



**THURBER** ENGINEERING LTD.



**FOUNDATION DESIGN REPORT  
FARM ACCESS ROAD BRIDGE (SITE 14A)  
HIGHWAY 401 – COUNTY ROAD 30 INTERCHANGE  
BRIGHTON TOWNSHIP, ONTARIO**

**Geocres Number: 30M13-220**

**Report to**

**AECOM**

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

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County Road 30\Farm Road Access Bridge\Final FDR\Farm Access Road Bridge Final FDR.doc

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## **1 INTRODUCTION**

This report provides foundation design recommendations for the proposed farm access road bridge to be constructed approximately 75 m south of the existing access road at the Highway 401 – County Road 30 Interchange in the Township of Brighton, Ontario.

The existing access road will be realigned to the south due to a southward shift of the W-N/S ramp. The new access road will be carried over the Proctor's Creek on the proposed new access road bridge.

This report was prepared based on interpretation of the geotechnical data in a report prepared by Golder Associates Ltd. (Golder) titled "Foundation Investigation and Design Report, Culverts, Highway 401-County Road 30 Interchange Ramp and Grade Improvements, G.W.P. 256-98-00, Geocres No.: 31C-180", dated November 2007.

Reference should be made to the above report for a written description of the subsurface conditions, borehole location plan, stratigraphic profile, record of borehole sheets and laboratory test results. It should be noted that Golder is solely responsible for the subsurface information provided in their Foundation Investigation and Design Report. The Record of Borehole sheets from the report have been enclosed in Appendix A of this report for reference.

The discussions and recommendations presented in this report are based on information provided by AECOM and on preliminary General Arrangement drawing dated March 2017. The recommendations are intended to provide the designer with sufficient information to assess feasible foundation alternatives and to carry out the design of the foundations for the proposed structure.

This foundation design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction contractor. The contractor must make their own interpretation based on the factual data provided in the report. Where comments are made on construction, they are provided only in order to highlight those aspects, which could affect the design of the project. Contractors must make

their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

## **2 SUBSURFACE CONDITIONS**

The foundation investigation carried out by Golder at the proposed bridge site consisted of two boreholes numbered 07-4 and 07-10. Both boreholes were advanced to a depth of 12.6 m below the existing ground surface and terminated within the hard clayey silt till. The locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix A.

Based on the preliminary GA drawing, both boreholes 07-4 and 07-10 were advanced outside the footprints of the proposed abutment foundations. It is Thurber's understanding that MTO and AECOM have both agreed that the geotechnical recommendations for the proposed bridge shall be provided based on these two boreholes.

In general, the soil conditions at the bridge site consist of a thin layer of very loose surficial silty sand underlain by a typically compact sandy silt to silt deposit grading into silty sand (till) and sand & gravel at depths. A hard clayey silt to silty clay till was encountered below the cohesionless deposits. Both boreholes were terminated within the hard cohesive till.

A thin layer of brown to black silty sand deposit was encountered at the ground surface. The silty sand contained organics and rootlets. Thickness of the layer was 0.8 m in both boreholes with the lower boundary at Elev. 186.8 and 187.2 in 07-4 and 07-10, respectively.

Grey sandy silt to silt was encountered below the surficial silty sand in both boreholes. The layer thickness ranged between 3.0 and 5.0 m with the lower boundary at Elev. 183.8 and 182.2 in 07-4 and 07-10, respectively. SPT-N values recorded in the layer ranged from 6 to 15, indicating a loose to compact relative density. Golder has indicated in the borehole logs that the lower bound SPT-N values were the results of sample disturbance due to groundwater inflow to the borehole. Natural moisture contents of the deposit ranged from 18 to 26%. Two grain size distribution tests indicated that the sandy silt to silt contained 0% gravel, 5 to 9% sand, 84 to 86% silt and 7 to 9% clay.

A layer of grey silty sand (till) was encountered below silt in both boreholes. The layer thickness ranged between 3.3 and 3.5 m with the lower boundary at Elev. 180.3 and 178.9 in 07-4 and 07-10, respectively. SPT-N values recorded in the layer ranged from 3 to 26, indicating a very loose to compact relative density. Golder has indicated in the borehole logs that the lower bound SPT-N values were the results of sample disturbance due to groundwater inflow to the borehole. Natural moisture contents of the deposit ranged from 10 to 12%. A grain size distribution test indicated that the silty sand contained 11% gravel, 49% sand, 32% silt and 8% clay.

A layer of grey sand and gravel was encountered below silty sand (till) in 07-4 only. The layer thickness was 3.1 m with the lower boundary at Elev. 177.2. SPT-N values recorded in the layer ranged from 33 to

70 for 150 mm penetration, indicating a dense to very dense relative density. Natural moisture content was about 8%. A grain size distribution test indicated that the sand and gravel contained 37% gravel, 48% sand, 10% silt and 5% clay.

Grey clayey silt till was encountered below the sand and gravel layer in 07-4 and below silty sand in 07-10. The boreholes were terminated in this deposit at Elev. 175.0 and 175.4 in 07-4 and 07-10, respectively. SPT-N values recorded in the layer ranged from 89 to 107, indicating a hard consistency. Natural moisture contents of the till were approximately 13 to 14%. A grain size distribution test indicated that the cohesive till contained 0% gravel, 15% sand, 48% silt and 37% clay. Two Atterberg Limits tests indicated that the till has plastic limits of 15 to 16% and liquid limits of 29 to 31%, yielding plasticity indices of 14 to 15%.

The groundwater table was observed at approximately Elev. 186 m, or 1.5 to 2.1 m below the ground surface in open boreholes upon completion of drilling. The general arrangement (GA) drawing indicates high water level at Elevation 187.6 m with a 2-year return period.

### **3 STRUCTURE FOUNDATIONS**

Based on the preliminary GA drawing, the proposed farm access steel modular bridge will be constructed over Proctor's Creek on a new alignment approximately 75 m south of the existing access road.

Given the soil conditions encountered and the requirements of modular bridge design, the following foundation options were considered for the support of this bridge:

- Spread footings placed on native soil or engineered fill;
- Driven steel H-piles.

A table of comparison of different foundation alternatives is included in Appendix D. Recommendations for design of the feasible foundation options are presented in the following sections along with the corresponding geotechnical design parameters, where applicable. A preferred foundation option from a geotechnical perspective is indicated.

#### **3.1 Spread Footings**

Based on the subsurface conditions encountered at this site, spread footings founded on native soil or engineered fill are considered suitable for support of the bridge. Prior to subgrade preparation for footing construction, the very loose surficial silty sand containing rootlets and organics should be removed and replaced with granular 'A' material compacted as per OPSS.PROV 501 within the embankment footprints. The excavations will terminate approximately 1 m above the groundwater level observed in the open boreholes and approximately 0.8 m below the 2-year return high water level. Dewatering within the excavation using sump-and-pump method and surface runoff diversion is considered adequate. However,

if the sub-excavation for fill pad construction will be carried out during high water level season, a dewatering system consisting of sandbag cofferdam and active well points may be required to keep the groundwater level below the base of excavation.

### **3.1.1 Spread Footings on Engineered Fill Pads**

A modular bridge supported on concrete spread footings placed on minimum 1 m thick engineered fill pad can be considered at this site. The base of an engineered granular fill pad may be assumed at approximate Elev. 186.8 m for design. At that elevation, the engineered fill pad will be constructed on the loose to compact sandy silt to silt above the groundwater level observed in open boreholes.

Surface vegetation, peat, topsoil, organic deposits, disturbed material or loose/soft soils must be removed within the embankment footprints. Prior to constructing the engineered fill pad, inspection and approval of the subgrade surfaces by qualified geotechnical personnel must be conducted. A separation layer of non-woven geotextile is recommended to be placed on the silt subgrade prior to fill pad construction.

A sketch of the abutment footing placed on engineered fill pad is included in Appendix C. Prior to subgrade preparation for footing construction, the very loose surficial silty sand should be removed and replaced with granular 'A' material compacted as per OPSS.PROV 501.

The engineered fill pad should consist of OPSS Granular 'A' placed in 150 mm lifts and compacted to 100% of its SPMDD at  $\pm 2\%$  of optimum moisture content. The fill pad should extend laterally at least 1.0 m beyond the edge of the footing on all sides and slope down at 1H:1V inclination. The front edge of the footings must be located a minimum 2 m horizontally away from the surface of the proposed 2H:1V forward slope.

### **3.1.2 Spread Footings on Native Soil**

A minimum 300 mm thick bedding layer consisting of OPSS Granular A or B Type II material should be provided at the base of the footings founded on the native soils at Elev. 186.8 m. The sandy silt to silt subgrade is known to be highly susceptible to disturbances and must be protected from construction traffic and weather elements immediately following exposure. A separation layer of non-woven geotextile is recommended to be placed on the silt subgrade prior to footing construction.

Surface vegetation, peat, topsoil, organic deposits, disturbed material or loose/soft soils must be removed within the embankment footprints and replaced with granular 'A' material compacted as per OPSS.PROV 501. Prior to placing the bedding layer, inspection and approval of the subgrade surfaces by qualified geotechnical personnel must be conducted.

### 3.1.3 Axial Geotechnical Resistance and Reaction

Assuming a 2 m wide footing, the geotechnical resistances have been estimated for native soil and for engineered fill pad at least 1 m thick below the base of footing. The geotechnical resistance and reaction have been calculated taking into consideration of the 2H:1V forward slope in front of the footings as shown on the preliminary GA drawing. The recommended geotechnical resistances at the factored ULS and the SLS reaction are provided in Table 3.1. The geotechnical resistances at the SLS reaction correspond to a footing settlement not exceeding 25 mm.

**Table 3.1 – Recommended Geotechnical Resistance and Reaction**

Founding Option	Abutment (Borehole)	Approx. Founding Elevation (m)	Factored ULS (kPa)	SLS Reaction (kPa)
Native Soil	West (07-4)	186.8	200	120
	East (07-10)			
Engineered Fill	West (07-4)	188.0	180	120
	East (07-10)			

Ground settlement up to 30 mm is anticipated in addition to the 25 mm footing settlement in response to the embankment fill placement. It is recommended that the embankment fill placement to the final road grade be carried out to preload the foundation for at least two weeks prior to footing construction. The engineered fill pad may be constructed to the footing base level before placing preload embankment fill. Following completion of the preloading, the preload fill should be removed back to footing base level prior to completing the footing and fill pad construction.

The value of the Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (Typical), and a Resistance Factor of 0.5 (Typical), as per CHBDC 2014. The Geotechnical Reaction at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The geotechnical resistance provided above is for concentric, vertical loading conditions only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as indicated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

The lateral resistance developed along the base of the footings founded on the native soil and engineered fill should be computed using an ultimate friction coefficient of 0.35 and 0.55, respectively.

## 3.2 Driven H-Pile Foundations

The use of driven steel H-pile foundations to support the bridge abutments is considered feasible at this site.

### 3.2.1 Axial Geotechnical Resistance and Reaction

The axial factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for a steel HP 310x110 piles are provided in Table 3.2. It was assumed that the underside of the pile caps will be located at approximately Elev. 188 m.

**Table 3.2 – Axial Factored Geotechnical Resistance and Reaction for HP 310x110**

Abutment (Borehole)	Top of Pile Elevation (m)	Pile Length / Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
West (07-4)	188	12 / 176	800	650
East (07-10)	188	12 / 176	800	650

The value of the Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (Typical), and a Resistance Factor of 0.4 (Typical), as per CHBDC 2014. The Geotechnical Reaction at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

### 3.2.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving should be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and the designer should specify an ultimate pile resistance. Controlling of pile installation using the Hiley formula could start when the piles are within 2.0 m of the design tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS103-11 using an ultimate resistance of ‘R’ kN per pile”. ‘R’ should have a minimum value of twice the design load at ULS.

Pile tip protection is recommended for driven H-piles to prevent pile damage if cobbles or boulders are encountered. The tips of all driven H-piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Suggested wording for an NSSP on potential obstructions in the till is included in Appendix B.

### 3.2.3 Lateral Pile Resistance

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

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

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

Where  $z =$  depth of embedment of pile (m)



$D$	=	pile width or diameter (m)
$n_h$	=	coefficient related to soil relative density ( $\text{kN/m}^3$ )
$\gamma'$	=	effective unit weight ( $\text{kN/m}^3$ )
$K_p$	=	passive earth pressure coefficient

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

	$k_s$	=	$67 S_u / D$	( $\text{kN/m}^3$ )
	$p_{ult}$	=	$9 S_u$	(kPa)
Where	$S_u$	=	undrained shear strength (kPa)	
	$D$	=	pile width or diameter (m)	

The above equations and recommended parameters in Table 3.3 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

**Table 3.3 – Soil Parameters for Lateral Pile Resistance**

Soil Unit	Elevation (m)		$\gamma'$ (kN/m <sup>3</sup> )	$n_h$ (kN/m <sup>3</sup> )	$K_p$	$S_u$ (kPa)
	Top	Bottom				
West Abutment (07-4)						
Granular Fill (Very loose silty sand replaced)	188*	187	21	3,000	3.0	-
Silt	187	184	8	2,000	2.9	-
Silty Sand	184	180	10	4,000	3.2	-
Sand and Gravel	180	177	11	7,500	3.5	-
Clayey Silt Till	177	176	11	-	-	400
East Abutment (07-10)						
Granular Fill (Very loose silty sand replaced)	188*	187	21	3,000	3.0	-
Sandy Silt to Silt	187	182	8	2,000	2.9	-
Silty Sand	182	179	10	4,000	3.2	-
Clayey Silt Till	179	176	11	-	-	400

\* Assumed underside of pile cap

The spring constant,  $K_s$ , for analysis may be obtained by the expression,  $K_s = k_s L D$  ( $\text{kN/m}$ ), where  $k_s$  is the coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  $D$  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,  $P_{ult} = p_{ult} L D$ . This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 3.4. Intermediate values may be obtained by linear interpolation.

**Table 3.4 – Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing (Centre to Centre)	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

### 3.3 Recommended Foundation

From cost effectiveness and constructability perspectives, spread footings supported on granular engineered fill pads is the preferred foundation option at this site taking account of the site conditions and the requirements of modular bridge design.

### 3.4 Frost Cover

The depth of frost penetration at this site is approximately 1.4 m. If spread footings founded on native soil or piles are used, the base of footings/pile caps should be provided with a minimum 1.4 m of earth cover as protection against frost action.

For concrete spread footings resting on engineered fill pad, the frost-related movement is considered not a concern provided that the native silt subgrade underlying the minimum 1 m thick engineered fill pad is protected with a minimum of 1.4 m earth cover.

In the case of spread footings or sleeper slabs used under a modular bridge, it is considered acceptable to found these on granular pads with a minimum embedment of 0.5 m.

## 4 LATERAL EARTH PRESSURES

Backfilling behind the abutment walls should be carried out in accordance with OPSS 902.

Lateral earth pressures acting on the abutment walls may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill and the underlying soils. For a fully drained

condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

where

$p_h$  = horizontal pressure on the wall at depth  $h$  (kPa)

$K$  = earth pressure coefficient (see table below)

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

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

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the retaining walls are dependent on the material used as backfill. Recommended values of the earth pressure coefficients are shown in Table 4.1.

**Table 4.1 – Lateral Earth Pressure Coefficients**

Loading Condition	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ ; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ ; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Retained Soil)	3.7	-	3.3	-

In accordance with Clause 6.12.3 of the CHBDC 2014, a compaction surcharge should be added.

## 5 SEISMIC CONSIDERATIONS

Based on the undrained shear strength of the silty clay, Site Class D (stiff soil) should be assumed to evaluate the seismic site response, as per Table 4.1, Clause 4.4.3.2 of the CHBDC 2014.

The peak ground acceleration, PGA, for a 2% in 50-year probability of exceedance at this site is 0.107 as per the National Building Code of Canada (NBCC).

In accordance with Clause 4.6.5 of the CHBDC 2014, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 5.1 may be used:

**Table 5.1 – Earth Pressure Coefficients for Earthquake Loading**

Loading Condition	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.33	0.37
Passive ( $K_{PE}$ )	3.5	3.1
At-rest ( $K_{OE}$ )**	0.61	0.65

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

\*\* After Woods

Given the low seismic ground motions and the presence of predominantly compact to dense soil, potential for liquefaction is considered low at this site.

## 6 APPROACH EMBANKMENTS

The new approach embankments to the bridge will be approximately 2 m in height. Provided that the embankments are constructed with earth fill to slope inclinations not steeper than 2H:1V, the global stability of the new approach embankments is considered satisfactory.

Foundation settlement associated with the fill placement is estimated to be in the order of 30 mm, and will be essentially complete 1 to 2 weeks after the end of embankment construction. Preloading at the abutment locations is recommended prior to the footing construction to minimize footing settlement associated with fill placement. The preloading period is anticipated to be two weeks.

Embankment construction should be carried out in accordance with OPSS.PROV 206. In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from within the embankment footprint prior to placement of fill.

## 7 EROSION CONTROL

Erosion protection must be provided along any soil surfaces that may be in contact with the creek flow. In general, this will involve placement of rock protection on the creek valley slopes to prevent erosion and undermining of the embankments.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

## 8 EXCAVATION AND GROUNDWATER CONTROL

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native silty sand and silt are classified as Type 3 soils above the water level and Type 4 soils below the water level at this site.

Dewatering within the temporary excavation for footing construction using sump-and-pump method and surface runoff diversion is considered adequate at this site. However, fluctuations of the creek level and groundwater table should be expected. Design of the dewatering system is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility. The dewatering system must remain operational and effective throughout the construction. Suggested wording for an NSSP in this regard is included in Appendix B.

## **9 CONSTRUCTION CONCERNS**

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

- The water level in the creek may fluctuate and be at higher elevation at the time of construction than indicated in the report.
- The thickness and depth to the base of the very loose alluvial deposit may vary at locations away from the boreholes.
- Cobbles or other obstructions may be present within the native till. These materials may interfere with the pile installation.

## 10 CLOSURE

Engineering analysis and preparation of this foundation design report was carried out by Mr. Keli Shi, P.Eng., and Mr. Jason Lee, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

### THURBER ENGINEERING LTD.



Keli Shi, P.Eng.  
Senior Geotechnical Engineer



Jason Lee, P.Eng.  
Principal, Senior Geotechnical Engineer



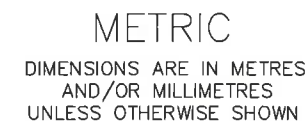
Dr. P. K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact



## **Appendix A**

### **Borehole Locations and Soil Strata Drawing Record of Borehole Sheets (Geocres No. 31C-180)**





SHEET  
14



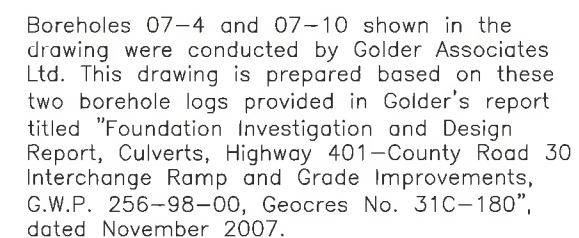
**THURBER ENGINEERING LTD**

[illegible]

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.

[illegible]

FILENAME: H:\Drafting\19\4406\20\led0620-Plan&Profile(CountvRd 30).dwg



Golder is solely responsible for the quality and accuracy of the borehole information provided in their report.



# RECORD OF BOREHOLE No 07-4

1 OF 1 **METRIC**

PROJECT 06-1111-057

W.P. 256-98-00

LOCATION N 4882094.4 ; E 202988.4

ORIGINATED BY SB

DIST HWY 401

BOREHOLE TYPE Track-Mounted CME-55, 108mm I.D. Hollow Stem Augers

COMPILED BY MWK

DATUM Geodetic

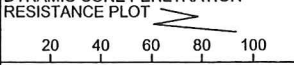
DATE May 9, 2007

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
187.6	GROUND SURFACE						20	40	60	80	100							
0.0	Silty SAND, trace clay, containing rootlets and organics Very loose Brown Moist		1	SS	WH	▽	187											
186.8							186											
0.8	SILT, trace to some sand, trace clay Loose to compact Grey Moist to wet		2	SS	8		185											
			3	SS	15		184											
			4	SS	11		183											
			5	SS	10		182											
183.8							181											
3.8	Silty SAND, trace to some gravel, some clay (TILL) Compact Grey Moist to wet		6	SS	3*		180							NP	11 49 32 8			
			7	SS	3*		179											
			8	SS	26		178											
180.3							177											
7.3	SAND and GRAVEL, trace to some silt Dense to very dense Grey Wet		9	SS	33		176								37 48 10 5			
			10	SS	70/15	175												
177.2																		
10.4	CLAYEY SILT, trace sand and gravel (TILL) Hard Grey Moist		11	SS	105													
			12	SS	104													
175.0																		
12.6	END OF BOREHOLE																	
	Notes:  1. * Low SPT "N" values (3 blows/0.3 m of penetration) are the result of sample disturbance due to groundwater inflow to the borehole.  2. Water level in open borehole at a depth of 1.5 m (Elev. 186.1 m) on completion of drilling.																	

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT 06-1111-057			RECORD OF BOREHOLE No 07-10			1 OF 1 METRIC					
W.P. 256-98-00			LOCATION N 4882111.7 ; E 202995.2			ORIGINATED BY SB					
DIST _____ HWY 401			BOREHOLE TYPE Track-Mounted CME-55, 108mm I.D. Hollow Stem Augers			COMPILED BY MWK					
DATUM Geodetic			DATE May 11, 2007			CHECKED BY LCC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
188.0	GROUND SURFACE										
0.0	Silty SAND, containing organics and rootlets Very loose Brown and black Moist		1	SS	1						
187.2											
0.8	Sandy SILT, trace gravel, containing rootlets Compact Grey Moist to wet		2	SS	15						
			3	SS	9						
185.7											
2.3	SILT, trace to some sand, trace clay Loose to compact Grey Wet		4	SS	9						
			5	SS	8						
			6	SS	6*						
			7	SS	6*						
182.2											
5.8	Silty SAND, trace gravel Compact Grey Wet		8	SS	5*						
			9	SS	4*						
178.9											
9.1	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist		10	SS	89						
			11	SS	96						
175.4											
12.6	END OF BOREHOLE		12	SS	107						
Notes: 1. * Low SPT "N" values (4, 5 and 6 blows/0.3 m of penetration) are the result of sample disturbance due to groundwater inflow to the borehole. 2. Water level in open borehole at a depth of 2.1 m (Elev. 185.9 m) on completion of drilling.											

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## **Appendix B**

### **List of OPSSs and OPSDs and Suggested Wording for NSSP**

**1. List of OPSS and OPSD Documents Relevant to this Project**

- OPSS.PROV 206
- OPSS.PROV 501
- OPSS.PROV 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010

**2. Suggested Wording for NSSP on “Dewatering”**

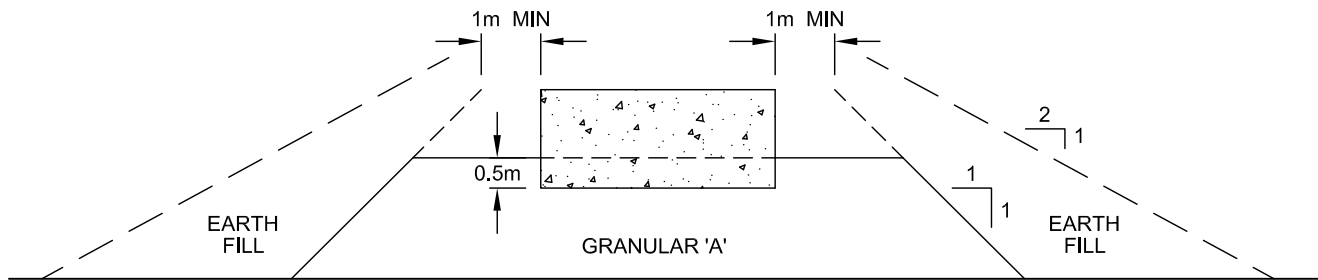
Dewatering shall be provided by the Contractor during excavation, fill pad construction and backfilling to allow the work to proceed in the dry. The Contractor is notified that failure to implement dewatering in advance of excavating below the groundwater table may result in sloughing and boiling of the soil in the excavation and a loss in stability and bearing resistance. Design and provision of an effective dewatering system is the responsibility of the Contractor. The dewatering system must remain operational and effective until the bridge foundations are completed and backfilled.

**3. Suggested Wording for NSSP on “Obstructions”**

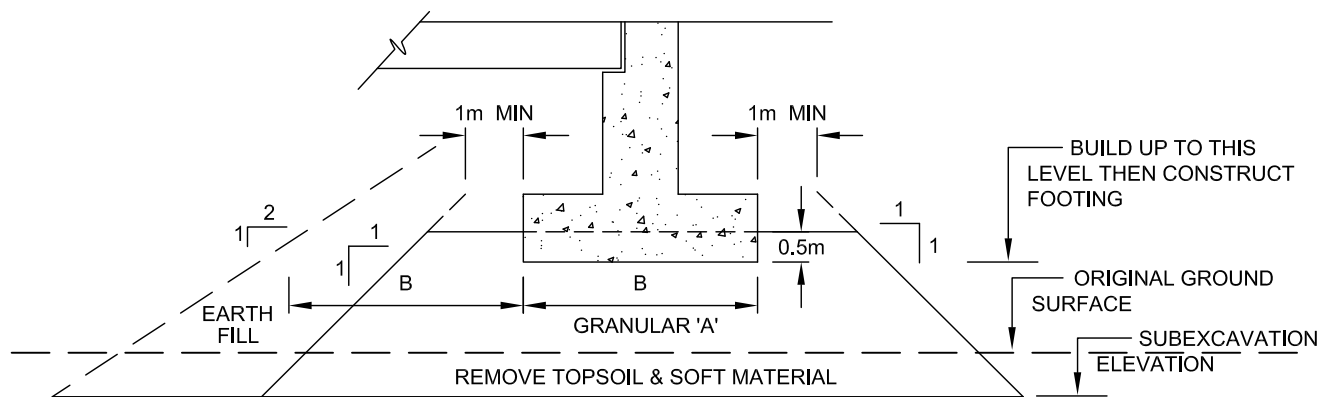
Installation of piles may encounter obstructions such as cobbles and boulders embedded in the native soils. Such obstructions may impede pile installation. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions to achieve the design pile tip elevations.

## Appendix C

**Figure 1 – Abutment Footing on Engineered Fill Pad**



## CROSS-SECTION



## LONGITUDINAL SECTION

### NOTES:

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

ABUTMENT ON COMPACTED FILL  
SHOWING GRANULAR 'A' CORE



**THURBER ENGINEERING LTD.**

ENGINEER :	KS	DRAWN :	MFA	APPROVED :	PKC
DATE :	MARCH 2017	SCALE :	N.T.S.	DRAWING No.	FIGURE 1



## **Appendix D**

### **Comparison of Foundation Alternatives**



Spread Footings on Native Soil	Spread Footings on Engineered Fill	Driven H-Piles
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Ease of construction.</li> <li>ii. Lower cost than deep foundations.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Low geotechnical resistance is available in native soils at shallow depths.</li> <li>ii. Dewatering may be required, depending on depth of excavation.</li> <li>iii. More expensive than the engineered fill option if footings are to be extended down to competent soil stratum at depth.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Easy of construction.</li> <li>ii. Less costly than deep foundations.</li> <li>iii. Allows use of perched abutments.</li> <li>iv. Higher geotechnical resistance than on native soil.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Cost of engineered fill placement.</li> <li>ii. Dewatering may be required during high water level season.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Piles will develop high geotechnical resistance in hard till.</li> <li>ii. Installation of piles could continue in freezing weather.</li> <li>iii. Requires less excavation than footings.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit costs than footings.</li> <li>ii. Cobbles and boulders may be encountered in the native till deposits.</li> </ul>
NOT RECOMMENDED	RECOMMENDED	NOT RECOMMENDED