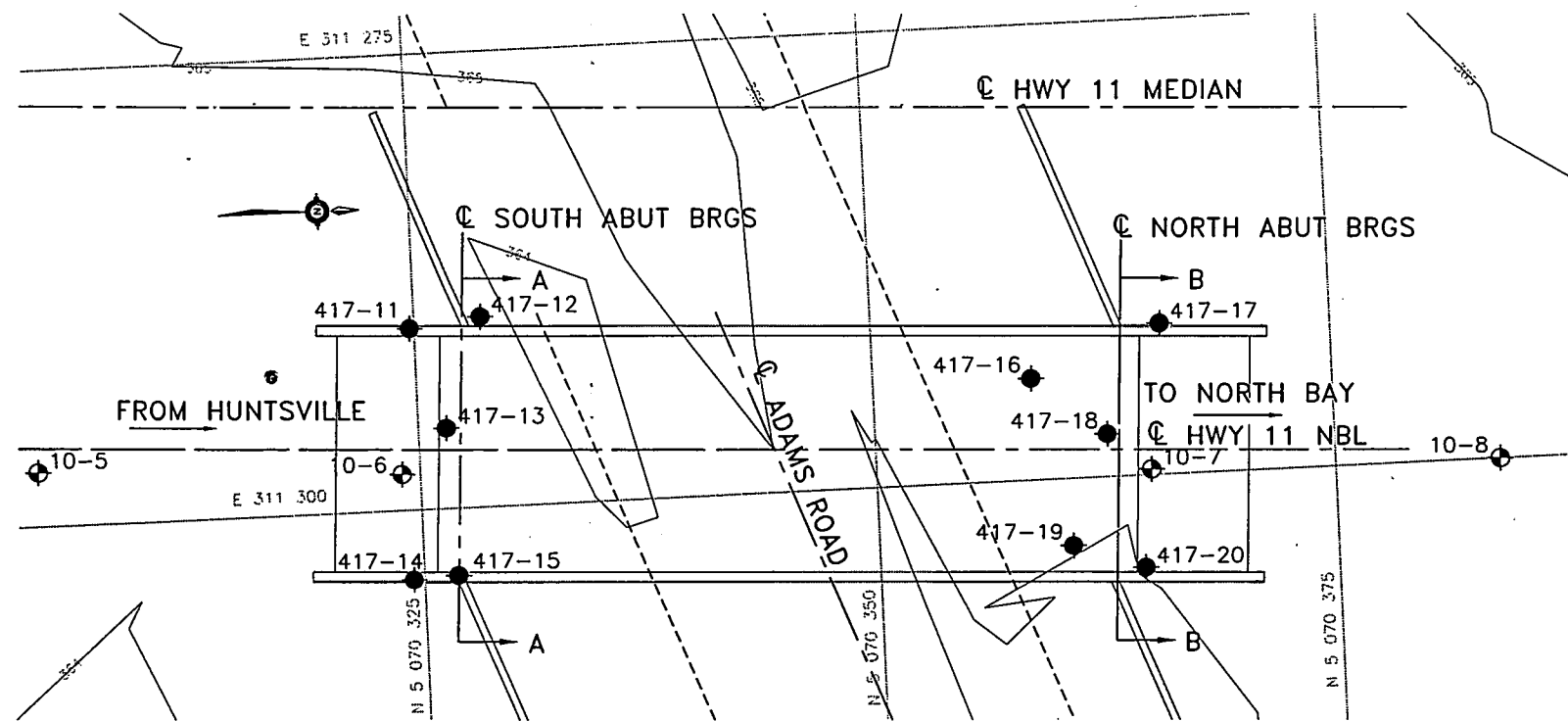
Payment for retapping the minimum specified number of piles is included in the Pile Item. Where additional retapping is required, payment will be made based on the ratio of the number of piles retapped in a pile group above the minimum requirement, to the total number of piles in that pile group, times the tender price for retapping all piles for that pile group.

WARRANT: Always with these tender items.

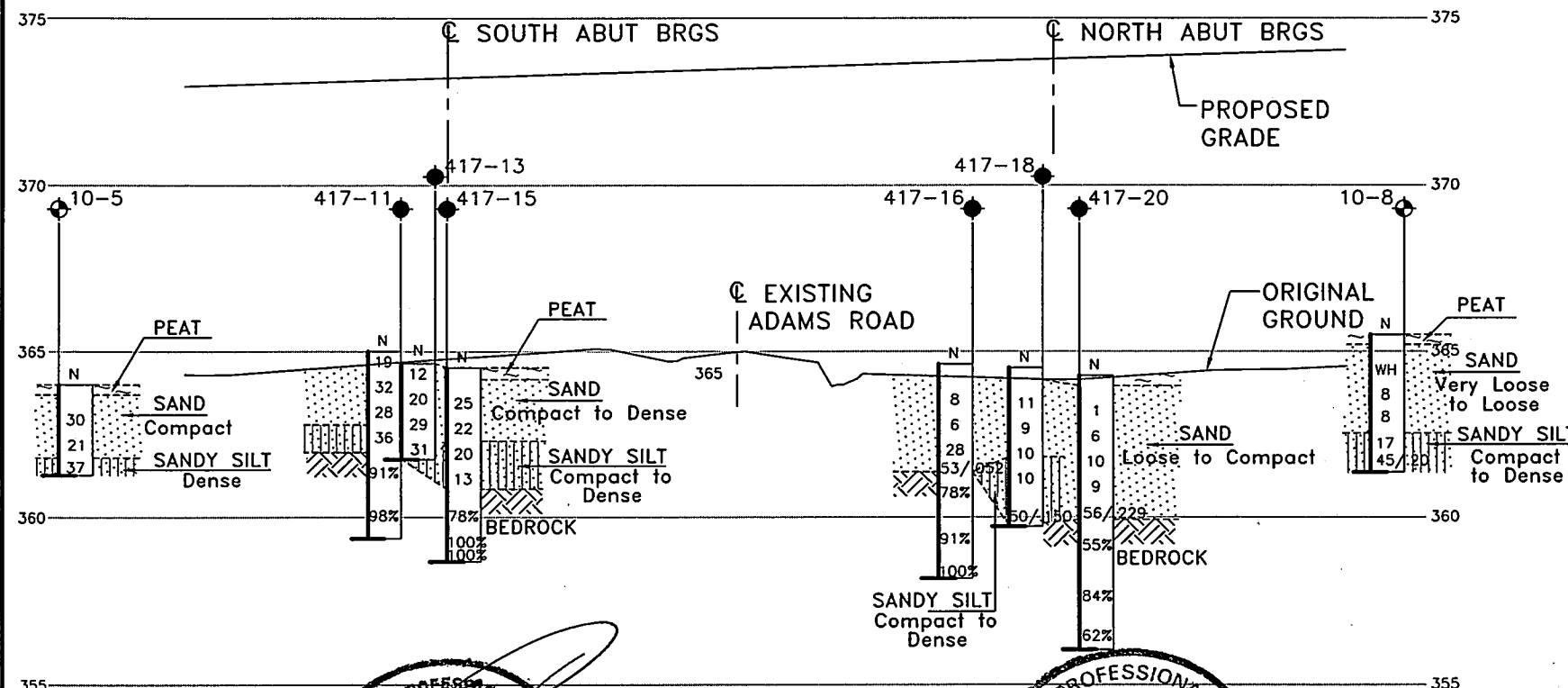


**Appendix F**

**Borehole Locations and Soil Strata Drawing**

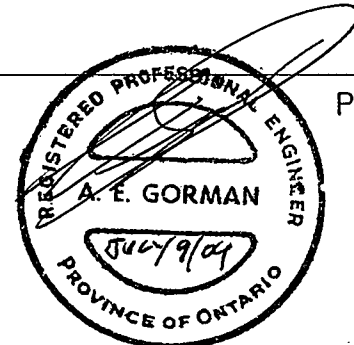


PLAN  
SCALE: 1:200



PROFILE  $\odot$  HWY 11 NBL

SCALE: 1:200  
HORIZ: 1:200  
VERT: 1:100



BENCH MARK  
Cut cross on rock outcrop  
58.30 LT of 21+984.8  
BM Elev. 367.946

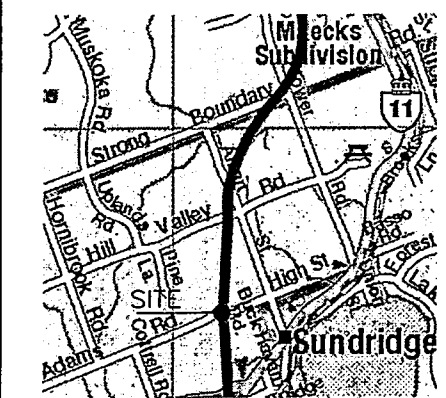
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

HWY 11  
CONT No  
WP No 745-93-01

STRONG TWP 10/11  
(ADAMS RD) OVERPASS NBL  
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



KEYPLAN

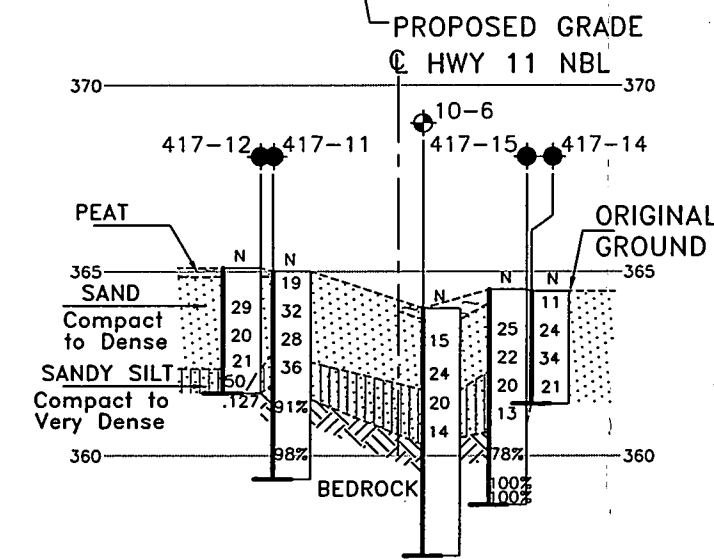
# LEGEND

- $\bullet$  Bore Hole by Thurber
- $\odot$  Bore Hole by Golder
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
417-11	365.0	5 070 324.8	311 290.0
417-12	365.1	5 070 328.6	311 289.5
417-13	364.6	5 070 326.5	311 295.6
417-14	364.5	5 070 324.4	311 303.9
417-15	364.5	5 070 326.8	311 303.7
417-16	364.6	5 070 358.6	311 294.4
417-17	364.6	5 070 365.7	311 291.7
417-18	364.5	5 070 362.6	311 297.6
417-19	364.4	5 070 360.5	311 303.7
417-20	364.3	5 070 364.4	311 305.1
10-5	364.0	5 070 304.0	311 297.0
10-6	364.0	5 070 324.0	311 298.0
10-7	364.5	5 070 364.9	311 299.7
10-8	365.5	5 070 384.0	311 300.0

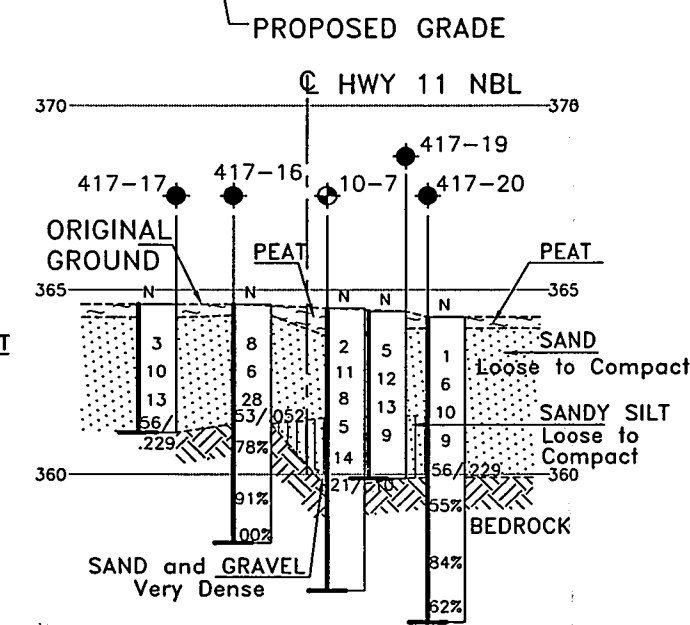
# NOTE

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



SECTION A-A

SCALE: 1:200  
HORIZ: 1:200  
VERT: 1:100



SECTION B-B

SCALE: 1:200  
HORIZ: 1:200  
VERT: 1:100

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN	AEG	CHK
DRAWN	SS	CHK
CODE	CHBDC 2000	LOAD 44-417
DATE	APRIL 2004	
ISCH	SCHEME	DWG 2