PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
BOWEN ROAD I/C UNDERPASS
QUEEN ELIZABETH WAY, FORT ERIE
ONTARIO
W.P. 2482-04-00

Geocres Number: 30L15-13

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation at the Bowen Road Underpass structure on the existing four-lane Queen Elizabeth Way (Q.E.W.) near Fort Erie, Ontario.

The existing Bowen Road bridge and interchange currently spanning the Q.E.W. are proposed to be replaced. The replacement will include the widening of Bowen Road from two to four lanes over the Q.E.W. and the reconstruction of the interchange ramps.

No preliminary foundation data was available for this site.

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 the data obtained in the course of the present investigation.

Thurber carried out the investigation as a sub-consultant to NCE-Genivar Ltd. (NCE) under the Ministry of Transportation Ontario (MTO) Agreement Number 2006-E-0014.

2 SITE DESCRIPTION

The existing Bowen Road underpass is located approximately 7 km northwest of the town of Fort Erie. Bowen Road will be realigned slightly to the south of the existing alignment.

The existing Bowen Road underpass is a two-lane paved road about 11 m wide with paved shoulders. The underpass crosses the highway on approach embankments about 5 to 6 m high. The existing structure is two spans and has a total length of approximately 45 m.

The general site area is located within the southern portion of the physiographic region known as the Haldimand Clay Plain, characterized by deep water glacio-lacustrine silts and clays. The local topography is a level plain, typical of lake sediments. Though submerged by glacial Lake Warren, the till in the area is commonly interbedded with layers of the glacio-lacustrine deposits and is commonly difficult to differentiate from the lake sediments. Locally, a ridge sloping south, part of the Ononaga Escarpment is located immediately west and north of the site.

The existing bridge structure is located in a rural area currently used for agricultural purposes. Occasional residential dwellings associated with the farm land are located around the site. To the east of the site, the land is used for industrial and commercial purposes.

3 SITE INVESTIGATION AND FIELD TESTING

3.1 General

The site investigation and field testing for this project were carried out between November 17 and November 21, 2008. The site investigation consisted of drilling and sampling a total of five boreholes to depths ranging from 1.2 m to 3.1 m. The borehole numbers and locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

The borehole locations were surveyed using a differential GPS and Thurber obtained all necessary highway occupancy permits and utility clearances prior to any drilling being carried out. Traffic control was provided by Miller Maintenance out of Fort Erie, Ontario.

3.2 Drilling and Sampling

Solid stem auger drilling techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

Each of the boreholes was advanced to the surface of or into the assumed bedrock by augering until grinding refusal was encountered.

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

3.3 Installations and Backfilling

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A 19 mm diameter piezometer was installed in BH08-05 to allow longer term monitoring of groundwater levels. The screened portion of the standpipe piezometer was surrounded with sand prior to backfilling with bentonite holeplug to the ground surface. Additional standpipes were not installed into the overburden at the other foundation elements due to the shallow depth of the soils encountered at the borehole locations. The location and completion details of the boreholes and standpipe piezometer are shown in Table 3.1.

The remaining boreholes were backfilled using a mixture of bentonite holeplug and drill cuttings.

Table 3.1 – Borehole Details

Borehole Number	Piezometer Tip Details		Backfill
	Depth (m) / El. (m)	Stratum	
08-01	None installed	-	Borehole backfilled with drill cuttings and holeplug to surface.
08-02	None installed	-	Borehole backfilled with drill cuttings and holeplug to surface.
08-03	None installed	-	Borehole backfilled with drill cuttings and holeplug to surface.
08-04	None installed	-	Borehole backfilled with drill cuttings and holeplug to surface.
08-05	2.1 / 182.9	Silty Clay Till	Sand filter from 2.2 to 0.9 m, holeplug from 0.9 m to surface.

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.

More than 25% of the recovered samples were subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A for details of the encountered soil stratigraphy. For illustrative purposes a stratigraphic profile is presented on the Borehole Locations and Soil Strata Drawing, Appendix D. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy encountered at the borehole locations typically consists of topsoil of a variable thickness, overlying layers of native cohesive glacio-lacustrine silty clay deposits underlain by silty clay glacial tills deposited in contact with ice. More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

A dark brown topsoil was encountered in all boreholes to depths ranging from 50 to 100 mm.

5.3 Silty Clay

A layer of low plasticity glacio-lacustrine silty clay was encountered underlying the topsoil in all boreholes. The cohesive layer extended to depths ranging from 0.7 to 1.4 m below ground surface (El. 183.3 to 185.0 m).

Grain size analyses conducted on 2 samples retrieved from the silty clay layer are presented on the Record of Borehole sheets and Figure B1 of Appendix B. Atterberg Limits testing carried out on 1 sample is presented on the Record of Borehole sheets and Figure B3 of Appendix B. A summary of the results of laboratory tests carried out on the silty clay deposit are as follows:

Gravel %	2 to 3
Sand %	24 to 27
Silt %	42 to 43
Clay %	27 to 31
Liquid Limit %	32
Plastic Limit %	16

SPT N-values obtained in the silty clay ranged from 9 to 31 blows per 0.3 m of penetration, indicating a stiff to hard consistency.

The moisture content of the silty clay samples ranged from 8 to 23%.

5.4 Silty Clay Till

A layer of low plasticity silty clay till was encountered below the silty clay layer to depths of up to 3.1m below ground surface in all of the boreholes. The till is believed to be part of the Halton Till sheet common to the Niagara region. The thickness of the till layer varied from 0.5 to 0.8 m in the boreholes drilled at the west approach, west abutment and the pier. At the east abutment and east approach, the silty clay till thickness ranged from 1.5 to 2.4 m.

The underside of this glacial till stratum lies immediately above the surface of the assumed bedrock in all of the boreholes as determined by grinding auger and split spoon refusal. The assumed bedrock was encountered at depths ranging from 1.2 to 1.5 m (El. 184.5 to 183.7 m) in the boreholes located at the western approach and abutment respectively. In the borehole drilled at the pier, the assumed bedrock was encountered at a depth of 1.9 m below the ground surface (El. 183.1 m) and from depths of 3.1 to 2.2 m below the ground surface (El. 180.9 and 182.8 m) in the boreholes located at the east abutment and approach respectively.

SPT N-values in the silty clay till deposit varied from 32 to 100 blows/0.3 m penetration. N-values greater than 100 blows/0.05 m of penetration were encountered in samples recovered immediately above the assumed surface of the bedrock. The N-values indicate that the consistency of the cohesive till is hard.

Grain size distribution results for the silty clay till are presented on the Record of Borehole sheets and Figure B2 of Appendix B. Atterberg Limits testing carried out on 3 samples is presented on the Record of Borehole sheets and Figure B3 of Appendix B. A summary of the results of laboratory tests carried out on 3 samples of the cohesive till deposit were as follows:

Gravel %	2 to 6
Sand %	21 to 42
Silt %	38 to 47
Clay %	14 to 29
Liquid Limit %	19 to 29
Plastic Limit %	12 to 15

Moisture contents in the silty clay till deposit varied from 8 to 15%.

Though not encountered in any of the boreholes, glacial till often contains cobbles and boulders and should be anticipated during construction. It is also possible that excavations near the assumed surface of the bedrock may encounter rock shatter within the silty clay till.

5.5 Bedrock

The soils described above were underlain by assumed bedrock. Geologic mapping[1] of the area indicates that the bedrock generally consists of bituminous, dark brown dolostone of the Bertie Formation. The lithology of the Bertie Formation have been documented based on a nearby quarry that has exposed the lower four of the five sub-units identified by alternating carbonate and mixed carbonate-shale units.

The surface of the bedrock was assumed from spilt spoon and auger refusal. The bedrock surface was assumed to occur at the elevations shown in Table 5.1.

Table 5.1 – Ground Surface and Assumed Bedrock Elevations

Borehole No.	Ground Surface Elevation (m)	Assumed Bedrock Elevation (m)
BH 08-01	185.7	184.5
BH 08-02	185.3	183.7
BH 08-03	185.0	183.1
BH 08-04	184.0	180.9
BH 08-05	185.0	182.8

5.6 Water Levels

Following completion of drilling, the groundwater levels were observed in the open boreholes. No groundwater or cave material was observed in any of the open boreholes upon completion.

A single 19 mm diameter standpipe piezometer was installed in the borehole advanced at the east approach. The groundwater level was measured approximately one week following installation to be at 1.1 m below the ground surface (El. 183.9 m).

It should be noted that these piezometric levels are based on short term observations and groundwater levels are subject to seasonal fluctuations and after severe weather events. Seepage from local, cohesionless lenses interbedded within the glacial till may be encountered and should be anticipated.

6 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the as-drilled locations, and provided northing and easting coordinates and ground surface elevations using a differential GPS.

[1] Freenstra, B.H., 1984. Quaternary Geology of the Niagara-Welland Area, Ontario Geological Survey, Map 2496, Quaternary Geology Series, scale 1:50,000

Elite Drilling Limited of Fort Erie, Ontario supplied and operated a truck-mounted CME 55 drill rig for one borehole. Groundwork Drilling Ltd. of Etobicoke, Ontario supplied and operated a BOA-6M Bombardier-mounted drill rig to conduct the drilling, sampling and in-situ testing operations at the remaining borehole locations.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger of Thurber.

Laboratory testing was carried out by Thurber Engineering Ltd. in its MTO-approved Oakville laboratory.

Interpretation of the field data and preparation of the investigation report was completed by Mr. David Elwood, P.Eng and Mr. Alastair E. Gorman, P. Eng. Overall supervision of the field program was performed by Mr. Alastair E. Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

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

The project will consist of the replacement of the existing two-lane, 45 m long, two-span structure with a four-lane, two-span structure of similar length and number of foundation elements.

At the site, the Q.E.W. runs northwest-southeast and Bowen Road runs east-west. The grade of Q.E.W. lies close to the original ground surface and the original Bowen Road grade is also close to the original ground surface and slopes gently from west to east, following the local topography. Bowen Road crosses the Q.E.W. on the existing structure and the associated approach fills. At the west and east abutments, the finished grade will be respectively at approximate El. 193.8 and 193.5 m, resulting in an approach fill approximately 8 m above the exiting Q.E.W. mainline.

The replacement structure will overlap the footprint of the existing structure and construction will require either a closure of Bowen Road or staged replacement of the existing structure if traffic has to be maintained.

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.

8 STRUCTURE FOUNDATIONS

8.1 Foundation Alternatives

Five foundation types have been considered:

- Spread footings on native soil
- Spread footings on bedrock
- Spread footings on engineered fill
- Drilled shafts (also referred to as caissons or bored piles)
- Steel H-Piles

These foundations alternatives are discussed below.

8.2 Spread Footings on Native Soil

From a geotechnical perspective, spread footings bearing on native soil are feasible, though they are not the preferred alternative at the site.

Spread footings may be designed on the basis of the geotechnical resistances and founding elevations given in Table 8.1. The geotechnical resistances apply at or below the stated elevations.

Table 8.1 Spread Footing Design Parameters

Foundation Element	Elevation	SLS (kN)	ULS_f (kN)
East abutment (BH08-04)	183.2 182.3	400 500	600 750
Pier (BH08-03)	183.5	500	750
West abutment (BH08-02)	184.5	400	600

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

The SLS resistances are based on settlements not exceeding 25 mm. Differential settlements between foundation elements are not expected to exceed 20 mm.

Initial calculations of the sliding resistance may be carried out using a value of 0.55 for the ultimate friction factor of concrete poured on native soil.

8.3 Spread Footings on Bedrock

From a geotechnical perspective, spread footings founded on bedrock are considered feasible at the two abutments and the pier, where the bedrock is located approximately 1.5, 1.9 and 3.1 m respectively below the existing ground level. However it is recognized that spread footings may not be appropriate at the abutments.

For preliminary design, the footings may be sized on the basis of a concentric, vertical geotechnical resistance of 3,000 kPa at factored ULS. The SLS condition will not govern on the bedrock. The assumed bedrock elevations are shown below in Table 8.2.

Table 8.2 – Bedrock Elevations

Borehole	Bedrock Elevation
BH 08-01	184.5
BH 08-02	183.7
BH 08-03	183.1
BH 08-04	180.9
BH 08-05	182.8

The above resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clauses 6.7.3 and 6.7.4.

The concrete footings may be constructed directly on the surface of the dolostone bedrock. In cases where the underside of a footing is higher than the bedrock subgrade due to over-excavation or otherwise, mass concrete of the same class as the foundation should be used to raise the subgrade to the design footing level. The top surface of the bedrock should be stripped of all overburden and be cleaned. All shattered and loosened rock fragments must be removed from the footprint of the footing or mass concrete fill.

All footing excavations must be inspected prior to placing concrete to confirm that the base has been adequately cleaned. Hand cleaning may be required to remove loose rock.

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

8.4 Spread Footings on Engineered Fill

The use of spread footings bearing on engineered fill pads is considered to be feasible provided that the engineered fill pad is founded on the undisturbed native soil or bedrock.

A minimum thickness of 2.0 m of engineered fill is required between the underside of the footing and the founding elevation. The engineered fill must be founded at elevations no higher than those given in Table 8.1. Lower elevations may be necessary to accommodate the minimum thickness of engineered fill.

Provided the engineered fill is constructed as described in this section of the report, footings may be designed on the basis of the following vertical, geotechnical resistances:

- 900 kPa at factored ULS
- 350 kPa at SLS

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

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

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 15 mm across the width of the structure.

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

8.5 Drilled Shafts (Caissons or Bored Piles)

The foundations may also be supported on drilled shafts founded in the bedrock. A qualified geotechnical engineer or technician must visually inspect the exposed bedrock in the caisson base to assess the bedrock quality and geotechnical resistance.

Preliminary design of drilled shafts may be based on a concentric, vertical geotechnical resistance of 5,000 kPa at factored ULS. The SLS condition will not govern in the bedrock. The caisson should be penetrate a nominal 300 mm into competent bedrock in the case of vertical loads. For preliminary design purposes, penetration of 1.0 m into the sound bedrock can be assumed to provide fixity.

The caissons must be installed in accordance with 903SP01.

As it is not considered practical to install caissons on a batter, lateral loads must be resisted by socketing the caissons into bedrock.

8.6 Steel H-Piles

The soil conditions encountered at the site are considered to be suitable for the support of steel H-piles. The recommended minimum pile length is 5.0 from below the underside of the abutment stem to the founding elevation. The length of a driven H-pile at this site will be governed by the elevation of the underside of the abutment stem. Based on our understanding of the proposed grades and assuming a 4.5 m high abutment stem, the anticipated lengths of driven piles will be as shown in Table 8.3.

Table 8.3 – Estimated pile Lengths

Foundation	Assumed Bedrock Elevation	Pile Length
East abutment (BH08-04)	180.9	8.1 m
Pier (BH08-03)	183.1	2.0± m
West abutment (BH08-02)	183.7	5.6 m

Based on these values, driven steel H-piles are expected to be feasible at both abutments but not feasible at the pier. At the abutments, it is anticipated that the approach embankment will first be constructed and the piles driven through the fill material to the founding elevation.

In the event that a lower abutment stem elevation is determined to be necessary, it may still be possible to utilize H-piles and develop an integral abutment design by socketing the piles into the bedrock.

Piles driven to refusal in the bedrock at or below the elevations shown in Table 8.3 may be designed on the basis of a factored geotechnical resistance at ULS equal to:

- 1,800 kN for HP 310 X 110
- 2,000 kN for HP 360 X 132

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

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

8.6.1 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.6.2 Integral Abutment Considerations

The soil conditions at this site are considered to be suitable for integral, semi-integral or conventional abutments. It is recognized that the highway geometry is not suitable for integral abutment design.

If an integral abutment design is considered, the piles must possess flexibility in the upper 3 m of the length below the abutment stem. In the very stiff to hard soils encountered at this site, flexibility should be achieved by placing the piles inside double, concentric CSPs as described in Report SO-96-01 Integral Abutment Bridges.

After the pile is driven, the space between the pile and the inner CSP should be filled with sand. An NSSP should be included in the contract documents specifying the gradation of the sand according to Table 8.5.

Table 8.5 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 μm	#30	80%-100%
425 μm	#40	40%-80%
250 μm	#60	5%-25%
150 μm	#100	0%-6%

8.6.3 Lateral Resistance

The lateral resistance of a pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

Non-cohesive

$k_s = n_h \cdot z / D$ (kN/m^3)

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

Cohesive

$k_s = 125 * S_u/D$ (kN/m^3)

$p_{ult} = 9*S_u$ (kPa) at a depth of 3*D (m) reduce to zero at the ground surface

where
z = depth of embedment of pile in metres
D = pile width in metres
n_h = coefficient of horizontal subgrade reaction (Table 8.6)
γ = unit weight (Table 8.6)
S_u = undrained shear strength (q_u / 2)
K_p = passive earth pressure coefficient

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 8.6 – Recommended Soil Parameters

Reference Borehole No	Applicable Elevation	Soil Type	Bulk or Submerged Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	S_u (kPa)	n_h (kN/m^3)
West Abutment BH 08-02	Above 185.0	Gran. Fill	21.2	30	-	4000
	185.0 – 184.5	Silty Clay	8	-	75	-
	184.5 – 183.5	Silty Clay Till	8	-	150	-
East Abutment BH 08-04	Above 184.0	Gran. Fill	21.2	30	-	4000
	184.0 – 183.0	Silty Clay	8	-	75	-
	183.0 – 181.0	Silty Clay Till	8	-	150	-

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

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

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

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

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

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

Pile Spacing Parallel To Direction of Loading	**Horizontal Subgrade Reaction Reduction Factor, R**
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

For conventional abutments, the lateral resistance may be provided by battered piles.

8.6.4 Pile Installation

Pile installation should be in accordance with Special Provision No. SP903S01.

Pile tips should be protected by driving shoes in accordance with OPSD 3000.100 Type I.

8.7 Recommended Foundation

The preliminary GA for the bridge shows an RSS false abutment. From a geotechnical standpoint, the following foundations are recommended:

East Abutment	Drilled shafts to bedrock
Pier	A spread footing founded on the native soil or on bedrock
	A drilled shaft, bearing on bedrock can be used. This will eliminate roadway protection requirements in the QEW median.
West abutment	Drilled shafts to bedrock.

A comparison of foundation alternatives based on advantages and disadvantages of each foundation alternative is included in Table C1, Appendix C.

Additional design recommendations for drilled shafts may be provided in the detailed design report if this foundation type is selected for this site.

8.8 Frost Protection

The depth of earth cover required to provide frost protection for footings at this site is 1.2 m.

It should be noted that rock fill does not provide the insulation value of soil cover. Where rock fill is used as backfill or for the construction of forward slopes consideration should be given to incorporating synthetic insulation.

Rigid, extruded polystyrene (EPS) insulation may be used and it may be assumed that 25 mm of this insulation provide protection equivalent to 600 mm of soil cover.

8.9 Abutment Considerations

Retained soil system (RSS) walls may be used at both abutments provided that the levelling pad for the RSS wall is formed directly on the undisturbed native soil or on a pad of engineered fill. Engineered fill should be designed in the same manner as the engineered fill to support foundations as described elsewhere in this report. The geotechnical resistance of the bedrock or engineered fill is as stated elsewhere in this report.

RSS walls should be specified to be “High Performance” and “High Appearance”.

The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The global stability of an RSS wall founded as described above will be satisfactory.

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

The settlement of a wall founded on engineered fill pad is expected to be small and should occur essentially as the RSS is constructed.

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site are classed as Type 2 soils. All fills must be classed as Type 3 soils. Excavation, unwatering and backfill must also meet the requirements of SP902S01.

In most cases, the excavations are anticipated to be formed in near vertical open cuts to a maximum depth of 1.2 m. For excavations depths greater than 1.2 m, the sidewalls must be sloped at 1V : 1H in accordance with OSHA or provided with adequate shoring as indicated in OPSS 539 and SP109S46.

Temporary groundwater control will likely involve perimeter ditches and pumping from filtered sumps. Surface drainage should be diverted away from the footing excavations at all times.

10 APPROACH EMBANKMENTS

10.1 Stability

The materials comprising the existing approach embankments were not sampled during this investigation. These fills are approximately 5 to 6 m in height and are assumed to consist of mostly stiff clay fill similar to the overburden materials encountered at the site. The embankments are constructed with side slopes of approximately 2H : 1V and appear to be performing satisfactorily.

Any new or widened embankment must be constructed of SSM or granular material with side slopes not steeper than 2H:1V. All topsoil and other deleterious material must be stripped from the site prior to fill being placed. In the case of embankment widening, the face of the existing embankment must be benched in accordance with OPSD 208.010.

Disturbed or regraded earth slopes must be provided with erosion protection in accordance with OPSS 572.

10.2 Settlement

Considering that the thickness and nature of the overburden materials encountered throughout the site, it is expected that the approach embankments will not experience significant long term settlements. This issue should be further addressed during the detailed design.

11 ROADWAY PROTECTION

The preliminary GA provided to Thurber indicates that the footprint of the proposed structure will overlap that of the existing structure. If traffic has to be maintained during construction and staged replacement of the structure is carried out, then it may be necessary to provide roadway protection to support the portion of the roadway remaining in service.

This issue should be addressed during detail design after the GA and construction sequence have been finalized.

12 DEWATERING

On the basis of the preliminary investigation and in view of the low-permeability of the soils encountered on this site, dewatering is not expected to be a significant issue.

However, it should be anticipated that the median area will carry runoff from the highway and steps must be taken during construction of intercept surface runoff and near-surface seepage water.

All foundation excavations must be unwatered prior to placing concrete as required by SP902SO1.

13 CONSTRUCTION CONCERNS

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

The construction concerns pertinent to this site will become apparent as the design of the structure and its foundations is developed. There are, however, some potential issues that can be identified at this stage that it should be possible to mitigate through design and operational constraints developed at the detail design stage. These include:

- The surface of the bedrock may vary significantly between the borehole locations. More extensive investigation during detail design will provide better definition of the bedrock surface.
- While bedrock has not been proved, dolostone bedrock with a comparatively high strength is anticipated. Machinery equipped with rock breakers and rippers designed for high strength bedrock should be employed if excavation into the bedrock is required.
- Socketting of piers or caissons into bedrock is expected to require coring or drilling equipment, as opposed to auger equipment. This must be further addressed during detail design.
- Possible groundwater infiltration into open excavations from discontinuities in the bedrock

14 INVESTIGATION DURING DETAIL DESIGN

The requirements for foundation investigation during detail design must be determined after the location and GA of the bridge have been finalized. At that time, the existing pattern of boreholes should be superimposed on the GAs in order to determine the extent of additional investigation that may be required.

Typically, it is recommended that there be a minimum of two sampled boreholes at each foundation element for deep foundation design and a minimum of two for shallow foundation design. Boreholes will also be required at the approaches to the structure.

Typically, it is recommended that there be a minimum of two sampled boreholes at each foundation element for deep foundation design and a minimum of two for shallow foundation design. Boreholes will also be required at the approaches to the structure.

In particular, the investigation, analysis and recommendations produced during detail design must address the quality and strength of the bedrock and variability of the bedrock topography.

15 CLOSURE

Engineering analysis and preparation of this preliminary foundation design report was carried out by Mr. David Elwood, P.Eng. and Mr. Alastair E. Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

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
- Bedrock (general)

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.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength** (MPa)	(psi)	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

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RECORD OF BOREHOLE No 08-01

METRIC

G.W.P. 2482-04-00 | LOCATION N 4 754 977.7 E 346 915.2 | ORIGINATED BY SLL

HWY Q.E.W. | BOREHOLE TYPE Solid Stem Auger | COMPILED BY AN

DATUM Geodetic | DATE 2008.11.21 - 2008.11.21 | CHECKED BY DEE

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
185.7	Geodetic											
0.0	**TOPSOIL** (50mm)											
0.0	Silty **CLAY**, trace roots, trace rootlets Stiff Brown (CL)	1	SS	10					3	27	42	27
185.0 0.7	Silty **CLAY**, trace gravel, trace shale fragments Very Hard Reddish Brown (TILL) (CL)	2	SS	57/ 0.200		185						
184.5 1.2	END OF BOREHOLE (SAMPLER BOUNCING) AT 1.2m ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG BENTONITE, MIXED WITH CUTTINGS TO SURFACE.											

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

RECORD OF BOREHOLE No 08-02

METRIC

G.W.P. 2482-04-00 LOCATION N 4 754 978.5 E 346 942.7 ORIGINATED BY SLL
HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
DATUM Geodetic DATE 2008.11.17 - 2008.11.17 CHECKED BY DEE

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	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
185.3	Geodetic										
0.0	**TOPSOIL** (100mm)										
0.1	Silty **CLAY**, trace roots and rootlets Stiff Brown (CL)		1	SS	9		185				
184.6 0.7	Silty **CLAY**, trace gravel Hard Brown (TILL) (CL)		2	SS	32		184				2 21 47 29
183.7 1.5	END OF BOREHOLE AT 1.5m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS AND HOLEPLUG TO SURFACE.										

+ 3, × 3: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 08-03

METRIC

G.W.P. 2482-04-00 LOCATION N 4 754 988.0 E 346 967.3 ORIGINATED BY SLL

HWY Q.E.W. BOREHOLE TYPE Soild Stem Auger COMPILED BY AN

DATUM Geodetic DATE 2008.11.21 - 2008.11.21 CHECKED BY DEE

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	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, × 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
185.0	Geodetic					185				
0.0	**TOPSOIL** (75mm)									
0.1	Silty **CLAY**, trace gravel, trace roots and rootlets Very Stiff Brown (CL)	1	SS	17				o		
		2	SS	31		184		o		
183.5 1.4	Silty **CLAY**, some sand, trace gravel Very Hard Brown (TILL) (CL)	3	SS	100/ 0.225				o⊢⊣		6 42 38 14
183.1 1.9	END OF BOREHOLE AT 1.9m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, MIXED WITH CUTTINGS TO SURFACE.									

+ ³, × ³: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 08-04

METRIC

G.W.P. 2482-04-00 LOCATION N 4 754 976.7 E 347 005.6 ORIGINATED BY SLL

HWY Q.E.W. BOREHOLE TYPE Soild Stem Auger COMPILED BY AN

DATUM Geodetic DATE 2008.11.21 - 2008.11.21 CHECKED BY DEE

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	Water content (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.0	Geodetic						
0.0	**TOPSOIL** (75mm)						
0.1	Silty **CLAY**, trace gravel, trace roots and rootlets Stiff Brown (CL)	1	SS	11			
183.3 0.7	Silty **CLAY**, trace gravel, shale fragments Hard Reddish Brown (TILL) (CL)	2	SS	43			
		3	SS	86			4 26 42 28
		4	SS	100			
180.9		5	SS	100/ 0.050			
3.1	END OF BOREHOLE AT 3.1m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN TO 2.9m AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, MIXED WITH AUGER CUTTINGS TO SURFACE.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (184, 183, 182, 181); 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

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

ONTMT4S 7455.GPJ 11/3/09

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RECORD OF BOREHOLE No 08-05

METRIC

G.W.P. 2482-04-00	LOCATION N 4 754 975.3 E 347 022.8	ORIGINATED BY SLL	
HWY Q.E.W.	BOREHOLE TYPE Soild Stem Auger	COMPILED BY AN	
DATUM Geodetic	DATE 2008.11.21 - 2008.11.21	CHECKED BY DEE	

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.0	Geodetic						
0.0	**TOPSOIL** (50mm)						
0.0	Silty **CLAY**, trace gravel, shale fragments Very Stiff Brown (CL)	1	SS	24	185		2 24 43 31
184.3 / 0.7	Silty **CLAY**, trace gravel, shale fragments Very Hard Reddish Brown (TILL) (CL)	2	SS	50	184		
		3	SS	80	183		
182.8 / 2.2	END OF BOREHOLE AT 2.2m UPON AUGER REFUSAL. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						

Column headings: SOIL PROFILE; 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
11/28/08	1.1	183.9

ONTMT4S 7455.GPJ 11/3/09

+ 3, × 3: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Appendix B

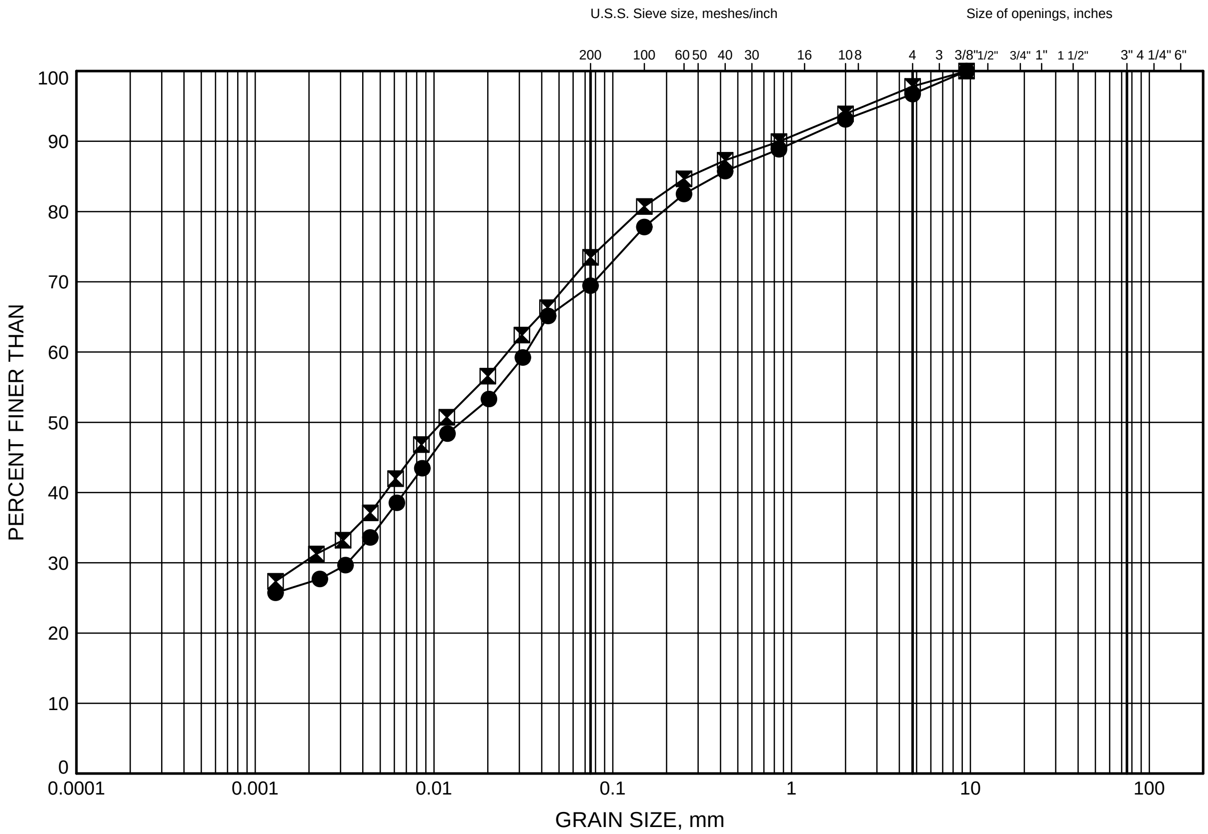
Laboratory Test Results

Bowen Road Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-01	0.30	185.40
⧗	08-05	0.30	184.72

GRAIN SIZE DISTRIBUTION - THURBER 7455.GPJ 11/3/09

W.P.# 2482-04-00

Prepared By MFA

Checked By DEE

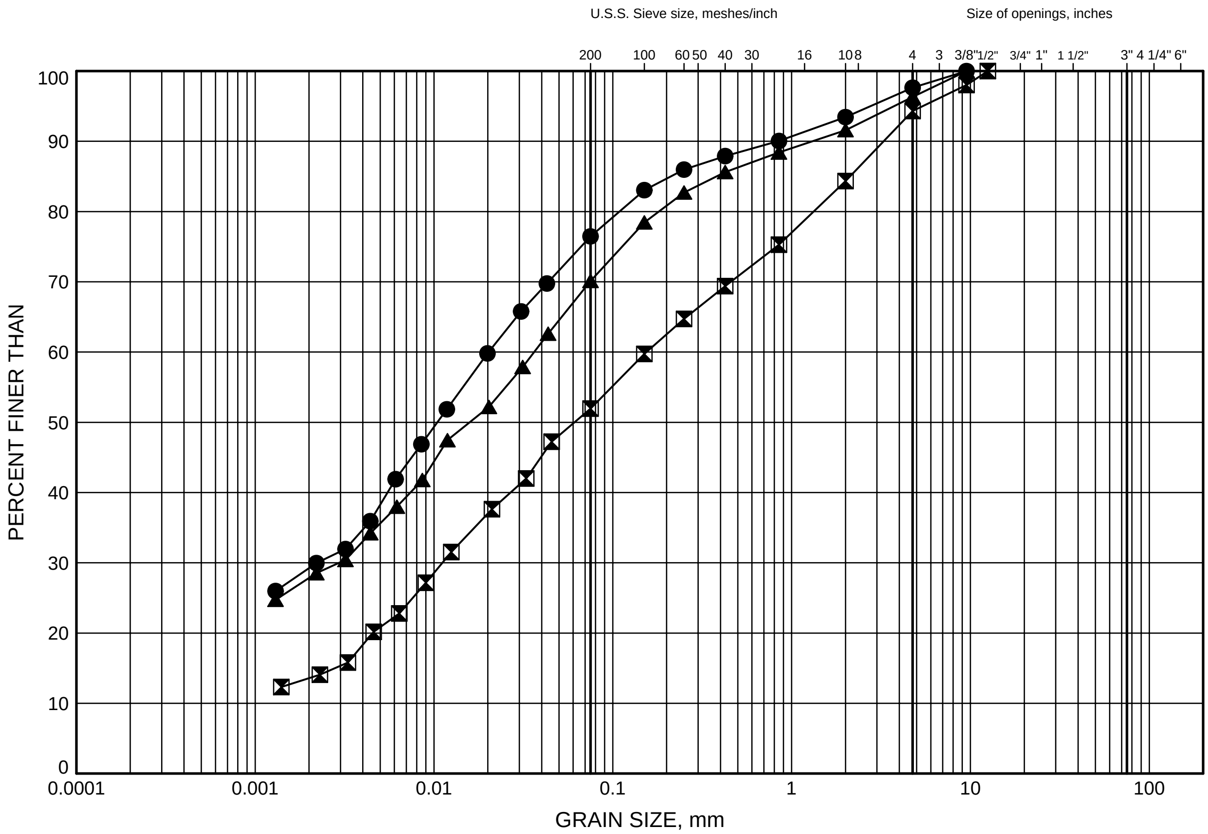
THURBER

Bowen Road Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-02	1.07	184.21
⊠	08-03	1.71	183.25
▲	08-04	1.82	182.14

W.P.# 2482-04-00

Prepared By MFA

Checked By DEE

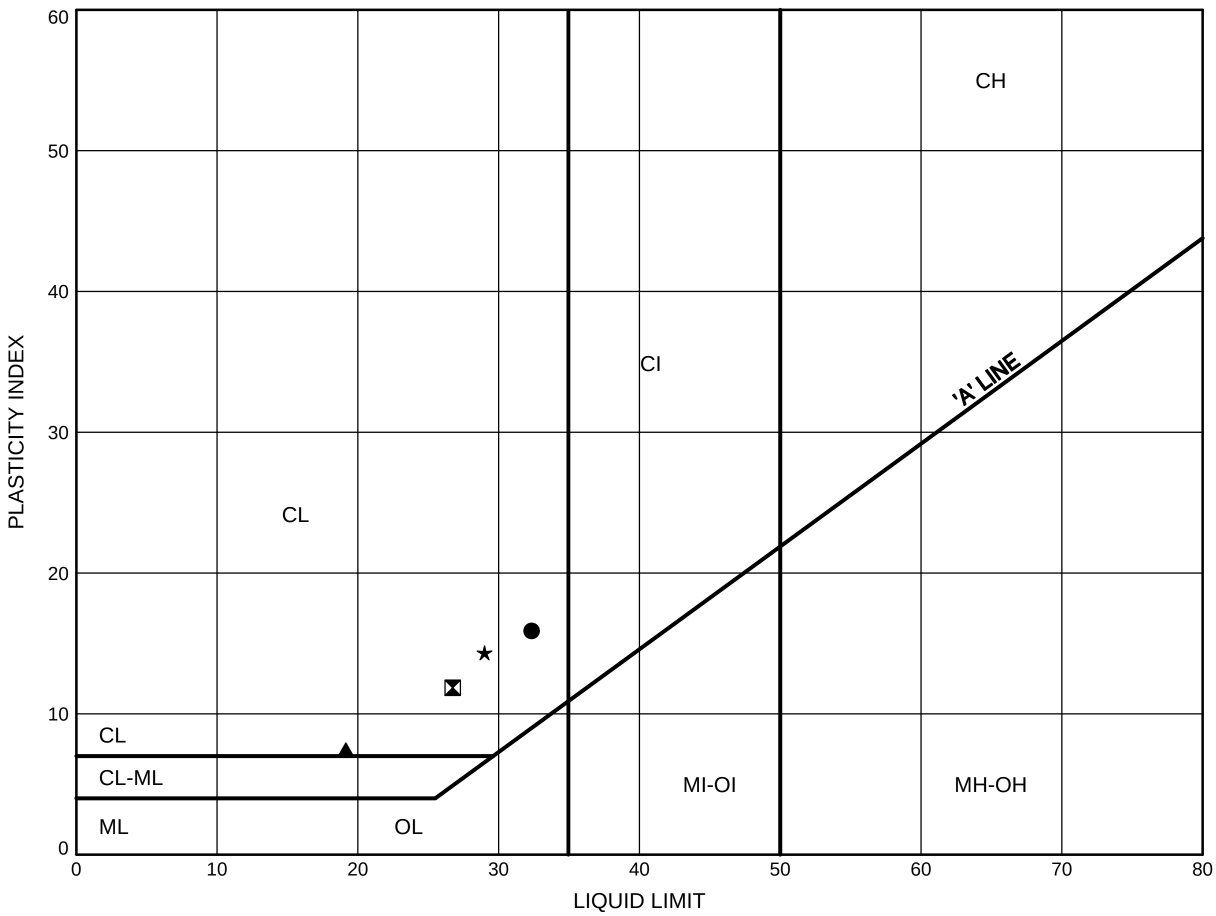
THURBER

Bowen Road Underpass

ATTERBERG LIMITS TEST RESULTS

FIGURE B3

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-01	0.30	185.40
⊠	08-02	0.99	184.28
▲	08-03	1.71	183.25
★	08-04	1.82	182.14

Date March 2009

Project 2482-04-00

THURBER

Prep'd MFA

Chkd. DEE

Appendix C

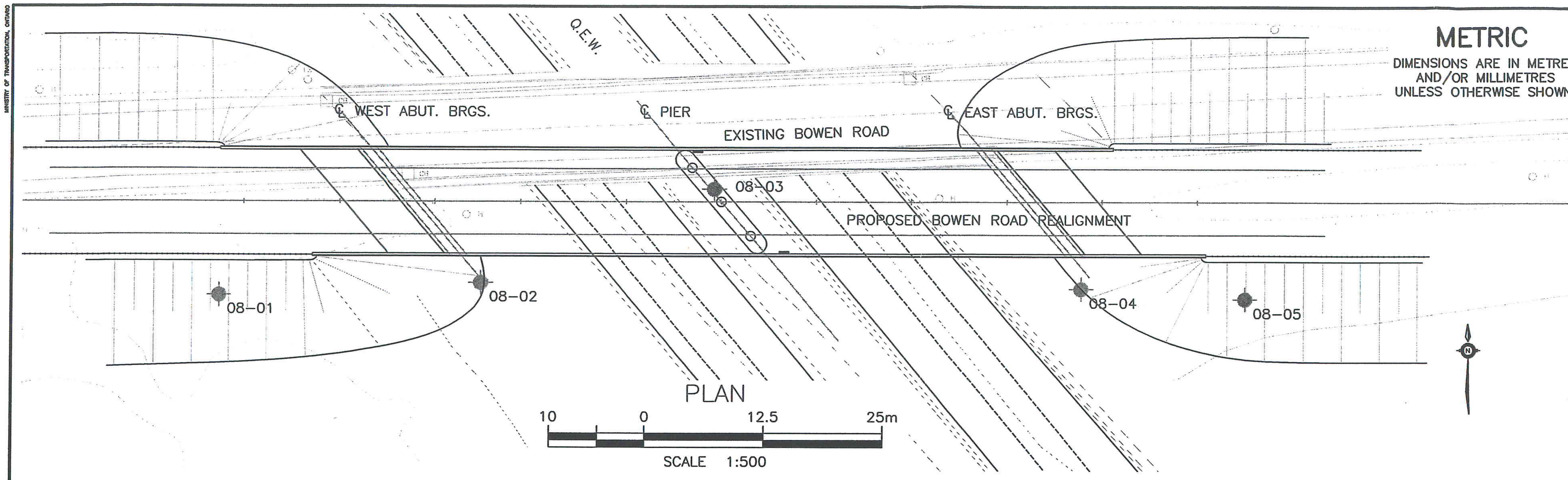
Foundation Comparison

TABLE C1 - COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

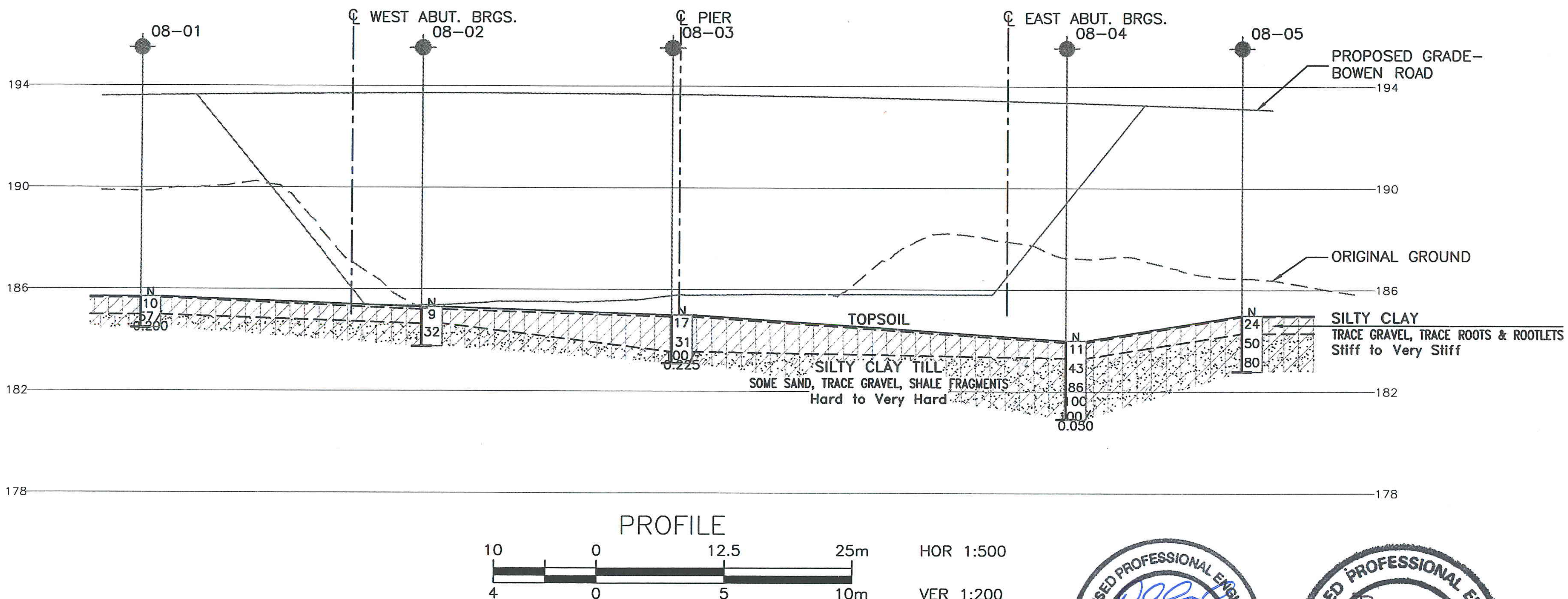
Foundation Element	Driven Piles	Spread Footing	Caissons
West Abutment	***Advantages:*** • High capacity for piles seating on bedrock • Relatively straightforward installation ***Disadvantages:*** • Higher cost than spread footings	***Advantages:*** • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on bedrock • Relatively straightforward installation • Least costly ***Disadvantages:*** • Minimal frost protection	***Advantages:*** • High bearing capacity on bedrock • Reduces requirements for roadway protection ***Disadvantages:*** • Higher cost than spread footings • Installation through bedrock; difficult installation if socketted into bedrock
Pier	***Advantages:*** • High capacity for piles seating on bedrock • Relatively straightforward installation ***Disadvantages:*** • Depending on the foundation elevation, piles may be impractical • Higher cost than spread footings	***Advantages:*** • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on bedrock • Relatively straightforward installation • Least costly ***Disadvantages:*** • Minimal frost protection	***Advantages:*** • High bearing capacity on bedrock • Reduces requirements for roadway protection ***Disadvantages:*** • Higher cost than spread footings • Installation through bedrock; difficult installation if socketted into bedrock
East Abutment	***Advantages:*** • High capacity for piles seating on bedrock • Relatively straightforward installation ***Disadvantages:*** • Depending on abutment configuration, piles may have to be socketed into bedrock • Higher cost than spread footings	***Advantages:*** • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on bedrock • Relatively straightforward installation • Least costly ***Disadvantages:*** • None identified	***Advantages:*** • High bearing capacity on bedrock • Reduces requirements for roadway protection ***Disadvantages:*** • Higher cost than spread footings • Installation through bedrock; difficult installation if socketted into bedrock

Appendix D

Borehole Locations and Soil Strata



PLAN



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Mar 11/09
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METRIC

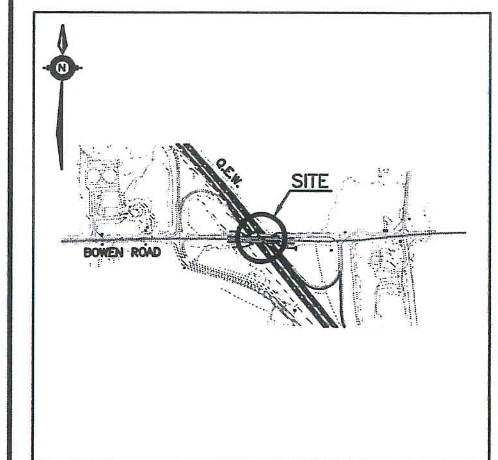
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 2482-04-00

SHEET

Q.E.W.
BOWEN ROAD UNDERPASS
REALIGNMENT
BOREHOLE LOCATIONS AND SOIL STRATA

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
08-01	185.7	4 754 977.7	346 915.2
08-02	185.3	4 754 978.5	346 942.7
08-03	185.0	4 754 988.0	346 967.3
08-04	184.0	4 754 976.7	347 005.6
08-05	185.0	4 754 975.3	347 022.8

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 30L15-13

REVISIONS

DATE	BY	DESCRIPTION			
DESIGN DEE	CHK PKC	CODE	LOAD	DATE FEB. 2009	
DRAWN MFA	CHK AEG	SITE	STRUCT	DWG 1	

FILENAME: H:\Drafting\19\3745\5\ted7455-Plan&Profile.dwg
PLOTDATE: Mar 11, 2009 - 9:10am

MINISTRY OF TRANSPORTATION, ONTARIO