

**FOUNDATION INVESTIGATION
AND DESIGN REPORT
BOUNDARY ROAD/TOWER ROAD I/C UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
G.W.P. 759-93-00, W.P. 748-93-01, SITE 44-415**

Geocres Number: 31E-196

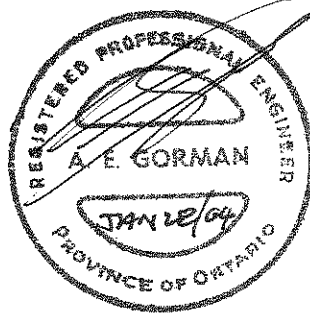
Report to

Marshall Macklin Monaghan

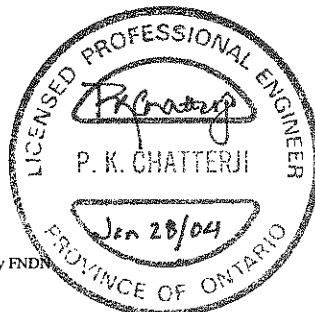
Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

January 28, 2004
File: 19-1423-12

AEG/C:\19\1423\12 Hwy 11\Bridges\415 Boundary Rd\Boundary FND\N
DRAFT.doc



Report prepared by:
Alastair E. Gorman, P.Eng.,
Senior Geotechnical Engineer



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

TABLE OF CONTENTS

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	2
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Topsoil	3
5.2	Sand and Gravel	3
5.3	Upper Sand	3
5.4	Silty Sand	4
5.5	Gravelly Sand	4
5.6	Lower Sand	4
5.7	Sandy Gravel	5
5.8	Bedrock	5
5.9	Water Levels	6
6	INTRODUCTION	8
7	STRUCTURE FOUNDATIONS	8
7.1	Spread Footings on Native Ground	9
7.2	Spread Footings on Engineered Fill	9
7.3	Lateral Resistance	10
7.4	Steel Piles Driven to Bedrock	10
7.4.1	Geotechnical Resistance	11
7.4.2	Pile Tips	11
7.4.3	Integral Abutment Considerations	11
7.4.4	Pile Spacing	11
7.4.5	Downdrag	12
7.4.6	Lateral Resistance of Piles	12
7.4.7	Pile Installation	13
7.4.8	Pile Driving	13

7.5	Recommended Foundation	14
7.6	Frost Cover	14
8	EXCAVATION	14
9	UNWATERING	14
10	APPROACH EMBANKMENTS	14
11	RETAINED SOIL SYSTEMS	15
11.1	Foundation	16
11.2	Global Stability	16
11.3	Internal Stability	17
12	BACKFILL TO STRUCTURES	18
13	EARTH PRESSURE COEFFICIENTS (ABUTMENTS)	18
14	SEISMIC CONSIDERATIONS	20
14.1	Seismic Design Parameters	20
14.2	Liquefaction Potential	20
14.3	Retaining Wall Dynamic Earth Pressures	20
15	CONSTRUCTION CONCERNS	21

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Foundation Comparison
Appendix D	Special Provisions

Drawings

Appendix E	Borehole Locations and Soil Strata
	Soil Strata



**FOUNDATION INVESTIGATION
AND DESIGN REPORT
BOUNDARY ROAD/TOWER ROAD I/C UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
G.W.P. 759-93-00, W.P. 748-93-01, SITE 44-415**

Geocres Number: 31E-196

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation at the Boundary Road Underpass structure on the proposed four-lane Highway 11 near Sundridge, Ontario. A previous, preliminary investigation had been carried out by Golder Associates Ltd. (Golder) at a prior location of the structure. The location of the original investigation is too far removed from the current location for that information to be used or incorporated in 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 from 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 lies at the intersection of the proposed four-lane Highway 11 and Boundary Road, which forms the boundary between Strong Township and Machar Township. The site lies approximately 5 km northwest of Sundridge and 4 km southwest of South River.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally, however, the site lies in a gently rolling area with the bedrock obscured by deposits of glacio-fluvial soils.

The area is generally wooded with only scattered houses along the present township roads.



3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out between April 11 and 22, 2003. The site investigation consisted of drilling and sampling a total of seven sampled boreholes to depths ranging from 6.7 to 26.2 m. The boreholes were numbered 415-1, 415-2, 415-3, 415-4, 415-4a, 415-4b and 415-5. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing.

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

All-Terrain Drilling Limited of Waterloo, Ontario supplied a CME 75 drill rig mounted on a Nodwell tracked carrier and conducted the drilling, sampling and in-situ testing operation. Hollow-stem auger drilling techniques were used to advance the boreholes through the overburden and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Three of the boreholes, BH 415-2, BH 415-3 and BH 415-4b, were advanced 3 m into bedrock by diamond coring.

Standpipe piezometers were installed in the three deep boreholes drilled at the foundation elements to allow monitoring of groundwater levels. In Boreholes 03-3 and 03-4, a 19 mm standpipe with a 1.5 m long piezometer tip was placed to the bottom of the borehole and filter sand placed for the bottom 3 m of the borehole. Above these levels, the native, cohesionless soils collapsed as the drill string was withdrawn. The borehole was topped up with sand drill cuttings. In Boreholes 03-2, the native cohesionless soils collapsed around the standpipe before any filter sand could be placed. Given the characteristics of the soil, it is anticipated that the piezometer will perform satisfactorily. The balance of the borehole, at the top, was backfilled using sand drill cuttings.

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

On completion of drilling and sampling, all boreholes 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. Since only non-cohesive soils were encountered, no other laboratory testing was deemed necessary. A total of thirteen 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.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general terms, the site was found to be underlain by topsoil and 17 to 23 m of non-cohesive soils overlying Pre-Cambrian bedrock.

5.1 Topsoil

Brown, sandy topsoil was encountered in all seven boreholes drilled at this site. The thickness of topsoil ranged from 200 mm at BH 415-1, 415-2 and 415-3 to 500 mm at BH 415-4 and 415-4a. The thickness of topsoil should be expected to vary across the site.

5.2 Sand and Gravel

Borehole 415-3 encountered a layer of sand and gravel below the topsoil. The layer is approximately 2.8 m thick and the underside lies at approximately Elevation 369.6.

Based on SPT values generally exceeding 100 blows for 0.3 m of penetration, the soil is described as very dense. However, the high SPT values are believed to be due to the presence of cobbles and boulders and the sand and gravel matrix may be less dense.

The soil is described as sand and gravel, trace silt, occasional cobbles and boulders, brown and non-cohesive. The natural moisture content was found to range between 16 and 17% and the silt is described as moist.

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B3, Appendix B.

5.3 Upper Sand

An upper layer of sand was encountered below the sand and gravel in Borehole 415-3 and below the topsoil in Boreholes 415-1, 415-2 and 415-4, 415-4a and 415-4b.

The thickness of the upper sand layer ranged from 3.2 m at Borehole 415-4 to 6.8 m at Borehole 415-3. The underside of the upper sand layer lies at elevations ranging from 362.8 at Borehole 415-3 to 366.0 at Borehole 415-4.

Based on SPT values ranging from 16 to 43 blows for 0.3 m of penetration, the sand layer is described as compact to dense. Occasional lower SPT values are attributed to sample disturbance and occasional higher values are attributed to the presence of a piece of gravel or a cobble at the tip of the spoon.

The soil is described as fine to coarse grained sand with trace gravel and trace silt and is non-cohesive. The results of drilling indicate the possible presence of cobbles and boulders in this

layer. The natural moisture content was found to range between 2 and 19% and the sand is described as dry to moist. The soil is brown in colour.

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B2, Appendix B.

5.4 Silty Sand

Below the upper sand layer, Boreholes 415-1 and 415-2 encountered a layer of silty sand.

The thickness of the silty sand layer ranged from 1.6 m at Borehole 415-2 to 2.6 m at Borehole 415-1, though the latter borehole did not fully penetrate this layer. The underside of the sand layer was at approximately Elevation 361.6 at Borehole 415-2.

Based on SPT values ranging from 14 to 32 blows for 0.3 m of penetration, the sand is described as compact to dense. The soil is described as sand, silty and is non-cohesive. The natural moisture content was found to range between 18 and 23% and the sand is described as wet. The soil is brown in colour.

Borehole 415-1 terminated in this layer

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B1, Appendix B.

5.5 Gravely Sand

A layer of gravely sand was encountered below the upper sand in Boreholes 415-3 and 415-4 and immediately below the topsoil in Borehole 415-5.

The thickness of the gravely sand layer ranged from 1.0 m at Borehole 415-5 to 2.1 m at Borehole 415-4. The underside of the sand layer ranged from Elevation 361.0 at Borehole 415-3 to Elevation 366.7 at Borehole 415-5.

Based on SPT values ranging from 20 blows for 0.3 m of penetration to values exceeding 100, the gravely sand is described as compact to very dense. The soil is poorly sorted and is described as sand, gravely, trace silt, with cobbles and boulders and is non-plastic. The natural moisture content was found to range between 2 and 7% and the sand is described as moist to wet. The soil is brown in colour.

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B3, Appendix B.

5.6 Lower Sand

Below the silty sand or gravely sand layers, all boreholes except Borehole 415-1 encountered a lower sand layer.

The thickness of the lower sand layer, where fully penetrated, ranged from 7.8 m at Borehole 415-3 to 14.8 m at Borehole 415-2. The underside of the sand layer ranged from Elevation 346.7 at Borehole 415-2 to Elevation 352.1 at Borehole 415-4b.

Based on SPT values ranging generally from 12 to 64 blows for 0.3 m of penetration, the sand is described as compact to very dense. The soil is described as sand, fine to coarse grained, trace silt, trace to some gravel and is non-cohesive. From time to time during the drilling, grinding and scraping sounds were heard and the augers were observed to vibrate or to rotate unevenly. These observations were interpreted to indicate the possible presence of cobbles and boulders in this layer. The natural moisture content was found to range between 3 and 22% and the sand is described as ranging from dry to wet. The soil is brown in colour.

Borehole 415-5 terminated in this layer.

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B1, Appendix B4.

5.7 Sandy Gravel

Below the lower sand layer, Borehole 415-3 encountered a layer of sandy gravel.

The thickness of the sandy gravel layer was approximately 3.3 m and the underside of this soil lay at Elevation 349.9.

Based on SPT values ranging from 66 to in excess of 100 blows for 0.3 m of penetration, the sand is described as very dense. The soil is described as sandy gravel and is non-cohesive. From time to time during the drilling, grinding and scraping sounds were heard and the augers were observed to vibrate or to rotate unevenly. These observations were interpreted to indicate the possible presence of cobbles and boulders in this layer. The natural moisture content was found to range between 10 and 12% and the sand is described as wet. The soil is brown in colour.

The Grain Size Distribution curves for selected samples of this soil are shown in Figure B5, Appendix B.

5.8 Bedrock

The soils described above were found to be underlain by granitic gneiss bedrock of the Pre-Cambrian Canadian Shield. The bedrock was proved by coring in Boreholes 415-2, 415-3, and 415-4b. The length of core extracted from the boreholes ranged from 2.8 to 3.5 m.

The rock is described as white to black granitic gneiss with mica flakes with pink banding visible in some sections of core. Orange staining was evident in the fractures and in the more weathered zones near the surface.

Core recovery values ranged from 29 to 100% and RQD values ranged from 0 to 85% as shown in the Table 5.1 at the end of this section.

The bedrock was found to be slightly to moderately weathered in the upper 1 to 2 m but was highly fractured at the east abutment, Borehole 415-4b. Based on Point Load Testing, the unconfined shear strength of the bedrock in Boreholes 415-2 and 415-3 was estimated to range from 8 MPa to over 200 MPa. The rock encountered in Borehole 415-4b was highly fractured but testing on intact portions of core yielded strengths of 80 MPa to over 200 MPa.

The bedrock surface was encountered at the elevations shown in Table 5.2.

Table 5.2, Bedrock Elevations

Borehole	Approximate Bedrock Elevation
BH 415-2	346.7
BH 415-3	349.9
BH 415-4b	352.9

The information collected at the three discrete points in the boreholes indicates that the bedrock surface dips from east to west, falling 6.2 m in a distance of approximately 88 m.

5.9 Water Levels

The groundwater level data recorded at this site is shown below

Table 5.3, Groundwater Depths (in metres) and Elevations

Date	BH 415-2		BH 415-3		BH 415-4b	
	Depth	Elev.	Depth	Elev.	Depth	Elev.
April 15, 2003	-	-	-	-	6.3	363.3
June 20, 2003	8.5	360.2	4.3	368.3	5.5	364.1

Table 5.1, Bedrock Characteristics

Borehole	Depth (m) from Ground Surface	Total Core Recovery (%)	Solid Core Recovery (%)	RQD (%)	Quality Descriptor	Fracture Index	Strength (MPa) on Selected Core	Strength Descriptor
BH 415-2	22.0 – 23.5	93	89	54	Fair	4.5	8 to 41	Moderately weak to moderately strong
	23.5 – 24.9	90	78	63	Fair	2.2	41 to 156	Moderately strong to very strong
BH 415-3	22.7 – 23.2	81	44	38	Poor	12	96	Strong
	23.2 – 24.7	77	74	60	Fair	5	126 to 184	Very strong
	24.7 – 26.2	95	93	85	Good	1	188 to 240	Very strong to extremely strong
BH 415-4b	16.7 – 17.3	100	75	0	Very poor	*	167	Very strong
	17.3 – 18.8	100	33	13	Very poor	*	80 to 162	Strong to very strong
	18.8 – 19.4	29	17	0	Very poor	*	219	Extremely strong
	19.4 – 19.9	74	26	15	Very poor	5	162	Very strong

*Denotes core that is too broken to measure a meaningful Fracture Index.

**FOUNDATION INVESTIGATION
AND DESIGN REPORT
BOUNDARY ROAD/TOWER ROAD I/C UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
G.W.P. 759-93-00, W.P. 748-93-01, SITE 44-415**

Geocres Number: 31E-196

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 INTRODUCTION

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

A two-span (44 m, 44 m), structure will be built at this site. The foundations will be skewed at 25°. The proposed finished grade over the structure will be Elevation 380.7 at the west abutment and Elevation 380.3 at the east abutment.

The new NBL grading will require only nominal fill. The SBL grading will require approximately 3 m of fill.

The Boundary Road approaches will be constructed on fill embankments that will be approximately up to 13 m high.

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.

7 STRUCTURE FOUNDATIONS

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

This section discusses the feasible foundation alternatives, provides geotechnical design parameters and recommends a preferred foundation scheme. The stratigraphy encountered at this site consists of 17 to 23 m of non-cohesive overburden deposits overlying bedrock.

Initial consideration was given to the following foundation types:

- Spread footings
- Driven piles

A table comparing the relative merits of these foundation types is included in Appendix C.

7.1 Spread Footings on Native Ground

The near surface soils at the potential founding elevations consist of generally compact sands. Locally, apparently denser sand deposits are due to the presence of cobbles and boulders.

The geotechnical resistances for spread footings bearing on native soil shown below were calculated using the following assumptions:

- Minimum footing width = 3 m
- Minimum depth of embedment of footing base = 1 m
- Groundwater level effectively at base of footing (assumption for design)

Foundation (Highest Founding Elevation)	Factored ULS (kPa)	SLS (kPa)	Settlement at SLS (mm)
West Abutment (BH 415-2) (Elevation 368)	500	300	30
	300	200	20
Centre Pier (BH 415-3) (Elevation 372)	500	300	30
	300	200	20
East Abutment (BH 415-4) (Elevation 368)	500	300	30
	300	200	20

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.

The anticipated settlements at SLS are shown above. Differential settlements are not expected to exceed 20 mm in a 6 m run.

7.2 Spread Footings on Engineered Fill

If it is beneficial to the overall design, spread footings may be founded on an engineered fill pad.

If an engineered fill pad is used at this site, all topsoil should be stripped from below the footprint of the footing and the native soil should be stripped at least to the following elevations, and deeper if required to achieve the minimum thickness of engineered fill.

Foundation Element	Elevation
West Abutment (BH 415-2)	368.4
Centre Pier (BH 415-3)	372.3
East Abutment (BH 415-4)	369.0

The thickness of engineered fill placed below the underside of the footing should be equal to at least twice the footing width. The founding surface for the engineered fill should be re-compacted. Acceptance of the re-compaction should be based on OPSS 501, Method A modified by a NSSP. Suggested wording for the NSSP is provided in Appendix D.

The engineered fill should be placed directly on prepared native soil surface and should consist of OPSS Granular “A” 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 the following concentric, vertical geotechnical resistance:

Factored ULS – 900 kPa

SLS – 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.

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 a 6 m run.

The thickness of engineered fill placed below the underside of the footing should be the greater of:

- 2 m
- a thickness equal to twice the footing width

7.3 Lateral Resistance

The lateral resistance of the footings may be computed using the following unfactored friction coefficients

- Native soil 0.50
- Granular “A” 0.55

7.4 Steel Piles Driven to Bedrock

The subsurface conditions at the site are considered to be suitable for the design of foundations supported on steel H-piles driven to bedrock.

Four steel pile sections believed to be currently available have been considered for use in the proposed foundations.

7.4.1 Geotechnical Resistance

The factored, vertical, concentric, geotechnical resistances at ULS for these 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

The SLS condition will not govern for piles founded in bedrock.

The structural resistance of the pile should be reviewed by the structural designer to ensure it does not exceed the values given in this section.

7.4.2 Pile Tips

The H-piles for the recommended foundation scheme will be driven to bedrock and will penetrate the thin layer of very dense sand and gravel, possibly containing cobbles, that overlies the bedrock. Due to the possibility of a locally sloping bedrock surface, it is recommended that the pile tips be fitted with rock points that provide reinforcement to the full pile section. A suitable rock point is that from Titus Steel Company (www.titussteel.com, 905 564 2446), or equal from another approved manufacturer.

7.4.3 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The near surface, native soils at this site are loose to compact and the lateral resistance of a pile in this soil might provide sufficient flexibility. However, the upper 3 m of the pile may lie partially within the compacted fill of the approach embankment and partially in the underlying native soils, which may become more dense under the embankment loading. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures. The use of concentric CSPs is preferred.

7.4.4 Pile Spacing

Since the piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, minimum pile spacing should be reviewed in accordance with the requirements of Clause 6.8.9.2 of the CHBDC.

It should be noted that driving piles in close proximity in very dense soils can lead to heave or lateral displacement of piles already driven. Also, the impact of pile spacing on the lateral resistance of the piles must be considered, as discussed in Section 7.4.6 below.

7.4.5 Downdrag

Downdrag on the piles is not considered to be an issue at this site provided construction of the approach fills is carried out in advance of pile driving.

7.4.6 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:

$$k_s = f/z/D$$

where:

z = depth below ground surface in metres

D = pile width in metres

f = (kN/m^3) a factor from the table below

Location	Elevation	f (kN/m^3)
Borehole 415-2 (West Abutment)	368.7 to 365	2,000
	365 to 347	3,000
Borehole 415-3 (Centre Pier)	372.6 to 370	3,500
	370 to 353	2,000
	353 to 350	3,500
Borehole 415-4 (East Abutment)	369.6 to 366	1,500
	366 to 353	2,500

The lateral resistance assigned to a pile should not exceed the ultimate lateral resistance. The ultimate lateral resistance " p_{ult} " may be assessed from the expression:

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

Where $\gamma = 20 \text{ kN/m}^2$

z = depth below ground surface (m)

$K_p = 3.3$ (passive earth pressure coefficient)

Spring constant (K_s) and ultimate spring load (P_{ult}) values for numerical analysis of the integral abutment piles can be obtained by multiplying the respective k_s and p_{ult} values above

by the pile diameter or width (in a direction perpendicular to the pile movement) and the vertical distance between nodal points of the numerical model mesh along the pile.

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

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

Pile spacing	Multiplier 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	Multiplier 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 on conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

7.4.7 Pile Installation

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

The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in some soil strata at this site and to the possibility of piles reaching apparent refusal at higher elevations than anticipated. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

7.4.8 Pile Driving

The piles should be driven to bedrock. Note No. 6 from Article 3.3.3 Pile Driving Notes in the MTO Structural Manual should be used on the Foundation Drawing, i.e. "Piles to be fitted with rock points and driven into bedrock in accordance with 903S01".

7.5 Recommended Foundation

The use of H-piles at the abutments allows for the design of an integral abutment structure.

H-piles are also suitable for the support of the pier foundation.

From a geotechnical point of view, it is recommended that all foundations be supported on steel H-piles driven to bedrock.

7.6 Frost Cover

Pile caps and footings should be provided with a minimum of 1.9 m of soil cover as frost protection. If the pile caps are underlain by free-draining granular fill and the approach embankments are constructed using rock fill, frost protection is not required for the pile caps.

8 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classed as Type 3 soils. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1V:1H above the groundwater level. Excavation below the groundwater level without prior dewatering is not recommended.

9 UNWATERING

If any excavation is required that will penetrate below the groundwater level at the site, the groundwater level should be temporarily depressed prior to excavation. It is recommended that the groundwater level be temporarily depressed to 1 m below the deepest excavation.

The design of the unwatering system should be the responsibility of the Contractor. However, suitable systems that might be employed include pumping from filtered sumps for nominal penetration below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

10 APPROACH EMBANKMENTS

The approach embankments for this structure will be constructed over a 17 to 23 m deep deposit of non-cohesive sand (with some gravel layers) overlying bedrock.

Approach embankment construction using either earth fill or rock fill would be feasible on the foundation soils encountered at this site. Some settlement will occur under the loading imposed by the approach embankment fill but due to the non-plastic nature of the soils the settlement will be immediate in nature and essentially complete when construction of the fill is completed.

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 up to 1.25H:1V. Embankments constructed using granular material or select subgrade material will have stable side slopes at inclinations of up to 2H:1V. Earth fill embankments will also have stable side slopes at 2H:1V provided the fill has an angle of internal friction of at least 30°, as assumed in the stability analysis.

The requirement for the embankment fill to have an angle of internal friction of at least 30° should be noted and incorporated in the Contract documents.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. In each case the factor of safety against global failure was greater than 1.5.

It is recommended that the topsoil be stripped prior to construction of the approach fills. 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 D.

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

- 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 should be incorporated at a height of 6 m below the subgrade level. The berms should:

- 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

If retaining walls are required at this site, retained soil system (RSS) walls may be used, subject to the requirements presented in this section.

RSS walls should be specified to be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and

base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

11.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system.

At this site, it is recommended that the levelling pad for the RSS wall be centred on top of a mat of engineered fill consisting of OPSS Granular A compacted in accordance with OPSS 501, Method A. The engineered fill mat should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

In addition to the requirements for the levelling pad, the RSS should be founded on approach fill compacted in accordance with the Contract requirements or on native soil. In the latter case, the native soil under the RSS foundation should be re-compacted. Acceptance of the re-compaction should be based on OPSS 501, Method A modified by a NSSP. Suggested wording for the NSSP is provided in Appendix D.

The following parameters may be used in design of the RSS:

- Bearing resistance for levelling pad on engineered fill:
 - Factored ULS 320 kPa
 - SLS 250 kPa
- Coefficient of sliding resistance of cast insitu concrete levelling pad on Granular A = 0.55
- Coefficient of sliding resistance of RSS mass on Granular A = 0.40
- Coefficient of sliding resistance of RSS mass on native soil = 0.35

11.2 Global Stability

The global stability of the RSS is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment.

Some selected geometries were analyzed to illustrate the global stability requirements. These selected geometries were:

- RSS founded at the base of the embankment, i.e. at the level of the mainline grade
- RSS founded in a 2H:1V earth slope
- RSS founded in a 2H:1V slope in engineered fill consisting of OPSS Granular A or Granular B Type II compacted to 100% of SPMDD

In all cases, it was assumed that the fill behind the RSS was horizontal.

For the purposes of this analysis, the following geotechnical properties were assumed:

Material	Cohesion (kPa)	Angle of Internal Friction	Unit Weight
Embankment fill	0	30°	21 kN/m ³
Engineered fill	0	35°	22 kN/m ³
Native soil	0	30°	21 kN/m ³

It was further assumed that, at this site, the groundwater table was below the depth of analysis.

In the analyses, parameters were adjusted to produce a factor of safety of 1.5 against global failure. The results of these trial analyses are summarized below:

Height of RSS (h)	Position in Embankment	Foundation Material	Length of RSS reinforcement	Factor of Safety
7 m	At base	Native	4.0 m (60% h)	1.54
9 m	At base	Native	5.5 m (60% h)	1.50
2 m	2 m up slope	Engineered fill	3.0 m (150% h)	1.67
4 m	2 m up slope	Engineered fill	5.0 m (125% h)	1.52

Analyses carried out on RSS walls located up a slope in the embankment earth fill indicated difficulty in achieving an acceptable factor of safety of 1.5 using the conventional anchor length. Such RSS walls must be designed with extended anchor lengths as indicated in the table above.

In summary, it may be assumed that RSS walls founded at the base of the embankment, i.e. not above the mainline grade, will be stable against global failure. If the RSS wall has to be founded in the embankment slope, the specific geometry and soil conditions must be analyzed to determine the requirements for global stability.

11.3 Internal Stability

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

12 BACKFILL TO STRUCTURES

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

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

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

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

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

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

13 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

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)

γ = unit weight of retained soil (see table below)

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

q = value of any surcharge (kPa)

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

Table 3 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 30^\circ, \gamma = 21 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 21 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.33	0.50*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.5	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.0	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral 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.

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

The site is treated as lying in Seismic Zone 2.

14.1 Seismic Design Parameters

The following seismic parameters should be used for design, based on Table 3.1.7 of the CHBDC, Burk's Falls:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1

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 foundations soils has been assessed using the Seed and Idriss (1971) method¹.

Using this method, it was determined that the foundation soils at this site are unlikely to undergo liquefaction.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction.

Some toe failure may occur in the approach embankments but this is expected to be minor in nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

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

¹ 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.

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

$$\begin{aligned}\phi &= 35^\circ \text{ for OPSS Granular A or Granular B Type II} \\ \phi &= 30^\circ \text{ for OPSS Granular B Type I} \\ \phi &= 42^\circ \text{ for rock fill} \\ \delta &= 50\% \text{ of } \phi\end{aligned}$$

Where ϕ = the angle of internal friction of the backfill and δ = the angle of friction between the wall and the backfill.

The seismic earth pressure coefficients to be used in design at this site are shown in Table 13.1 at the end of the text.

15 CONSTRUCTION CONCERNS

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

- The possibility of piles encountering boulders and not reaching bedrock
- Preparation of the founding surface for any RSS walls
- The nature and geotechnical properties of earth fill used in approach fills. A minimum angle of internal friction of 30° has been assumed in stability analysis and is required in the field for the analysis to be valid.

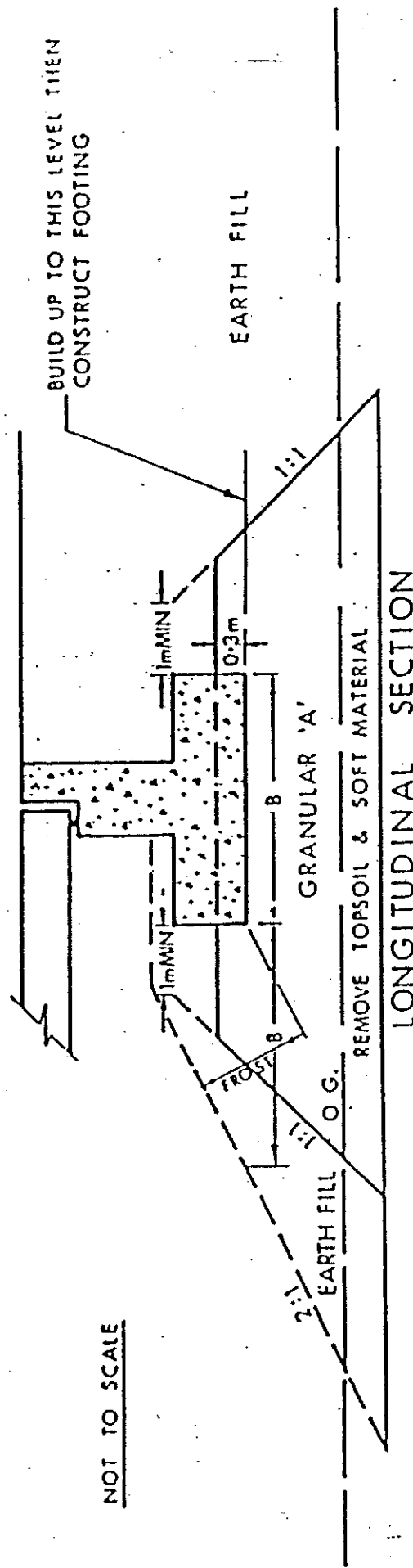
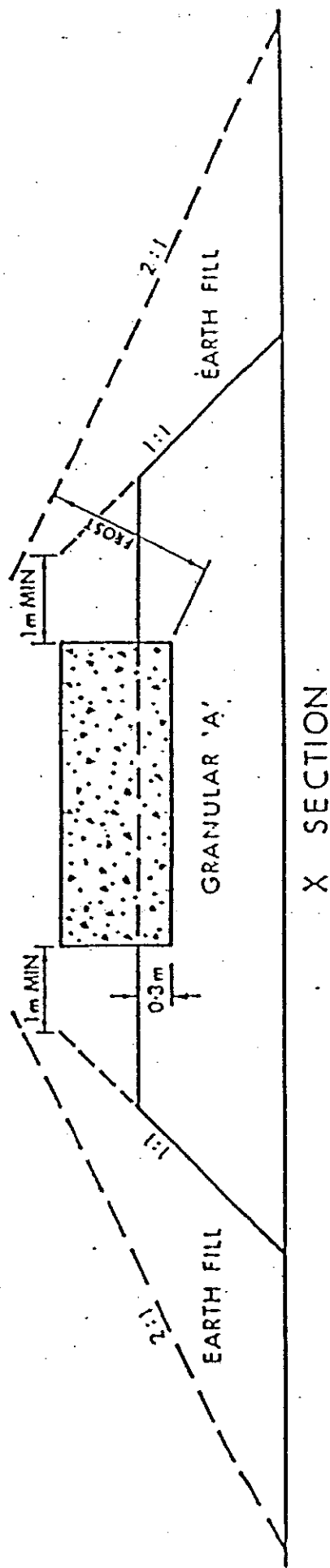
Table 13.1
Earth pressure Coefficients for Seismic Design

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 $\phi = 30^\circ, \delta = 15^\circ$		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})* Passive (K_{PE})* At Rest (K_{OE})**	0.30 6.3 0.59	0.45 6.3 0.66	0.36 4.3 0.66	0.63 4.3 0.33	0.31 12.0 0.33

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

** After Woods

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED

Figure 1

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 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 RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


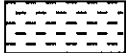
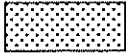

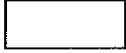
 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

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
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

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

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 200	Greater than 29,200	Requires many blows of geological hammer to break.
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-200	14,600 to 29,200	Requires a few blows of geological hammer to break.
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,300 to 14,600	Breaks under single blow of geological hammer.
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Moderately Strong	12.5 to 50.0	1,825 to 7,300	¼” indentations with sharp end of geological pick.
<u>TERMS</u>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Moderately Weak	5.0 to 12.5	730 to 1,825	Too hard to cut by hand into triaxial specimen.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Weak	1.25 to 5.0	182 to 730	Crumbles under firm blows of geological pick.
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.	Very Weak (Rock)	0.60 to 1.25	85 to 182	May be broken in the hand with difficulty.
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No BH 415-1

1 OF 1

METRIC

W.P. 748-93-01 LOCATION N 5 075 282.0 E 312 459.8 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 17.04.03 - 17.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100	20 40 60									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
367.3																		
0.0 367.1	Sandy TOPSOIL, some rootlets Brown																	
0.2	SAND, fine grained, trace to some silt Compact to dense Brown Dry		1	SS	17		367											
			2	SS	35		366											
			3	SS	25		365											
			4	SS	16		364											
363.2																		
4.1	Silty SAND, fine grained Compact Brown Wet		5	SS	24		363											
							362											
			6	SS	14		361											
360.6																		
6.7	END OF BOREHOLE AT 6.71m.																	

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

RECORD OF BOREHOLE No BH 415-2

1 OF 3

METRIC

W.P. 748-93-01 LOCATION N 5 075 289.2 E 312 484.2 SITE 44-415 ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel COMPILED BY SS
DATUM Geodetic DATE 16.04.03 - 17.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
368.7														
368.5	Sandy TOPSOIL													
0.2	Brown SAND, fine grained, trace to some silt, trace gravel Compact to dense Brown Dry to damp		1	SS	18		368							2 87 11 (SI+CL)
			2	SS	21		367							
			3	SS	26		366							
			4	SS	30		365							
	Wet													
	Possibly very dense below elevation 364m		5	SS	54		364							
363.1							363							
5.6	Silty SAND, fine to very fine grained Dense Brown Wet		6	SS	32		362							0 58 42 (SI+CL)
361.6							361							
7.2	SAND, fine to coarse grained, trace silt, trace to some gravel, occ. cobbles and boulders Compact to very dense Brown Wet		7	SS	12		360							
			8	SS	16		359							

Continued Next Page

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

RECORD OF BOREHOLE No BH 415-2

2 OF 3

METRIC

W.P. 748-93-01 LOCATION N 5 075 289.2 E 312 484.2 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel COMPILED BY SS
 DATUM Geodetic DATE 16.04.03 - 17.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
							20 40 60 80 100												
<div>Occ. grinding of augers indicates possible presence of cobbles or boulders</div>			9	SS	52		358					○				1 92 7 (SI+CL)			
							357					○							
			10	SS	22		356					○							
							355					○							
							354												
			12	SS	54		353					○							
							352					○							
							351												
			14	SS	18		350					○							
							349												

Occ. grinding of augers indicates
possible presence of cobbles or
boulders

1 92 7
(SI+CL)

Continued Next Page

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

METRIC

[illegible]

RECORD OF BOREHOLE No BH 415-3

1 OF 3

METRIC

W.P. 748-93-01 LOCATION N 5 075 271.0 E 312 516.3 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel COMPILED BY SS
 DATUM Geodetic DATE 21.04.03 - 22.04.03 CHECKED BY PJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
372.6												
372.0	Sandy TOPSOIL Brown											
0.2	SAND and GRAVEL, some silt, trace rootlets. occ. cobbles and boulders Dense to very dense Brown Moist (GP/SP)	1	SS	50/ .000		372						
		2	SS	50/ .025		371						
		3	SS	36		370						
369.6												
3.0	SAND, fine to coarse grained, trace silt, occ. gravelly pockets Compact Dry	4	SS	16		369						21 75 4 (SI+CL)
	Occ. grinding of augers indicates possible presence of cobbles or boulders	5	SS	22		368						
		6	SS	19		366						0 96 4 (SI+CL)
		7	SS	43		365						
		8	SS	67		363						
362.8												
9.8	Gravelly SAND, occ. cobbles and											

Continued Next Page

+ 3, x 3. Numbers refer to
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 415-3

2 OF 3

METRIC

W.P. 748-93-01 LOCATION N 5 075 271.0 E 312 516.3 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel COMPILED BY SS
 DATUM Geodetic DATE 21.04.03 - 22.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
							20 40 60 80 100	20 40 60 80 100	20 40 60					
361.0	boulders Wet		9	SS	50/ .127									
11.6	SAND, fine to coarse grained, some silt, some gravel to gravelly Dense to very dense Wet		10	SS	13									
			11	SS	47									
			12	SS	64									
			13	SS	42									
			14	SS	51									
353.2	Sandy GRAVEL Very dense Brown Wet													

Continued Next Page

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

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

METRIC

[illegible]

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 415-4

1 OF 2

METRIC

W.P. 748-93-01 LOCATION N 5 075 271.6 E 312 569.4 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 14.04.03 - 14.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
369.6								20 40 60 80 100							
0.0	Sandy TOPSOIL Brown							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
369.2								20 40 60 80 100							
0.5	SAND, medium to coarse grained, trace silt, trace gravel, occ. cobbles Loose to compact Brown Dry		1	SS	7		369								
			2	SS	17		368								
			3	SS	12		367								3 94 3 (SI+CL)
			4	SS	9		366								
366.0							365								
3.7	Gravelly SAND Dense Brown Dry to damp		5	SS	30		364								
363.9							363								
5.8	SAND, fine to coarse grained, trace to some gravel, trace silt Loose to dense Brown Wet		6	SS	16		362								0 93 7 (SI+CL)
			7	SS	9		361								
			8	SS	42		360								

Continued Next Page

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

METRIC

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

[illegible]

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 415-4a

1 OF 1

METRIC

W.P. 748-93-01 LOCATION N 5 075 272.6 E 312 570.4 SITE 44-415 ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 15.04.03 - 15.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
369.6	TOPSOIL Brown													
369.2														
0.5	SAND, fine to coarse grained, trace silt, trace gravel, occ. cobbles Brown													
	Interpreted from augering													
364.2														
5.5	Gravelly SAND, occ. cobbles and boulders Brown													
	Interpreted from augering													
362.9														
6.7	END OF BOREHOLE AT 6.71m. AUGER REFUSAL AT 6.71m													

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 415-4b

2 OF 3

METRIC

W.P. 748-93-01 LOCATION N 5 075 271.6 E 312 567.9 SITE 44-415 ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel COMPILED BY SS
DATUM Geodetic DATE 15.04.03 - 15.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60	W _p W W _L				
359														
358														
357														
356														
355														
354														
353	16.7 BEDROCK RUN#1: TCR=100%, SCR=75%, RQD=0% Highly fractured, white black and pink, orange staining in fractures, strong GRANITIC GNEISS Broken core from 17.1m to 17.7m RUN#2: TCR=100%, SCR=33%, RQD=13% FI=5 from 17.7m to 18.1m Broken core from 18.1m to 19.5m RUN#3: TCR=29%, SCR=17%, RQD=0% FI=5 from 19.5m to 19.9m RUN#4: TCR=74%, SCR=26%, RQD=15%		1	RUN										
352			2	RUN										
351			3	RUN										
350			4	RUN										
349.8														

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

Continued Next Page

+ 3, x 3 : Numbers refer to
Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

METRIC

SOIL PROFILE						SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	W _p	W	W _L	Y	GR	SA	SI	CL
								20 40 60 80 100								
19.9	END OF BOREHOLE AT 19.89m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
WATER LEVEL READINGS DATE DEPTH(m) ELEVATION(m) 16/04/03 6.3 363.3 20/06/03 5.48 364.1																

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH 415-5

1 OF 1

METRIC

W.P. 748-93-01 LOCATION N 5 075 265.8 E 312 580.5 SITE 44-415 ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 11.04.03 - 11.04.03 CHECKED BY PJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
368.1												
0.0	Sandy TOPSOIL Brown		1	GS			368					
367.7												
0.5	Gravelly SAND, fine to very fine grained, trace silt Compact Brown Dry		1	SS	20		367					
366.7												
1.5	SAND, fine to coarse grained, some gravel, trace silt Compact Brown Dry		2	SS	20		366					14 82 4 (SI+CL)
365.9												
2.2	SAND, fine to coarse grained Compact Brown Dry		3	SS	13		365					
365.2												
3.0	SAND, some gravel to gravelly, trace silt Compact Brown Dry to wet		4	SS	14		364					
			2	GS								
			5	SS	18		363					
			6	SS	22		362					20 78 2 (SI+CL)
361.4												
6.7	END OF BOREHOLE AT 6.71m.											

ONTMT4 415BOUNDARY RD.GPJ 02/02/04

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

Appendix B

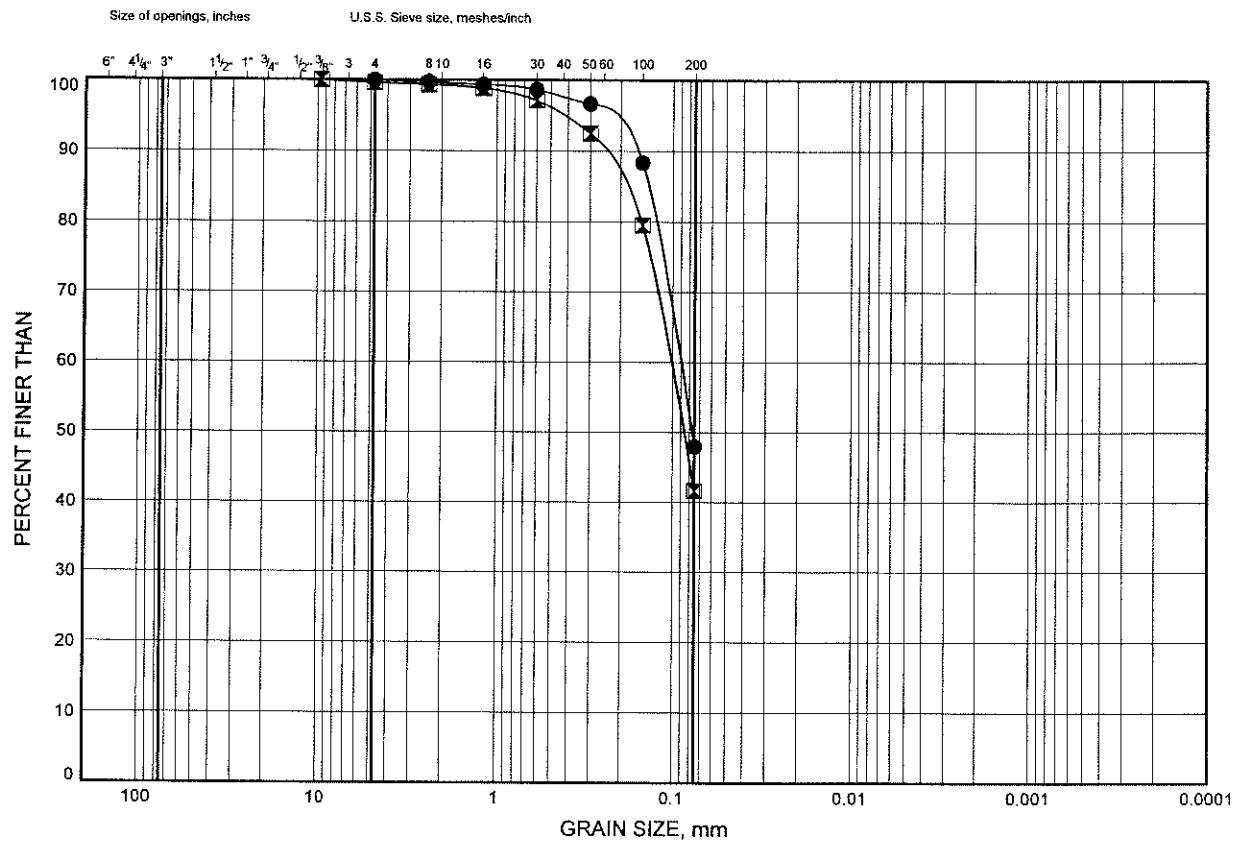
Laboratory Test Results

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY SAND

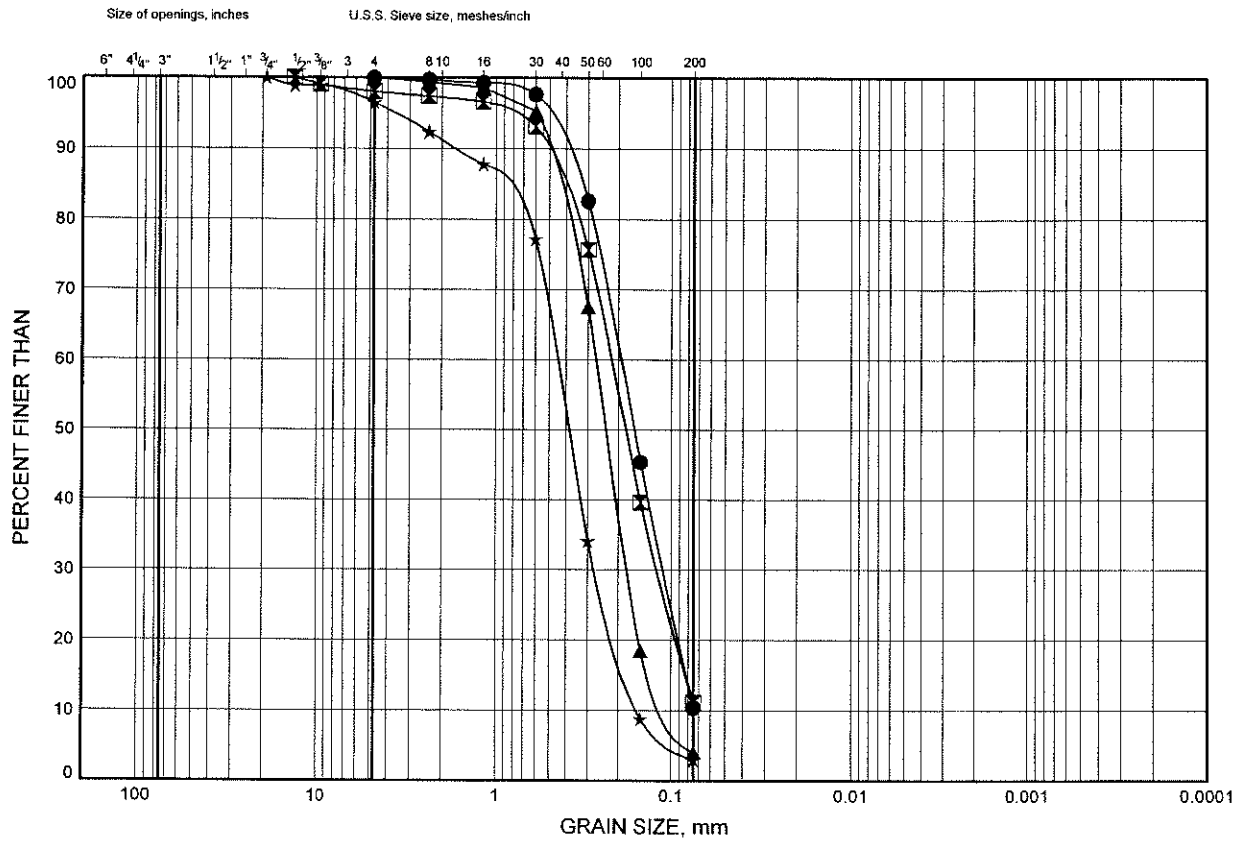


Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2

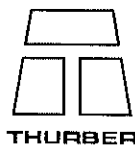
SAND, trace silt, trace gravel



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH 415-1	1.83	365.51
⊠	BH 415-2	1.07	367.67
▲	BH 415-3	6.40	366.18
★	BH 415-4	2.59	367.05

Date January 2004

Project 759-93-00



Prep'd SS

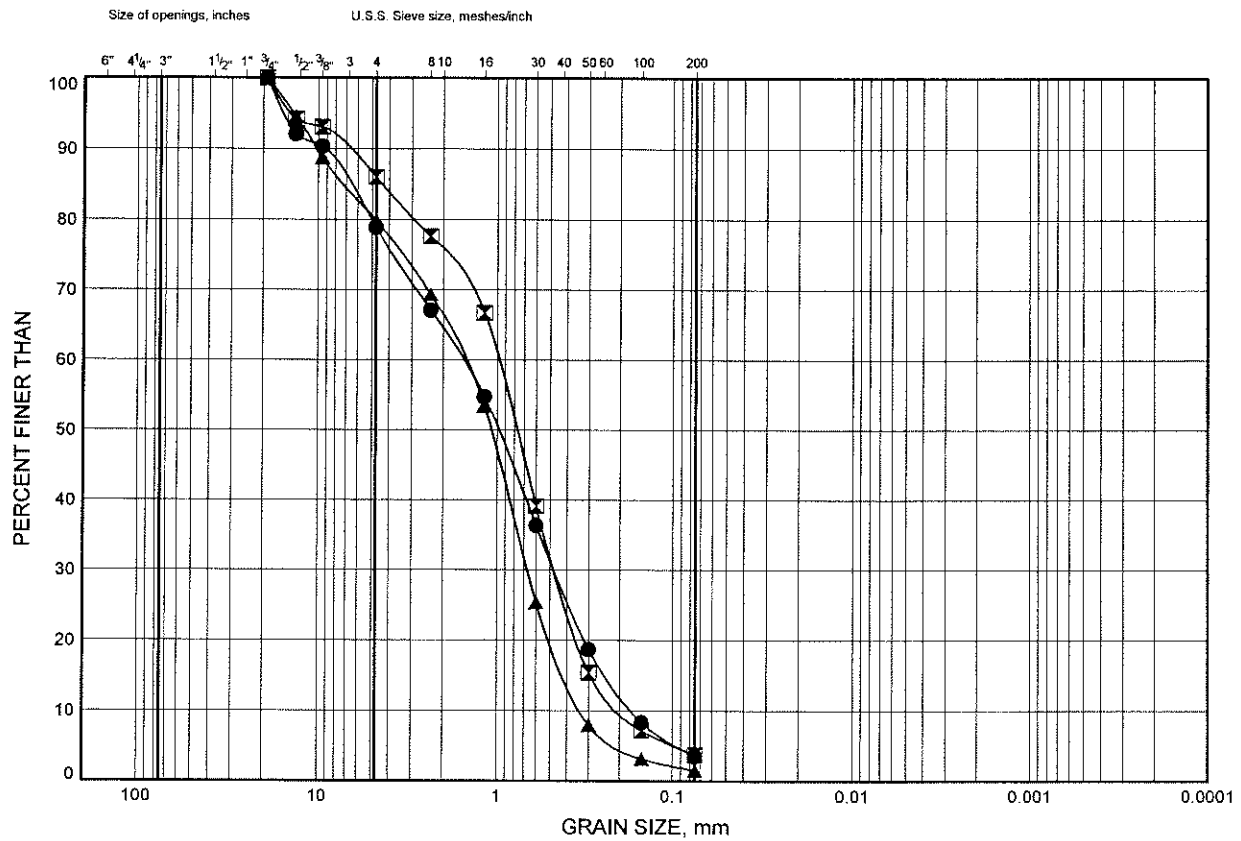
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3

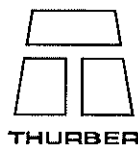
SAND, trace to some gravel, trace silt



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH 415-3	3.35	369.23
⊠	BH 415-5	1.83	366.29
▲	BH 415-5	6.40	361.72

Date January 2004
Project 759-93-00



Prep'd SS
Chkd. AEG

FIGURE B4

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (Solid Circles)	Percent Finer Than (Crosses)
10	100	100
4.75	100	100
2.0	97	100
0.85	93	100
0.425	81	98
0.25	53	76
0.15	19	25
0.075	5	5

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH 415-2	14.02	354.72
☒	BH 415-4	6.40	363.24

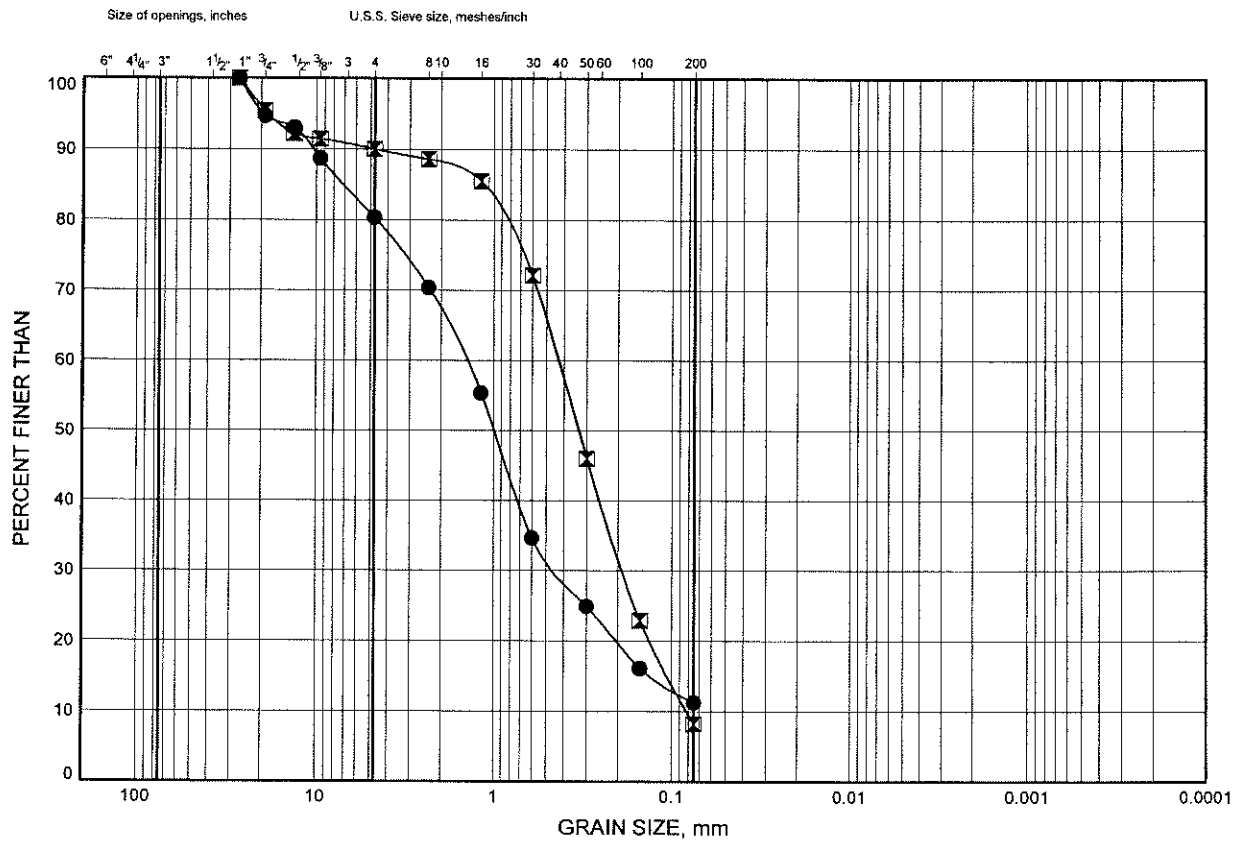
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND, trace to some gravel, trace silt



Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil	Footing on Engineered Fill
<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance if driven to refusal in the very dense soil. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Construction concerns related to the possibility of pile being obstructed by a boulder during driving. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Comparatively limited geotechnical resistance when compared to deep foundation systems. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows choice of conventional or semi-integral abutment. iii. Allows use of perched abutments. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> ii. Cost of constructing engineered fill. iii. Comparatively limited geotechnical resistance when compared to deep foundation systems.

Appendix D

Special Provisions

AMENDMENT TO OPSS 206, DECEMBER 1993

Special Provision

November 25, 2002

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

206.01 SCOPE

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

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

206.04 SUBMISSION AND DESIGN REQUIREMENTS

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

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

206.06 EQUIPMENT

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

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

206.07 CONSTRUCTION

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

206.07.01.03 Compaction

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

206.07.01.03.01 Compaction of Earth Embankments

Compaction of earth materials shall conform to OPSS 501.

206.07.01.03.02 Compaction of Rock Embankments

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

206.07.05 Rock Excavation, Grading

206.07.05.01 General

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

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

206.07.08 Rock Embankments

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

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

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

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

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

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

The materials shall be well distributed to form a solid embankment constructed to full width as the work progresses, or as stage construction allows.

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

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

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

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

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.

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.

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 ± 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.

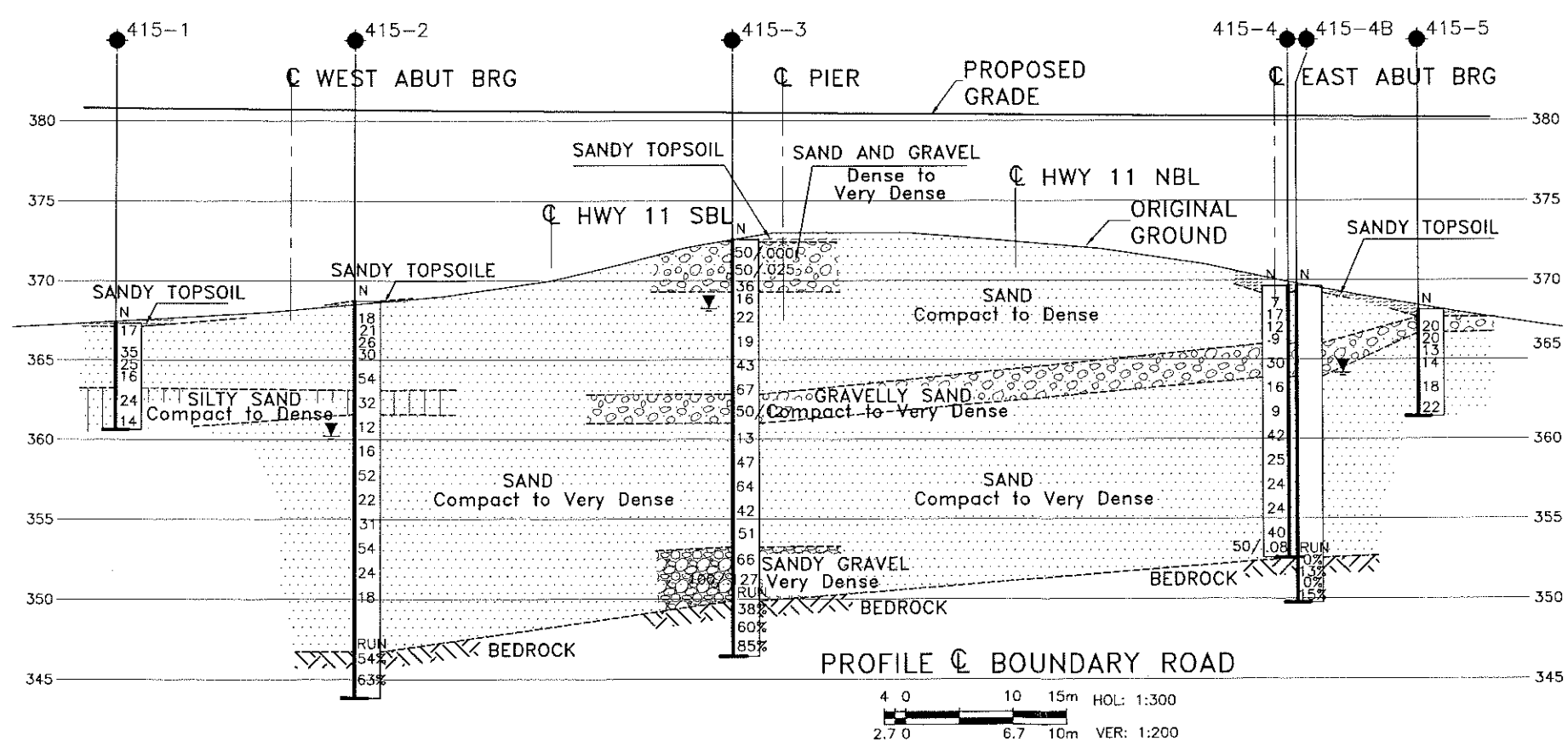
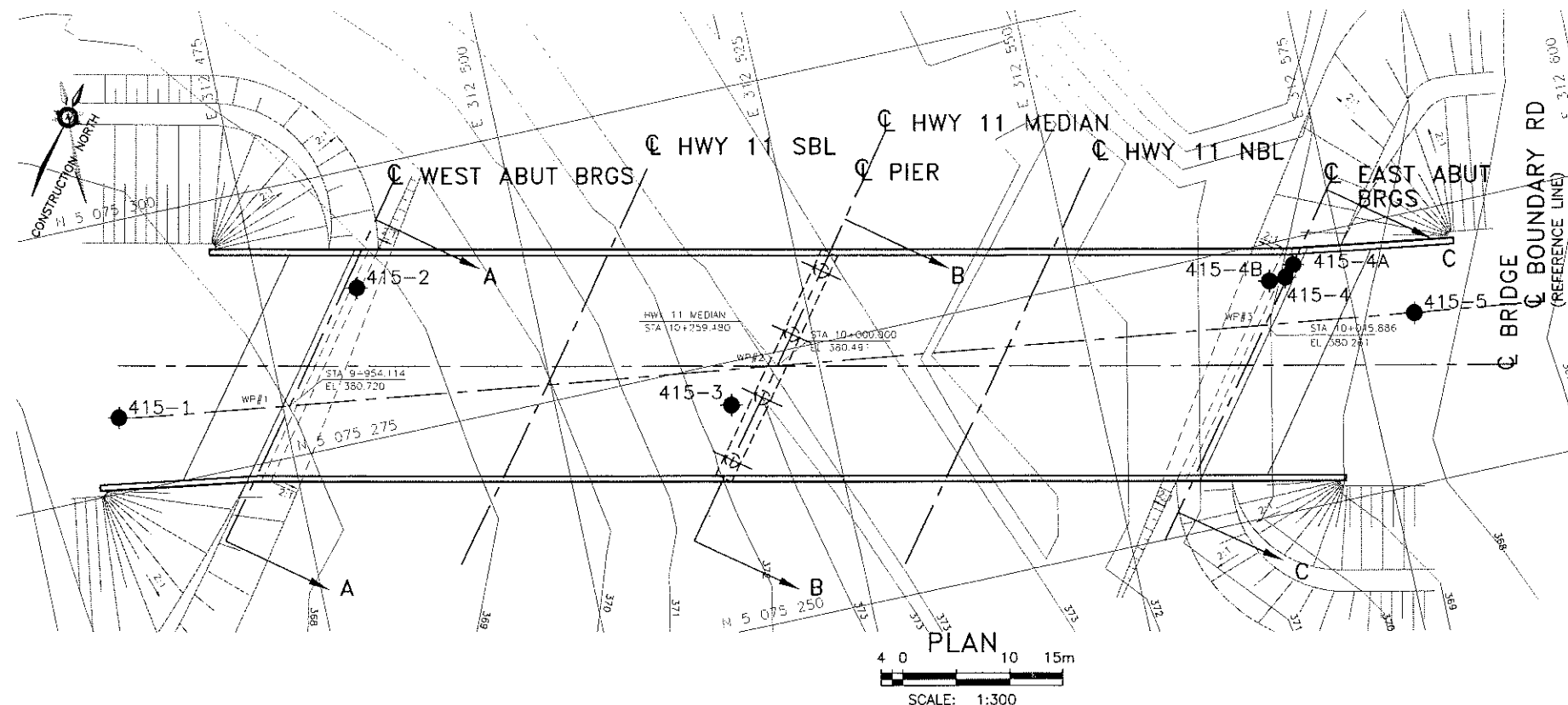
903.10.03 Retapping Piles – Item

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 E

Drawings



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

HWY 11
CONT No
WP No 748-93-01

BOUNDARY ROAD/TOWEL ROAD I/C
UNDERPASS

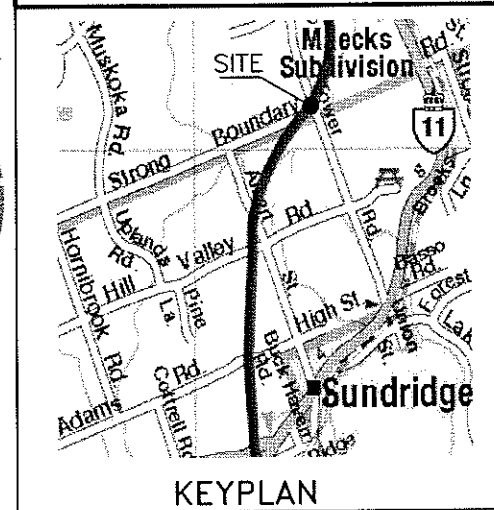
BOREHOLE LOCATIONS AND SOIL STRATA

Marshall Macklin Monaghan
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.
THURBER

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
JAN 28/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 28/04
PROVINCE OF ONTARIO



LEGEND			
●	BoreHole by THURBER		
⊕	Dynamic Cone Penetration Test (cone)		
N	Blows /0.3m (std pen Test, 475J/blow)		
CONE	Blows /0.3m (60' Cone, 475J/blow)		
PH	Pressure, Hydraulic		
WL	Head Artesian Water		
⊥	Piezometer		
90%	Rock Quality Designation (RQD)		
NO	ELEVATION	NORTHING	EASTING
BH 415-1	367.3	5 075 282.0	312 459.8
BH 415-2	368.7	5 075 289.2	312 484.2
BH 415-3	372.6	5 075 271.0	312 516.3
BH 415-4	369.6	5 075 271.6	312 569.4
BH 415-5	368.1	5 075 265.8	312 580.5
BH 415-4A	369.6	5 075 272.6	312 570.4
BH 415-4B	369.6	5 075 271.6	312 567.9
— NOTE —			
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.			

BENCH MARK

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

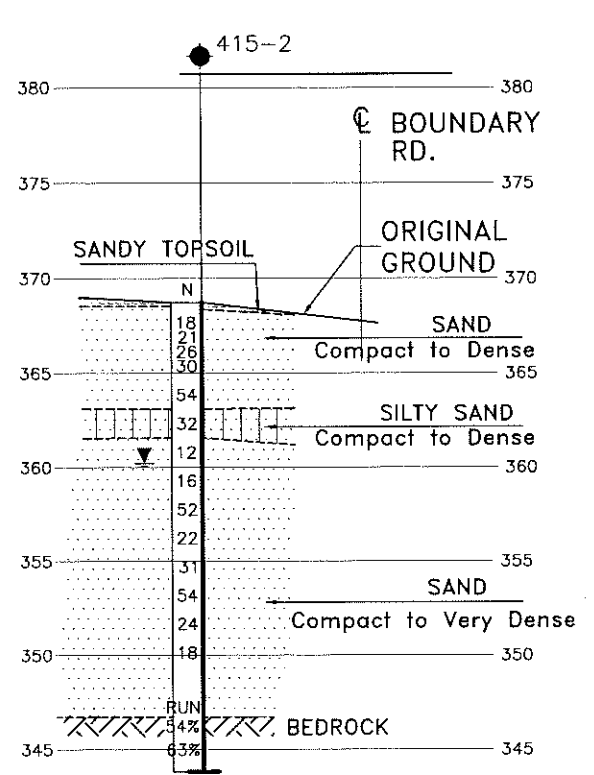
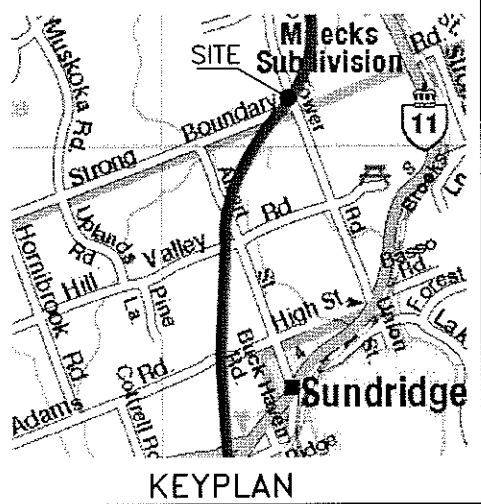
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AEG	CHK	CODE CH9DC 2000/LOAD R-625-0N/DATE JAN 2004
DRAWN	SS	CHK AEG	SITE 44-415/STRUCT./SCHEME/DWG

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

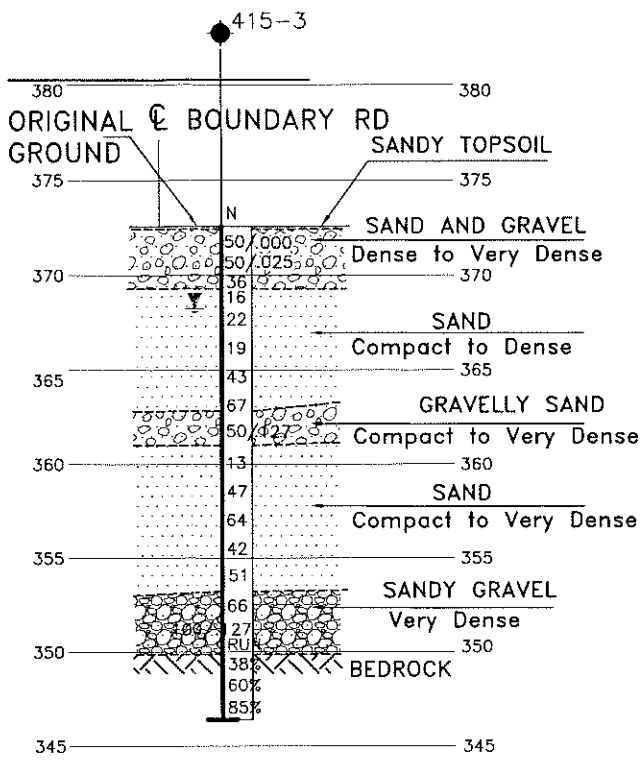
HWY 11
CONT No
WP No 748-93-01
BOUNDARY ROAD/TOWEL ROAD I/C
UNDERPASS
SOIL STRATA
SHEET

**Marshall Macklin Monaghan**
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

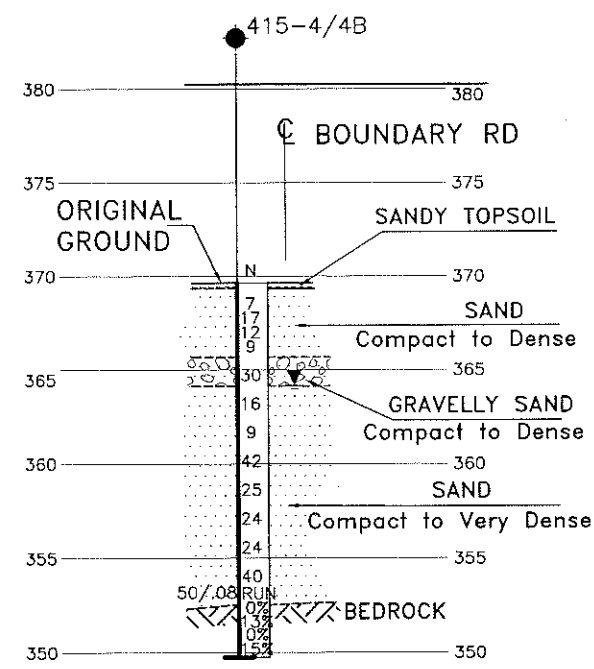
**THURBER ENGINEERING LTD.**







SECTION A-A
4 0 10 15m HOL: 1:300
2.7 0 6.7 10m VER: 1:200

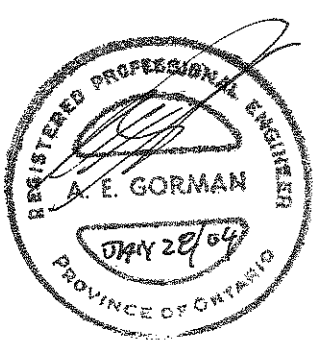


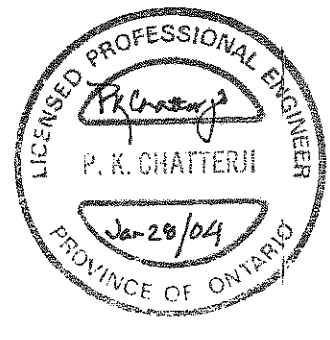
SECTION B-B
4 0 10 15m HOL: 1:300
2.7 0 6.7 10m VER: 1:200



SECTION C-C
4 0 10 15m HOL: 1:300
2.7 0 6.7 10m VER: 1:200

LEGEND			
	BoreHole by THURBER		
	Dynamic Cone Penetration Test (cone)		
N	Blows /0.3m (std pen Test, 475J/blow)		
CONE	Blows /0.3m (60° Cone, 475J/blow)		
PH	Pressure, Hydraulic		
WL	WL		
	Head Artesian Water		
	Piezometer		
90%	Rock Quality Designation (RQD)		
NO	ELEVATION	NORTHING	EASTING
BH 415-1	367.3	5 075 282.0	312 459.8
BH 415-2	368.7	5 075 289.2	312 484.2
BH 415-3	372.6	5 075 271.0	312 516.3
BH 415-4	369.6	5 075 271.6	312 569.4
BH 415-5	368.1	5 075 265.8	312 580.5
BH 415-4A	369.6	5 075 272.6	312 570.4
BH 415-4B	369.6	5 075 271.6	312 567.9
- NOTE -			
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.			





BENCH MARK

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DESCRIPTION	
DATE	BY	DATE	DESCRIPTION
DESIGN AEG CHK		CODE CHBDC 2000 [LOAD CL-625-INT]	DATE JAN 2004
DRAWN SS	CHK AEG	SITE 44-415	STRUCT SCHEME DWG