GEOCRES NO:
31E-195

FOUNDATION INVESTIGATION & DESIGN REPORT
PEVENSEY ROAD/STIRLING CREEK ROAD I/C
UNDERPASS
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
ONTARIO
G.W.P. 742-93-00, W.P. 743-93-01, SITE 44-422

Geocres Number: 31E-195

Report to

Marshall Macklin Monaghan

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
JAN 20/04
PROVINCE OF ONTARIO

Direction of fieldwork and engineering analysis by:
Alastair E. Gorman, P.Eng.,
Senior Geotechnical Engineer

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 20/04
PROVINCE OF ONTARIO

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

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation at the Pevensey Road Underpass structure over the proposed four-lane Highway 11 near Burk's Falls, Ontario. A previous, preliminary investigation had been carried out at the site by Golder Associates Ltd. (Golder) and the factual data from that investigation have been incorporated in the current assignment.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed through considering a combination of the data from the previous Golder investigation and the data obtained in the course of the present investigation.

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

2 SITE DESCRIPTION

The site lies at the intersection of existing Highway 11 and Pevensey Road/Stirling Creek Road, the Armour Township/Strong Township boundary road, and is approximately 6 km north of Burk's Falls.

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 comparatively flat to gently rolling area with soils characterized by glacio-lacustrine deposits.

The site environs drain to Stirling Creek to the west. Local drainage is comparatively well developed and the groundwater table was found to be in the order of 10 m below the ground surface.

The area is generally farmland, mostly pasture, and there are scattered houses close to the present intersection. South and west of the intersection, there is a commercial operation that apparently process wood waste into horticultural products.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between April 8 and 10, 2003. The site investigation consisted of drilling and sampling a total of four sampled boreholes to depths ranging from 6.7 to 22.3 m. The boreholes were numbered 422-1 to 422-4. In addition to these boreholes drilled by Thurber, four boreholes drilled by Golder as part of the preliminary design assignment were also used in the analysis. 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. Auger drilling and mud-rotary techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Two of the boreholes, BH 422-2 and BH 422–3, were advanced 3 m into bedrock by diamond coring, as was one of the boreholes, BH-G2–3, advanced by Golder in the preliminary investigation.

Standpipe piezometers were installed in the two deep boreholes drilled at the foundation elements to allow monitoring of groundwater levels.

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 (sieve and hydrometer). Since only non-cohesive soils were encountered, no other laboratory testing was deemed necessary. A total of nine samples were selected for these tests and the results are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and to the Record of Borehole sheets prepared by Golder and included in Appendix C. Golder's boreholes have been prefixed by "G" on the plan and profile. Details of the encountered soil and rock stratigraphy are presented in these appendices 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, sand and gravel fill and 12 to 19 m of non-cohesive soils overlying Pre-Cambrian bedrock.

5.1 Topsoil

Topsoil was encountered only in Boreholes 422-2 and 422-3, the balance of the boreholes having been drilled on the existing Stirling Creek Road or Pevensey Road. However, topsoil should be expected in the vicinity of the structure.

Borehole 422-2 encountered 100 mm of topsoil and Borehole 422-3 encountered 500 mm of sandy topsoil overlying 1 m of fill consisting of topsoil mixed with silt, trace clay.

5.2 Silt

All boreholes drilled at this site, except BH 422-1, encountered a layer of silt below the topsoil or fill, or below a thin veneer of silty sand.

The thickness of the silt layer increases in an easterly direction from 0.0 m at BH 422-1 to a maximum encountered thickness of 5.6 m at BH G2-2 at the east abutment. To the east of the east abutment, the thickness of silt decreases and the lower portion of the layer is replaced by sandy silt. The underside of the silt layer lies at elevations ranging from 331.8 to 325.0.

Based on SPT values generally ranging from 1 to 16 blows for 0.3 m of penetration, the silt is described as very loose to compact. The soil is described as silt, trace to some sand, trace clay and is non-plastic. The natural moisture content was found to range between 19 and 38% and the silt is described as moist to wet. The soil is brown in colour.

5.3 Sandy Silt

At the east portion of the site, BH 422-3, BH 422-4 and BH G2–2 encountered a layer of sandy silt underlying the silt layer.

The measured thickness of the sandy silt layer ranged from 1.5 m at BH G2-2 to 3.1 at BH 422-3. Borehole 422-4 terminated in the silt layer at a depth of 6.7 m. The underside of the sandy silt layer lies at elevations ranging from 323.5 to 323.0.

Based on SPT values ranging from 13 to 21 blows for 0.3 m of penetration, the sandy silt is described as compact. The soil is described as sandy silt, trace clay and is non-plastic. The natural moisture content was found to range between 15 and 18% and the sandy silt is described as moist. The soil is brown in colour.

5.4 Sand

Below the soils described above, a layer of sand was encountered in all boreholes except BH 422–4 and BH G2-2, which terminated at shallower depths.

The measured thickness of the sand layer ranged from 6.3 m at BH 422-3 to 8.7 m at BH 422-2. The underside of the sand layer ranged from Elevation 322.9 at BH G2-3 to Elevation 315.9 at BH G2-1

Based on SPT values ranging from 6 to 23 blows for 0.3 m of penetration, the sand is described as loose to compact. The soil is described as sand, trace silt to silty and is non-plastic. The natural moisture content was found to range between 3 and 20% and the sand is described as dry, wet below Elevation 320. The soil is brown in colour, becoming grey with depth.

5.5 Lower Sand with Cobbles and Boulders

The deeper boreholes, BH 422-2, 422-3, G2-2 and G2-3, encountered a lower layer of sand with gravel and occasional cobbles and boulders below the sand described above.

This lower sand comprises a comparatively thin layer on top of the bedrock and the measured thickness ranged from 0.8 m at BH G2-3 to 5.6 m at BH 422–3. Sampling in BH G2-2 stopped in this layer, but the results of a dynamic cone test from the bottom of the borehole indicates that the layer is 6.3 m thick. The underside of the layer ranged from Elevation 322.1 at BH G2-3 to Elevation 311 at BH 422–3.

Based on SPT values ranging from 11 blows for 0.3 m of penetration to values exceeding 100, the lower sand is described as compact with the higher SPT values ascribed to the presence of cobbles and boulders. The soil is poorly sorted and is described as sand, trace gravel to gravely, trace silt to silty and is non-plastic. The drilling observation indicated the presence of cobbles and boulders. The natural moisture content was found to range between 9 and 21% and the sand is described as wet. The soil is brown in colour.

5.6 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 422-2, 422-3, and G2-3 and was inferred from refusal to cone penetration in Borehole G2-2.

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

Core recovery values ranged from 82 to 100% and RQD values ranged from 46 to 90%.

The Fracture Index (FI) of the rock, expressed as the frequency of natural fractures per 0.3 m of core, was generally very low except at the surface. The first 0.3 m of core had FI values ranging up to 25 with the higher values occurring at the west abutment. Below the first 500 mm penetration into the bedrock, the FI values were found to be in the order of 1.

The condition of the joints ranged from planar to uneven and were generally rough though some smooth, planar joints were noted. The joints were mostly tight with no infilling or secondary weathering material.

The strength of the intact rock measured in Borehole 422-2 ranged from 110 to 190 MPa, with an average value of 155 MPa. In Borehole 422-3, the strength values ranged from 60 to 170 MPa, with an average value of 120 MPa. Based on the average values, the rock is classified as very strong.

The bedrock surface was encountered at the following elevations:

Borehole	Bedrock Elevation
BH 422-2	318.0
BH 422-3	311.0
BH G2-3	322.1
BH G2-2	309.5 (Inferred from dynamic cone test.)

5.7 Water Levels

The groundwater level data recorded at this site is shown below

Date	Depth/(Elevation) of Groundwater Surface		
	BH 422-2	BH 422-3	BH G2-2
Feb 9/00	-	-	13.3 (317.3)
Mar 8/00	-	-	0.3 (330.3) (frozen)
Mar 26/00	-	-	11.6 (319.1)
Apr 9 - 10/03	9.1 (321.3)	10.4 (319.7)	-
Jun 20/03	11.1 (319.3)	10.3 (319.8)	-

These water levels should be expected to fluctuate on a seasonal basis and after severe weather events.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 INTRODUCTION

This report presents 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.

The proposed bridge will consist of a two-span (44.5:44.5) underpass structure with a total of three foundation elements: two abutments and one pier. The foundation elements will be skewed at 5° with respect to the centreline of the structure. At the bridge site, the mainline of Highway 11 will run essentially at existing grade. Stirling Creek Road approaches from the west on an embankment up to 7.5 m high. Pevensey Road approaches from the east on an embankment up to 10 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 and available from the preliminary investigation.

7 STRUCTURE FOUNDATIONS

7.1 Foundation Alternatives

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 11 to 19 m of non-cohesive overburden deposits overlying bedrock.

Consideration has been given to the following foundation types:

- Spread footings on native soil or on engineered fill
- Driven piles
- Caissons

Appendix D contains a comparison, in tabular form, of the advantages and disadvantages of the different foundation types.

The near surface soils are considered to be unsuitable for the support of bridge foundations on shallow spread footings. The use of perched abutment footings on engineered fill was also considered and while possible, this is not recommended due to concerns related to the strength and settlement characteristics of the underlying loose to compact soils.

The subsurface soils are not considered suitable for the development of economical worthwhile resistance in a caisson. For higher geotechnical resistance, caissons to bedrock are possible, but it is considered likely that sealing and unwatering a caisson excavation would be problematic. Accordingly, caissons are not recommended.

The subsurface conditions at this site are considered suitable for the use of steel H-piles driven to bedrock. The use of piles would also allow development of an integral abutment design.

Accordingly, the recommended foundation type at this site is H-piles driven to bedrock.

7.2 Pile Resistance

The stratigraphy encountered at this site is considered to be suitable for the use of steel piles driven to bedrock to support the foundations. The use of H-section piles is recommended and the following concentric, vertical, geotechnical resistances (factored ULS) will be available for the pile sections shown.

Pile Section	ULS_f
HP 310 X 110	2,000 kN
HP 310 X 132	2,400 kN
HP 310 X 152	2,750 kN
HP 360 X 132	2,400 kN

The SLS case will not govern for piles driven to bedrock.

The following pile tip elevations should be used for design purposes.

Foundation Element	Pile Tip Elevation
West Abutment (BH 2-3)	322.1
Centre Pier (BH 422-2)	318.0
East Abutment (BH 2-2)	309.5

7.3 Pile Installation

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

The piles should be driven using a pile driving hammer capable of delivering an energy of at least 60 kJ per blow to the pile.

7.4 Pile Driving

The steel H-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".

If, during pile driving at the east abutment, it becomes apparent that the piles are developing high resistance but have not encountered bedrock then driving should be controlled by the Hiley formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate note is No. 2, i.e. "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance 'R' kN per pile but piles must be driven below: Elevation 315.0 at the east abutment."

"R" must have the following minimum values: 4,000 kN for HP 310X110; 4,800 kN for HP 310X132; 5,500 kN for HP310X152 and 4,000 kN for HP360X132.

In the Pile Data Table on the Foundation layout Drawing, Note (3) should specify a minimum hammer energy of 60 kJ.

The Contract should contain an NSSP alerting the Contractor to the fact that the piles may reach practical refusal in the gravelly sand with cobbles and boulders rather than reaching bedrock.

7.5 Pile Tips

It is anticipated that the piles will be driven to bedrock. Due to the fact that the piles may encounter a sloping bedrock surface (in the north-south direction) and that the overlying soil is loose to compact sand, all pile tips should be fitted with rock points from an approved manufacturer.

7.6 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 posses 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 will lie partially within the compacted fill of the approach embankment and the underlying loose native soils will be densified to some extent by the placement of this fill. 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 filled with sand in accordance with standard integral abutment design procedures.

7.7 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, as provided in the CHBDC Clause 6.8.9.2, pile interaction should be considered when the pile spacing is less than 750 mm or 2.5*b, where "b" is the width of the pile. The interaction effects are dealt with in Section 7.9 Lateral Pile Resistance.

7.8 Downdrag

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

7.9 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) and ultimate lateral resistance (p_{ult}):

$k_s = fz/D$ (kN/m^3)

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

where:

z = depth below ground surface in metres

D = pile width in metres

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

$\gamma = 22\ kN/m^3$

$K_p = 3.0$ (passive earth pressure coefficient)

Location	Elevation	f (kN/m^3)
Borehole G2-3 (West Abutment)	OGL to 323	1,400
	323 to 322	3,000
Borehole 422-2 (Centre Pier)	OGL to 327	1,000
	327 to 319	1,400
	319 to 318	3,000
Borehole 422-3 (East Abutment0	OGL to 326	1,000
	326 to 317	1,400
	317 to 311	3,000

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

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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.

7.10 Frost Cover

Pile caps and footings should be provided with a minimum of 1.8 m of soil cover as frost protection. If the pile caps are underlain by free-draining granular fill <u>and</u> the approach embankments are constructed using rock fill, frost protection is not required for the pile caps at this site.

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 silt at this site and any earth fill are classed as Type 3 soils.

9 APPROACH EMBANKMENTS

The approach embankments for this structure will be up to 10 m high at the east abutment and 7.5 m high at the west abutment and will be constructed over an 11 to 19 m deep deposit of silt and sand overlying bedrock.

Approach embankment construction using either earth fill or rock fill would be feasible on the foundation soils encountered at this site. 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. Settlements ranging from 60 to 120 mm are anticipated, with the larger settlements occurring under the east approach fill where the depth of loose soils is greater.

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, 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. An NSSP should be included in the Contract alerting the Contractor to this requirement and requiring that the QVE issue a certificate of conformance for the earth fill material prior to placement.

Global stability analyses were conducted for 2H:1V 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. Fill should be placed in accordance with OPSS 501.

Where earth fill embankments are higher than 8 m, a berm must be incorporated in the slope at 8 m below subgrade level, in accordance with OPSD 202.010. The berms should have positive grade of 2% laterally to promote drainage from the berm. In the case of rock fill embankments, 2 m wide berms should be incorporated at 6 m below subgrade. In either case the berm should extend for the length of the approach fill that exceeds the limiting height.

At this site, it is not considered necessary to wrap the berm around in front of the forward slope. It can be feathered out as it approaches the forward slope.

In view of the anticipated settlements, it is recommended that prior to pile driving construction of the embankment be completed as far as practical without interfering with pile driving or other construction activities.

10 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.

10.1 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 to 100% of its Standard Proctor maximum dry density (SPMDD) at ±2% of its optimum moisture content. 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 proof rolled and be compacted to 100% of its SPMDD at ±2% of its optimum moisture content.

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

10.2 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 that it is essentially impossible to achieve a factor of safety of 1.5 in a reasonable manner.

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.

10.3 Internal Stability

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

11 BACKFILL TO ABUTMENTS

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

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

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

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

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.

12 EARTH PRESSURE

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

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. In the case of an integral or semi-integral abutment design, the pressure on the ballast wall may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

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

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

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see below)

γ = unit weight of retained soil (typically 21 kN/m^3)

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

q = value of any surcharge (kPa)

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

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

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$		OPSS Granular B Type I $\phi = 30°$		Rock Fill $\phi = 42°$	
	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 or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consist of rock fill.

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

13 SEISMIC CONSIDERATIONS

The site is treated as lying in Seismic Zone 2. A peak horizontal acceleration factor $k_h = 0.12g$ (g is the acceleration due to gravity, 9.81 m/s^2) has been used in analysis. The value of k_h used in calculating the earth pressure coefficients is 67% of the peak value. The vertical acceleration factor k_v has been taken as 0.6 x k_h.

13.1 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 and a Peak Horizontal Acceleration (PHA) at ground surface of 0.11g should be used for design of the bridge.

13.2 Liquefaction Potential

The potential for liquefaction of the foundations soils has been assessed using the Seed and Idriss (1971) method[1].

[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.

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.

13.3 Retaining Wall Dynamic Earth Pressures

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

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

ϕ = 35° for OPSS Granular A or Granular B Type II

ϕ = 32° for OPSS Granular B Type I

ϕ = 42° for rock fill

δ = 50% of ϕ

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.

14 CONSTRUCTION CONCERNS

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

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

- Completion of the approach fills in advance of pile driving
- Control of pile driving
- Control of the earth fill material used in the approach fills

Table 13.1

Earth pressure Coefficients for Seismic Design

Condition	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient (K) for Earthquake Loading					
		Granular A or Granular B Type II $\phi = 35°$, $\delta = 17°$		OPSS Granular B Type I $\phi = 32°$, $\delta = 16°$		Rock Fill $\phi = 42°$, $\delta = 21°$	
		Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	36%	0.30	0.44	0.36	0.53	0.22	0.30
Passive (K_{PE})*	33%	6.9	6.9	5.4	5.4	13.8	13.8
At Rest (K_{OE})**	41%	0.43		0.47		0.33	

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

Appendix A

Record of Borehole Sheets

THURBER

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

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

$\underline{\blacktriangledown}$ Water Level

C_{pen} 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	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

- CLAYSTONE
- SILTSTONE
- SANDSTONE
- COAL
- BENTONITE

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 200	Greater than 29,200	Requires many blows of geological hammer to break.
Very Strong	100-200	14,600 to 29,200	Requires a few blows of geological hammer to break.
Strong	50-100	7,300 to 14,600	Breaks under single blow of geological hammer.
Moderately Strong	12.5 to 50.0	1,825 to 7,300	¼" indentations with sharp end of geological pick.
Moderately Weak	5.0 to 12.5	730 to 1,825	Too hard to cut by hand into triaxial specimen.
Weak	1.25 to 5.0	182 to 730	Crumbles under firm blows of geological pick.
Very Weak (Rock)	0.60 to 1.25	85 to 182	May be broken in the hand with difficulty.

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BH 422-1

METRIC

W.P. 743-93-01 | LOCATION N 5 059 230.0 E 310 721.4 (Pevensey Rd./Stirling Creek Rd. I/C Underpass) | ORIGINATED BY DP

HWY 11 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE 08.04.03 - 08.04.03 | CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
331.8 0.0	**SAND**, fine to very fine grained, Compact, Brown, Dry	1	GS			~15	
		1	SS	84	331	~12	
		2	SS	17	330	~5	
		3	SS	10	329	~4	0 98 2 (SI+CL)
	Occ. 1mm thick layers with some silt	4	SS	12	328	~5	
	Becoming grey	5	SS	10	327	~4	
					326		
325.1 6.7		6	SS	11		~3	
	END OF BOREHOLE AT 6.71m.						

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BH 422-2

METRIC

W.P.	743-93-01	LOCATION	N 5 059 251.2 E 310 782.6 (Pevensey Rd./Stirling Creek Rd. I/C Underpass)	ORIGINATED BY	DP
HWY	11	BOREHOLE TYPE	Hollow Stem Augers/NW Casing/NQ Core Barrel	COMPILED BY	SS
DATUM	Geodetic	DATE	08.04.03 - 09.04.03	CHECKED BY	PJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
330.4						
330.3 / 0.0	**ORGANICS**					
0.1	**SILT**, trace to some sand, fine to very fine grained; Loose to compact; Brown; Wet; (ML) (non-plastic)				330	
		1	GS			
					329	
		1	SS	6		0 3 90 7
	Trace clay				328	
		2	SS	12		
327.4 / 3.0	**SAND**, fine to very fine grained, trace to some silt; Loose to Compact; Brown; Dry					
		3	SS	12	327	
					326	
		4	SS	23		
					325	
		5	SS	18	324	
					323	
		6	SS	14		
					322	
		7	SS	20	321	0 91 9 (SI+CL)

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4 422STIRLING CK & PEVENSEY.GPJ 10/02/04

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BH 422-2

METRIC

W.P. 743-93-01 | LOCATION N 5 059 251.2 E 310 782.6 (Pevensey Rd./Stirling Creek Rd. I/C Underpass) | ORIGINATED BY DP

HWY 11 | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel | COMPILED BY SS

DATUM Geodetic | DATE 08.04.03 - 09.04.03 | CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
	Wet				320	
		8	SS	11		o (~22)
318.6 / 11.7	**SAND**, medium to coarse grained, some gravel Compact Brown Wet				319	
318.0 / 12.4		9	SS	55/ 0.00	318	o (~20)
	GRANITIC GNEISS (BEDROCK) Fresh to slightly weathered, very thinly to thinly bedded, black and pink with visible subvertical banding, very strong TCR=100%, SCR=98%, RQD=88% FI=1.1 quartz nodules from 12.6m to 41.9m and 12.8m to 13.1m subplanar and rough joint at surface, subvertical joint at 12.5m,14.5m, 14.6m, numerous horizontal fractures from 15.5m to 15.6m	1	RUN		317	
	TCR=95%, SCR=95%, RQD=84% FI=1	2	RUN		316	
314.7 / 15.7	TCR=92%, SCR=83%, RQD=46% FI=1	3	RUN		315	

END OF BOREHOLE AT 15.65m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEVATION(m)
09/04/03	9.1	321.3
20/06/03	11.1	319.3

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; STRAT PLOT; GROUND WATER CONDITIONS.

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BH 422-3

METRIC

W.P. 743-93-01
LOCATION N 5 059 264.8 E 310 829.3 (Pevensey Rd./Stirling Creek Rd. I/C Underpass)
ORIGINATED BY DP
HWY 11
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel
COMPILED BY SS
DATUM Geodetic
DATE 09.04.03 - 10.04.03
CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%) approx.	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
330.1						
0.0	Sandy **TOPSOIL** Brown					
329.7 0.5	**TOPSOIL** mixed with silt, trace clay Very loose to loose Dark brown Moist (FILL)	1	SS	4		
328.7 1.5	**SILT**, trace to some sand, fine grained, trace clay Loose to compact Brown Moist (ML)	2	SS	7		
		3	SS	16		0 9 82 8
		4	SS	15		
326.0 4.1	Sandy **SILT**, fine to very fine grained, trace silt Compact Brown Dry to moist (SP)	5	SS	19		0 44 51 4
		6	SS	14		
323.0 7.2	**SAND**, fine to very fine grained, trace to some silt Compact Brown Dry	7	SS	24		
		8	SS	29		0 69 31 (SI+CL)

ELEVATION SCALE: 330, 329, 328, 327, 326, 325, 324, 323, 322, 321

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 422-3

METRIC

W.P. 743-93-01 | LOCATION N 5 059 264.8 E 310 829.3 (Pevensey Rd./Stirling Creek Rd. I/C Underpass) | ORIGINATED BY DP

HWY 11 | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel | COMPILED BY SS

DATUM Geodetic | DATE 09.04.03 - 10.04.03 | CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
					320		
	Wet	9	SS	15	319		
		10	SS	6	318		
316.6	Iron oxide staining from 13.25m to 14.0m				317		
13.5	**SAND**, fine to coarse grained, some silt, some gravel, occ. cobbles Compact to very dense Brown, occ. iron oxide staining Wet	11	SS	50/ .127	316		
		12	SS	78	315		22 55 23 (SI+CL)
		13	SS	20	314, 313, 312		
311.0 19.1	**GRANITIC GNEISS BEDROCK** Slightly weathered, very thinly to thinly bedded, white and black with slight orange discoloration, with visible subvertical banding, very strong	1	RUN		311		

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BH 422-3

METRIC

W.P. 743-93-01	LOCATION N 5 059 264.8 E 310 829.3 (Pevensey Rd./Stirling Creek Rd. I/C Underpass)	ORIGINATED BY DP
HWY 11	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel	COMPILED BY SS
DATUM Geodetic	DATE 09.04.03 - 10.04.03	CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	subvertical joint at 19.9m to 20.4m, vertical joint from 20.1m to 20.3m, subplanar and rough joint surface Slightly weathered from 19.2m to 20.3m Slightly weathered at 20.4m RUN#1: TCR=96%, SCR=83%, RQD=78%, FI=1.3 RUN#2: TCR=98%, SCR=87.5%, RQD=87.5%, FI=0 RUN#3: TCR=82%, SCR=80%, RQD=67.4%, FI=1.2	2	RUN		310
					309
307.8		3	RUN		308
22.3	END OF BOREHOLE AT 22.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEVATION(m)
10/04/03	10.4	319.7
20/06/03	10.3	319.8

ONTMT4 422STIRLING CK & PEVENSEY.GPJ 10/02/04

$+^3, \times^3$: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH 422-4

METRIC

W.P. 743-93-01	LOCATION N 5 059 282.3 E 310 839.6 (Pevensey Rd./Stirling Creek Rd. I/C Underpass)		ORIGINATED BY DP
HWY 11	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY SS
DATUM Geodetic	DATE 09.04.03 - 09.04.03		CHECKED BY PJB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
331.7							
0.0	**SAND AND GRAVEL** Brown						
331.1 / 0.6	Silty **SAND**, fine to very fine grained Brown Moist (SM)	1	SS	50/100		331	
330.3 / 1.5	**SILT**, trace to some fine grained sand, trace to some clay Loose Brown Moist to wet	2	SS	9		330	
		3	SS	9		329	0 2 88 10
		4	SS	7		328	
327.6 / 4.1	Sandy **SILT**, fine grained, trace clay Compact Brown Moist	5	SS	18		327	0 34 61 4
						326	
		6	SS	21			
325.0 / 6.7	END OF BOREHOLE AT 6.71m.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

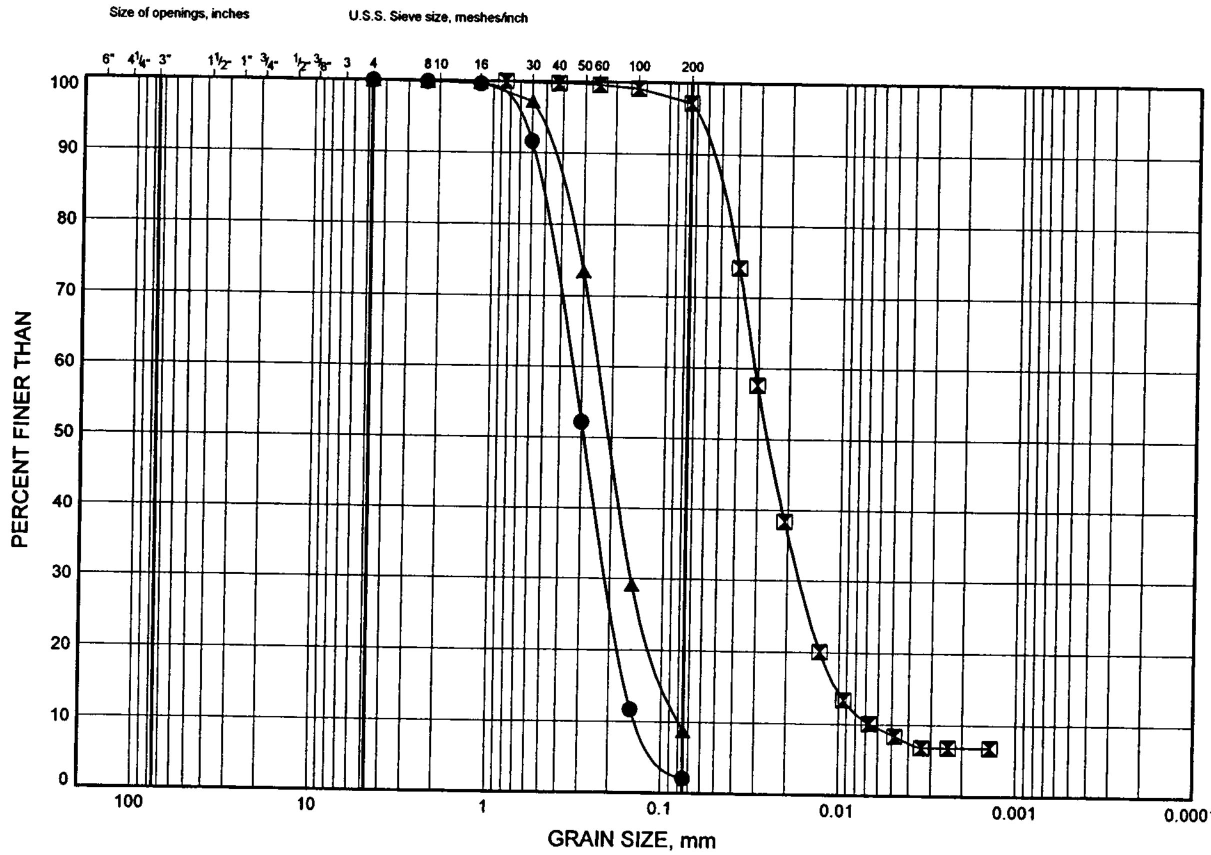
Appendix B

Laboratory Test Results

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B1



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH 422-1	2.59	329.20
⊠	BH 422-2	1.83	328.54
▲	BH 422-2	9.45	320.92

THURBGSD 422STIRLING CK & PEVENSEY.GPJ 30/07/03

Date July 2003

Project 742-93-00

THURBER

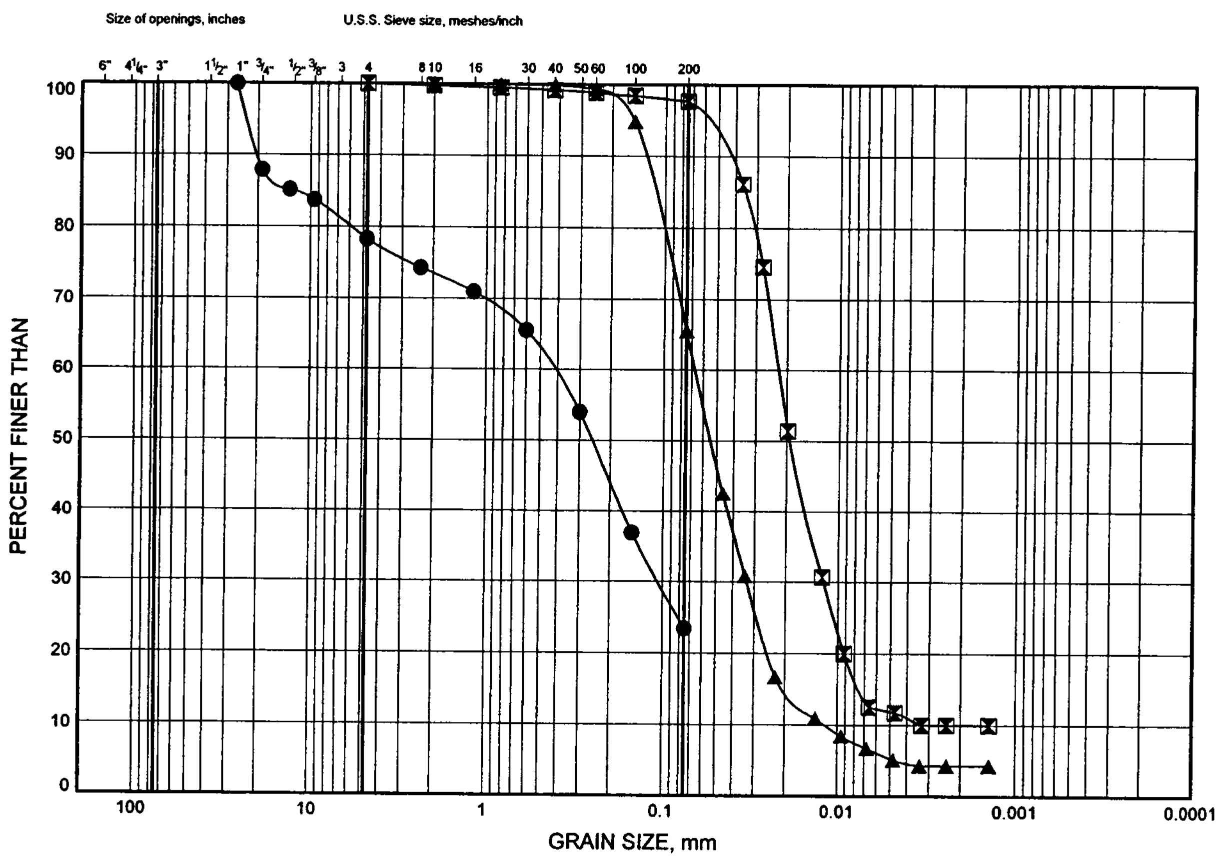
Prep'd EA

Chkd. MF

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH 422-3	15.54	314.58
⊠	BH 422-4	2.59	329.15
▲	BH 422-4	4.88	326.86

Date July 2003

Project 742-93-00

THURBER

Prep'd EA

Chkd. MF

THURBGSD 422STIRLING CK & PEVENSEY.GPJ 30/07/03

Appendix C

Factual Data from Golder's Report

RECORD OF BOREHOLE No 2-1

METRIC

PROJECT 991-1193
W.P. 335-98-00 | LOCATION N 5059288.92; E 310852.36 | ORIGINATED BY SB
DIST 54 HWY 11 | BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY DKB
DATUM GEODETIC | DATE Feb.9/00 | CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
332.42	GROUND SURFACE					
0.00	Sand and Gravel, trace silt Brown Moist (Fill)					
331.66 0.76	Silty Sand Very dense to compact Brown Moist	1	SS	62		
		2	SS	13		
330.21 2.21	Silt, trace clay and sand Compact to loose Brown and grey Moist to wet	3	SS	12		
		4	SS	6		
		5	SS	5		0 2 91 7
		6	SS	4		
		7	SS	6		
325.71 6.71	END OF BOREHOLE					

Note:
1. Open borehole dry upon completion of drilling.

Elevation scale: 332, 331, 330, 329, 328, 327, 326

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 2-2

METRIC

PROJECT	991-1193		
W.P.	335-98-00	LOCATION	N 5059271.30; E 310622.26
DIST	54	HWY	11
BOREHOLE TYPE	108mm I.D. HOLLOW STEM AUGERS		
DATUM	GEODETIC	DATE	Feb.9/00
ORIGINATED BY	SB		
COMPILED BY	DKB		
CHECKED BY	ASP		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
330.64	GROUND SURFACE
0.00	Silt, trace to some sand, trace clay Very loose to loose Brown Moist to wet Note: non-plastic Atterberg Limit test result for Sample 2.
325.00 / 5.64	Silty Sand Compact Brown Moist
323.48 / 7.16	Sand, trace to some silt Compact to loose Brown Moist to wet
315.86 / 14.78	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	6				
2	SS	1	0	7	88	5
3	SS	7				
4	SS	4				
5	SS	10	0	14	81	5
6	SS	7				
7	SS	13				
8	SS	15				
9	SS	19				
10	SS	10	0	77	23	0
11	SS	7				
12	SS	8				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (330 to 316); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation
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RECORD OF BOREHOLE No 2-2

METRIC

PROJECT 991-1193
W.P. 335-96-00
DIST 54 HWY 11
DATUM GEODETIC

LOCATION N 5059271.30; E 310822.26
BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS
DATE Feb.9/00

ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%): GR	SA	SI	CL
	— CONTINUED FROM PREVIOUS PAGE —									
	Gravelly Sand, trace to some silt, occasional cobbles and/or boulders Compact Brown Wet	13	SS	28	315	o				
		14	SS	11	314					
					313					
		15	SS	25	312	o	27	56	17	0
310.83 / 19.81	END OF BOREHOLE				311					
					310					
309.51 / 21.13	END OF CONEHOLE Refusal to further cone penetration; probable bedrock									

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_p, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Note:
1. Water level measured in piezometer at 13.3m depth (EL 317.3m) upon completion of installation.
2. Water frozen in piezometer at 0.3m depth (EL 330.3m) on March 8, 2000.
3. Water level measured in piezometer at 11.6m depth (EL 319.1m) on March 26, 2000.

ON_MOT 991-1193.GPJ ON_MOT.GDT 24/4/00

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 2-3

METRIC

PROJECT 991-1193			
W.P. 335-98-00	LOCATION N 5059244.8; E 310739.67		ORIGINATED BY SB
DIST 54	HWY 11	BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS	COMPILED BY DKB
DATUM GEODETIC	DATE Feb.7/00		CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
332.90	GROUND SURFACE					
0.00	Sand and Gravel, trace silt, Brown, Moist (Fill)					
332.14 / 0.76	Silt, trace to some sand, trace clay, Compact, Brown, Moist	1	SS	25		
		2	SS	12		
330.69 / 2.21	Silty Sand, Compact, Brown, Moist	3	SS	12		
329.93 / 2.97	Sand, trace to some silt, Compact, Brown, Moist	4	SS	12		
		5	SS	13		
		6	SS	13		
		7	SS	13		0 88 12 0
					∇	
		8	SS	16		
		9	SS	15		
322.90 / 10.00	Gravelly Sand, some silt, occ. cobbles and/or boulders, Very dense, Brown, Moist	10	SS	100/0.05		
322.11 / 10.79	Slightly weathered to fresh, grey-white with black streaks, moderately to widely jointed, moderately foliated (30°), medium to coarse grained, strong to very strong GNEISS. Bedrock cored from 10.79m to 13.74m depth. For bedrock coring details refer to Record of Drillhole 2-3					
319.16 / 13.74	END OF HOLE. Note: Water level measured in open borehole at 7.6m depth (El. 325.3m) upon completion of drilling.					

ELEVATION SCALE: 332, 331, 330, 329, 328, 327, 326, 325, 324, 323, 322, 321, 320

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT w_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

PROJECT: 991-1193

LOCATION: N 5059244.8; E 310739.67

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 2-3

DRILLING DATE: Feb.7/00

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

SHEET 1 OF 1

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		322.11												
11–13	NQ RC	Slightly weathered to fresh, grey-white with black streaks, moderately to widely jointed, moderately foliated (30°), medium to coarse grained, strong to very strong GNEISS. Becomes slightly to moderately weathered strong rock at 11.98-12.2m. Becomes grey-white pegmatite at 12.5-12.85m.		10.79	2		100						J,R,P J,R,P J,SM,P J,SM,P J,SM,P,clay coating J,R,P,FeO stain MB J,R,P,clay coating			
				319.16 13.74												
14		END OF HOLE														

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: SB

CHECKED: PD

DRILLHOLE 1193ROCK.GPJ GLDR CAN.GDT 24/4/00 PS

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2-4

METRIC

PROJECT 991-1193 | W.P. 335-98-00 | LOCATION N 5059229.83; E 310702.11 | ORIGINATED BY SB

DIST 54 | HWY 11 | BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY DKB

DATUM GEODETIC | DATE Feb.9/00 | CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR SA SI CL
331.16	GROUND SURFACE						
0.00	Sand and Gravel, trace silt Brown Moist (Fill)				331		
330.55 0.61	Sand, trace to some silt Dense to loose Brown Moist	1	SS	34	330	~19	
		2	SS	6	329		
		3	SS	5		~5	
		4	SS	6	328		
		5	SS	7	327	~1	0 98 2 0
		6	SS	8	326		
325.06 6.10	Gravelly Sand, some silt Compact Brown Moist	7	SS	22	325	~31	
324.45 6.71	END OF BOREHOLE Note: Open borehole dry upon completion of drilling.						

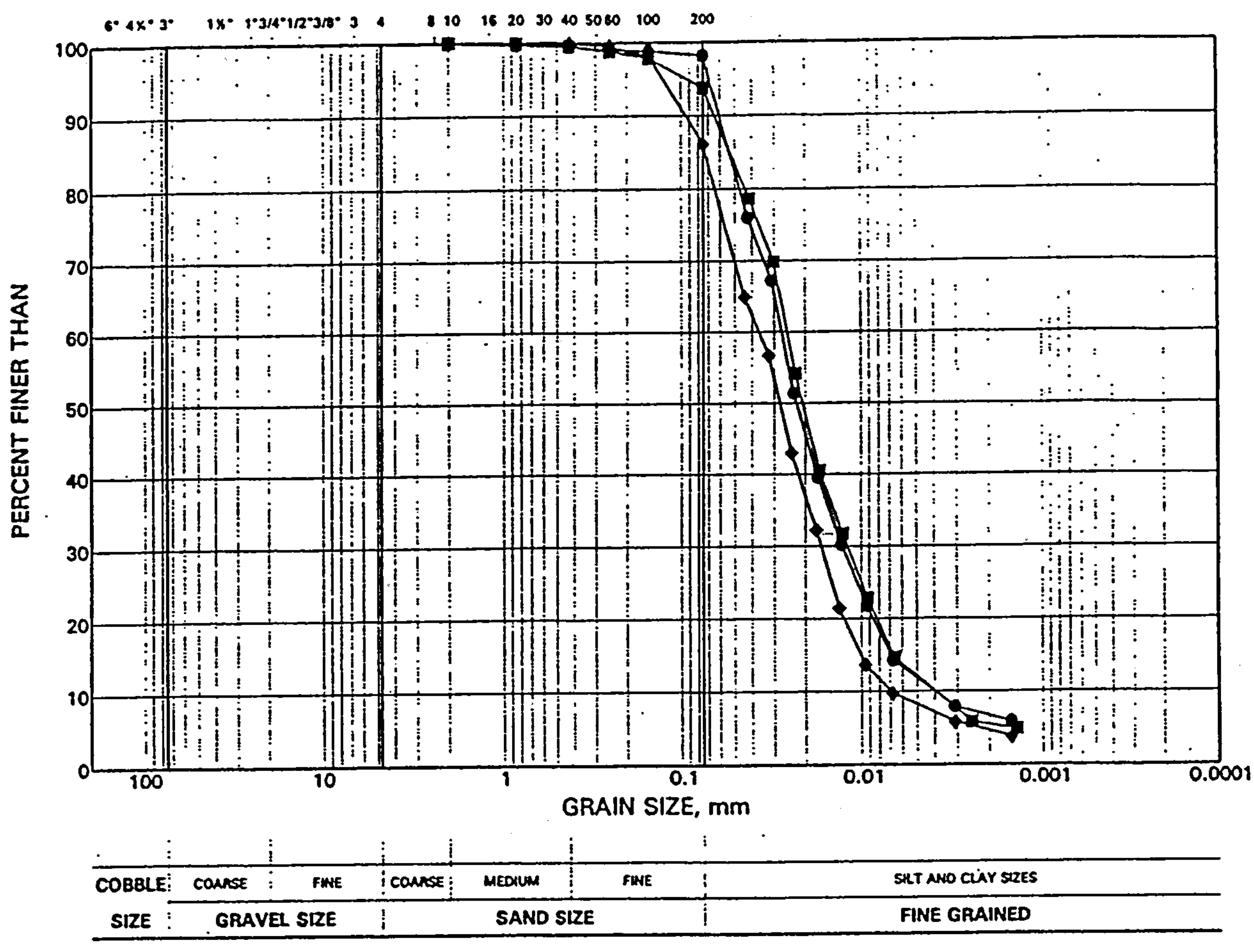
Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, X³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

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

GRAIN SIZE DISTRIBUTION
Silt, trace to some sand, trace clay

FIGURE 1



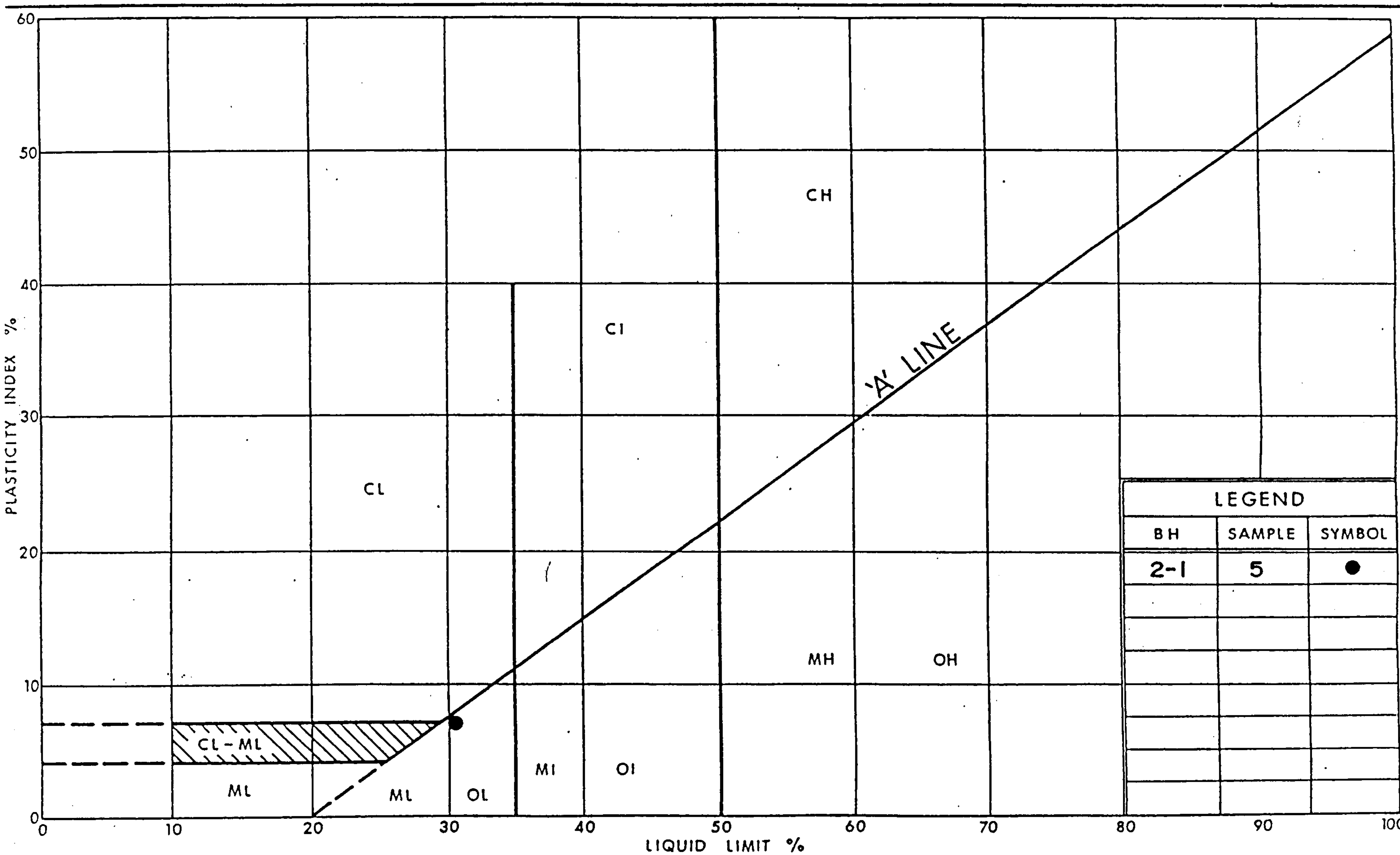
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2-1	5	328.0
■	2-2	2	328.5
◆	2-2	5	326.4

Project 991-1193

Golder Associates

Oct 75, FF-S-21



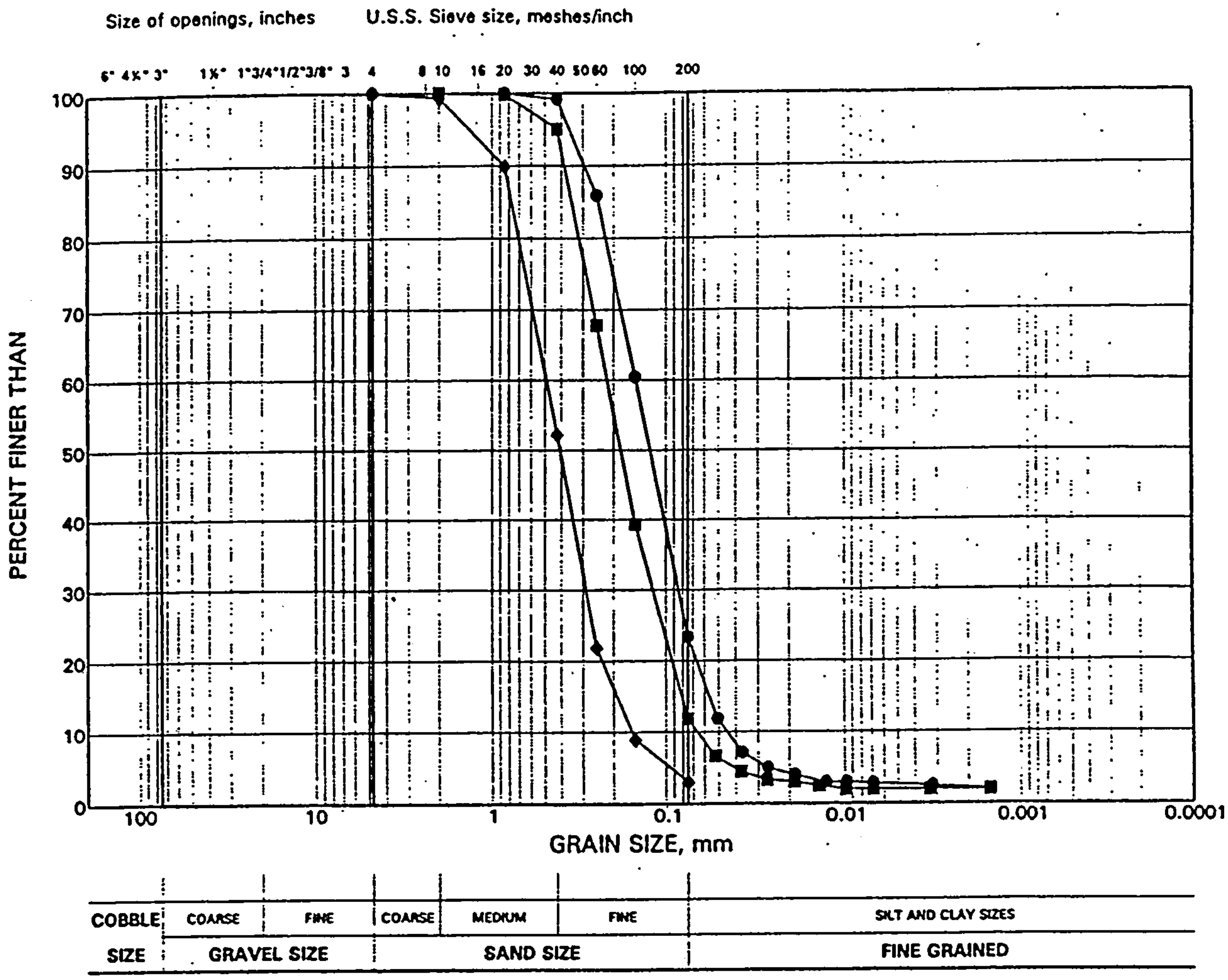
Ministry of Transportation
Ontario

PLASTICITY CHART
SILT

FIG No. 2
W P 335-98-00

GRAIN SIZE DISTRIBUTION
Sand, trace to some silt

FIGURE 3



LEGEND

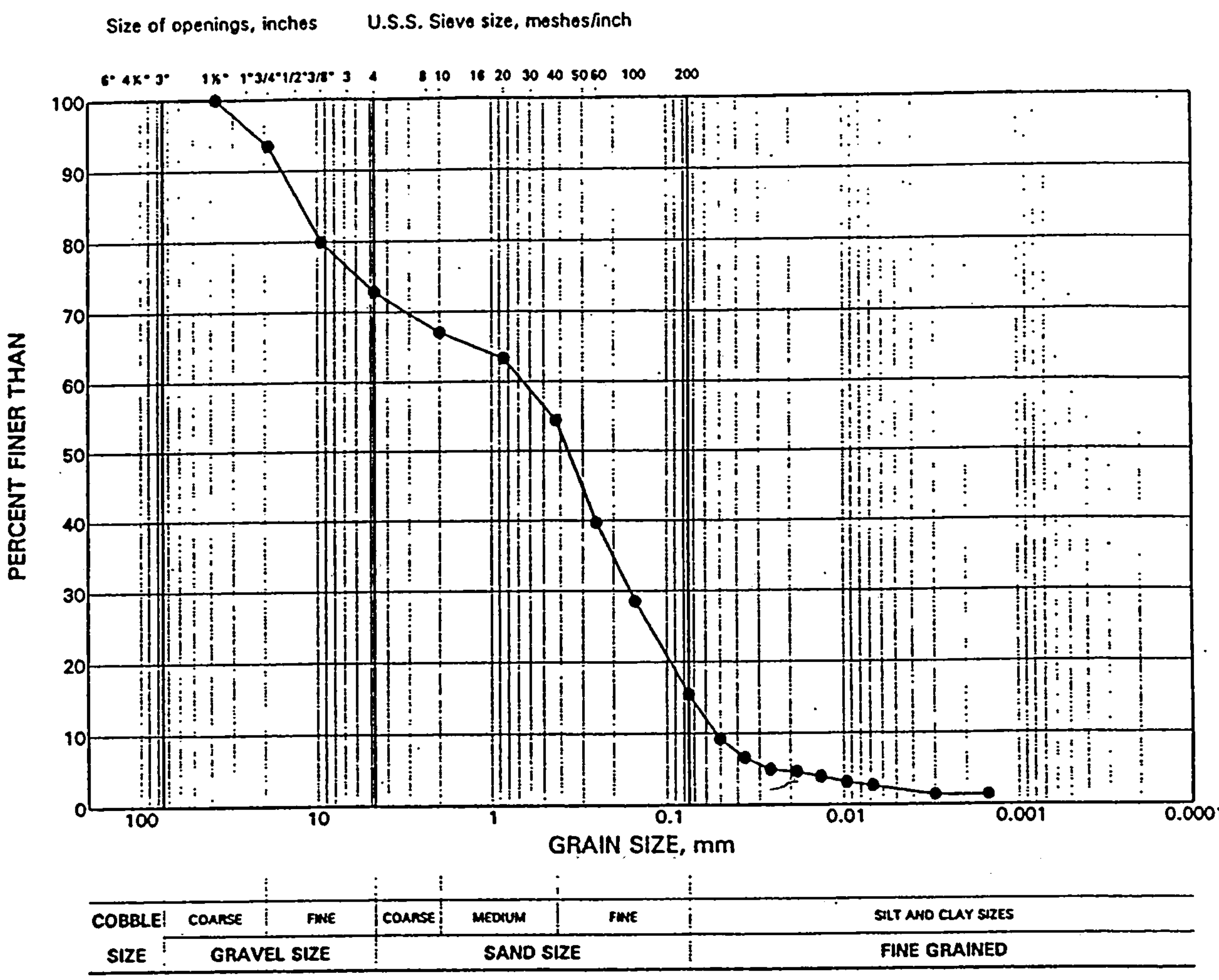
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2-2	10	319.3
■	2-3	7	326.2
◆	2-4	5	326.8

Project 991-1193

Golder Associates

GRAIN SIZE DISTRIBUTION
Gravelly Sand, some silt

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2-2	15	311.7

Project 991-1193

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Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil	Footing on Engineered Fill	Caisson
Advantages: i. Piles will develop high geotechnical resistance if driven to bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. ***Disadvantages:*** i. Higher unit costs than footings. ii. Construction concerns related to the possibility of pile being obstructed by a boulder during driving.	***Advantages:*** i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. ***Disadvantages:*** i. Soil conditions encountered at this site would permit only comparatively low geotechnical resistance. ii. Settlement under a spread footing is a potential concern.	***Advantages:*** 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. ***Disadvantages:*** i. Cost of constructing engineered fill. ii. Comparatively limited geotechnical resistance when compared to deep foundation systems.	***Advantages:*** i. High resistance is available for caissons founded on the bedrock. ii. Construction of caissons could continue in freezing weather. iii. Choice of conventional or semi-integral abutment design. ***Disadvantages:*** i. The subsurface conditions consist of relatively permeable, cohesionless soils with a groundwater level at a depth of approximately 10m. Temporary liners would be required and it would be difficult to obtain a seal into the bedrock to allow concrete placement in the dry, especially with the presence of cobbles. ii. Caissons preclude an integral abutment design.

THURBER

Appendix E

Special Provisions

THURBER

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 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 maxiumum 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.

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

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.

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903.04.02.03 Slurry

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

1. The type, source, physical and chemical properties of the bentonite or polymer.
2. Slurry mix proportions and procedure.
3. Quality Control Plan to control properties of slurry mix.
4. 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:

1. Type of equipment and hammer details including Contractors stated potential energy(rated energy) of the hammer, operating efficiency, weight of ram, anvil and helmet.
2. Procedure including sequence for pile installation.
3. Procedure for monitoring installation

903.04.03.02 Caisson Piles

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

1. Shop drawings that describe and illustrate equipment, materials.
2. Procedure for caisson excavation and construction.
3. 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:

1. Any interpretation of data or opinions expressed in the report are not warranted.
2. 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 Verfication 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 ¼ 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 Caissson 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

1. cut off ± 25 mm
2. 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
3. the deviation from the specified inclination for battered piles shall not exceed 1 in 25
4. 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

1. Cut off elevation ±25 mm
2. Horizontal location at cut-off not more than 5% of shaft diameter nor 75 mm
3. 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 made of the number of piles retapped above and beyond the minimum 10% but no fewer than two piles requirement and the number of piles in the pile group and a ratio will be determined.

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

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item
Supply Equipment for Installing Caisson Piles - Item
Supply Equipment for Installing Displacement Caisson Piles - Item

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

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

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

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

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

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

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

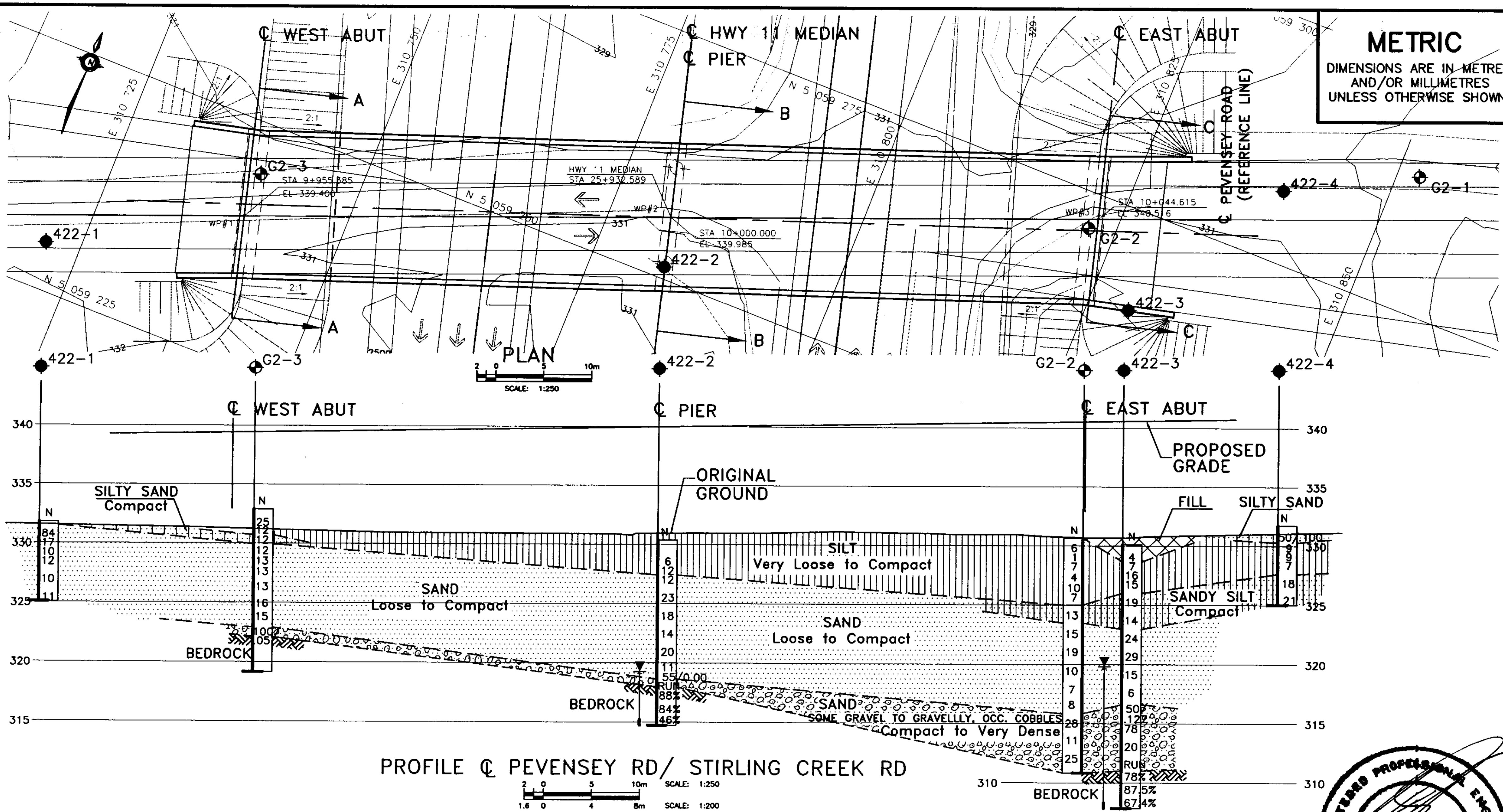
903.10.03 Retapping Piles – Item

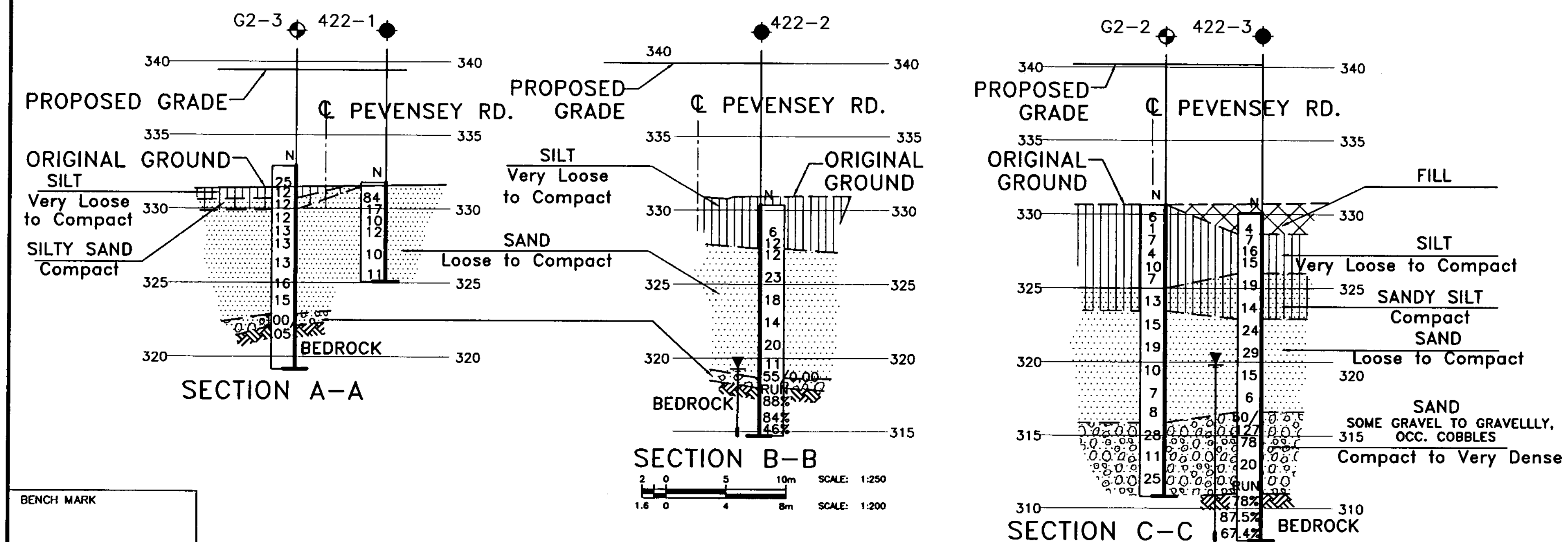
Payment for retapping the minimum specified number of piles is included in the Pile Item. Where additional retapping is required, payment will be made based on the ratio of the number of piles retapped in a pile group above the minimum requirement, to the total number of piles in that pile group, times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

WARRANT: Always with these tender items.

Appendix F

Borehole Locations and Soil Strata Drawing





BENCH MARK

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY 11
CONT No
WP No 743–93–01

PEVENSEY RD/STIRLING CREEK RD I/C U/P
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Marshall Macklin Monaghan
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.

KEYPLAN

LEGEND

●	BoreHole by THURBER
⊕	Dynamic Cone penetration Test (cone)
◆	BoreHole by GOLDER
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 422–1	331.8	5 059 230.0	310 721.4
BH 422–2	330.4	5 059 251.2	310 782.6
BH 422–3	330.1	5 059 264.8	310 829.3
BH 422–4	331.7	5 059 282.3	310 839.6
BH G2–1	332.4	5 059 288.9	310 852.4
BH G2–2	330.6	5 059 271.3	310 822.3
BH G2–3	332.9	5 059 244.8	310 739.7

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
JAN 20/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 20/04
PROVINCE OF ONTARIO

— NOTE —

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

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN AEG	CHK PKC	CODE CHBDC 2000	LOAD CL-625-ONT	DATE JUNE 2003	
DRAWN SS	CHK AEG	SITE 44–422	STRUCT .	SCHEME .	DWG 2