

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 STRUCTURE REPLACEMENT
OAKVILLE CREEK WEST BRANCH BRIDGE
HALTON REGION, ONTARIO
G.W.P. 2188-10-00, SITE No. 10-74/1&2**

GEOCRES Number: 30M12-374

Report to

AECOM

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation carried out at the location of the proposed replacement of the existing Highway 401 Eastbound (EB) and Westbound (WB) Structure over Oakville Creek West Branch in the Regional Municipality of Halton, Ontario. This investigation was carried out in support of the preliminary design, environmental assessment and planning for the bridge replacement. These works are part of the project involving preliminary design for Highway 401 Structure Replacement from Trafalgar Road westerly to Halton Region Boundary.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide borehole locations and soil strata drawings, records of boreholes, stratigraphic profiles and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained from the present investigation and selected data from previous investigation.

Thurber was retained by AECOM to carry out the foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO) under Consultant Assignment No. 2012-E-0016.

2 SITE DESCRIPTION

The bridge is located approximately 2.7 km southwest of Trafalgar Road and 1.5 km northeast of James Snow Parkway along the Highway 401 alignment. The bridge carries Highway 401 over the west branch of Oakville Creek which meanders in a west-east direction before merging with Oakville Creek East Branch. The site is bounded by agricultural lands to east and abuts the 5th Line overpass approach embankment to west.

The terrain in the area is generally flat. The height of the existing approach embankments are approximately 8 m and 9 m at the west and east abutments, respectively. The existing structure comprises a single span rigid frame structure with a span length of about 25 m and width of about 37 m. It crosses the creek at a 24 degree skew angle.

The site lies in the physiographic region known as the Peel Plain. The Plain slopes gently to the south towards Lake Ontario. The surface topography in the area is typically characterized by a surficial till sheet mainly consisting of silty clay to clayey silt (Halton Till) which overlies discontinuous glacially derived gravel, sand and silt deposit over bedrock. The Upper Ordovician bedrock underlying the area belongs to the Queenston Formation, characterized by thinly-bedded, reddish brown shale.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for the bridge consisted of advancing two boreholes (14-44 and 14-48) at the proposed bridge abutments and four boreholes (14-45, 14-46, 14-47 and 14-49) for the proposed widening of the approach embankments. The field work was carried out between July 17 and 22, 2014. The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawings attached in Appendix D. Previous boreholes (10-401 and 10-402) from a Preliminary Foundation Investigation and Design Report prepared by Golder Associates dated October 2011 (Geocres No. 30M12-328) are included in Appendix C.

In the present investigation, the borehole locations were staked and/or marked in the field. Utility clearance was obtained for all borehole locations prior to drilling. Borehole location data including northing, easting and surface elevation has been derived based on the preliminary design information provided by AECOM to Thurber.

Track-mounted D-50 and D-120 drill rigs were used in combination with continuous flight hollow/solid stem augers to advance the boreholes through embankment fill and native soils. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). The bedrock was cored for 3 m in Borehole 14-48. Borehole 14-44 was terminated upon refusal on inferred bedrock surface. Boreholes 14-45, 14-46, 14-47 and 14-49 drilled for the proposed widening of the approach embankments were advanced to approximately 10 m depth.

Upon auger refusal on bedrock in Borehole 14-48, HQ-sized coring equipment was used to obtain 3 m rock core in the bedrock. All rock cores were logged, and properties including Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and Fracture Indices (FI) were determined where applicable.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes 14-45, 14-46 and 14-48 to permit monitoring of the groundwater levels. The standpipe piezometer typically consists of 19 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screen positioned in the soil strata where groundwater fluctuations are to be monitored. The sand screen surrounded the pipe and extended at least 0.3 m above the slotted screen. Bentonite holeplug seals were placed above the sand screen in each installation to seal the annular space. Following the final water level reading, the piezometers were decommissioned in general accordance with MOE Regulation 903.

The drilling and sampling operations were supervised on a full-time basis by members of Thurber's

technical staff. The supervisors logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and natural moisture content determination. The results of the testing are shown on the Record of Borehole sheets attached in Appendix A. Selected soil samples were subjected to gradation analysis. The results of this laboratory testing program are presented on the Record of Borehole sheets in Appendix A and on the Figures in Appendix B.

Selected rock cores were subjected to Point Load Test (PLT). Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are shown on the Record of Borehole sheets in Appendix A.

5 SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in these records and on the "Borehole Locations and Soil Strata" drawings in Appendix D. Borehole information from a previous investigation is included in Appendix C. A general description of the subsurface conditions encountered in the current investigation is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of the site conditions.

In general, the subsurface conditions consist of a surficial layer of clayey silt to silty clay underlain by a relatively thick layer of sandy clayey silt till which overlies interbedded layers of sand and silt till, sand and gravel and clayey silt till above reddish brown Queenston shale bedrock. The sandy clayey silt till contains occasional sand layer.

5.1 Topsoil

Topsoil was encountered in Boreholes 14-44, 14-46 and 14-48 at the existing ground surface. Thickness of the topsoil encountered ranged from 50 to 125 mm.

5.2 Clayey Silt to Silty Clay (Fill)

Brown clayey silt to silty clay fill was encountered beneath the topsoil in Boreholes 14-44 and 14-48. Thickness of the fill encountered in the boreholes was about 0.6 m. The base elevations of the fill varied from 192.9 to 193.7 m.

Standard Penetration Tests (SPT) conducted within the fill gave 'N' values ranging from 18 to 25 blows per 0.3 m penetration, indicating a very stiff consistency. The measured natural moisture contents of the fill samples ranged from 10 to 31%.

5.3 Clayey Silt to Silty Clay

A layer of brown native clayey silt to silty clay was encountered below the embankment fill in Boreholes 14-44 and 14-48, at the surface in Boreholes 14-45, 14-47 and 14-49, and

below the topsoil in Borehole 14-46. The cohesive deposit contains some sand and trace gravel. Thickness of this layer ranged from 0.5 to 2.6 m with the base of the layer at elevations varying from 190.3 to 195.9 m.

Standard Penetration Tests (SPT) conducted within the layer produced 'N' values ranging from 3 to 15 blows per 0.3 m penetration, indicating soft to stiff consistency. The measured natural moisture contents of the samples ranged from 6 to 29%.

Results of grain size analyses conducted on selected clayey silt to silty clay samples are presented in Figure B1, and are summarized as follows:

Gravel	0 to 7%
Sand	25 to 38%
Silt	41 to 49%
Clay	16 to 28%

The result of one Atterberg Limits test conducted on the silty clay sample are provided on the Record of Borehole sheets in Appendix A and illustrated in Figure B6 of Appendix B. The results indicated that the deposit has a plastic limit of 18% and a liquid limit of 37%, suggesting low to intermediate plasticity.

5.4 Sandy Clayey Silt (Till)

A layer of brown to grey sandy clayey silt till was encountered below the clayey silt to silty clay layer in all boreholes. The layer contains trace to some gravel, and occasional cobbles and shale fragments. The layer was fully penetrated in Boreholes 14-44 and 14-48, where thicknesses of 7.0 m and 6.8 m were measured with the base of the layer at elevations 183.3 m and 185.3 m, respectively. Boreholes 14-45, 14-46, 14-47 and 14-49 were terminated within the layer at elevations ranging from 184.5 to 186.6 m.

Standard Penetration Tests (SPT) conducted within the layer produced 'N' values ranging from 14 to 80 blows per 0.3 m penetration, indicating stiff to hard consistencies. The measured natural moisture contents of the samples typically ranged from 2 to 34%.

Results of grain size analyses conducted on selected sandy clayey silt till samples are presented in Figures B2a, B2b and B2c, and are summarized as follows:

Gravel	0 to 12%
Sand	29 to 43%
Silt	35 to 56%
Clay	10 to 25%

The results of Atterberg Limits tests conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A and illustrated in Figures B7a and B7b of Appendix B. The results indicated that the deposit has plastic limits ranging from 11 to 14% and liquid limits ranging from 17 to 23%, suggesting low plasticity.

Glacial tills inherently contain cobbles and boulders.

5.5 Sand

A layer of greying brown sand was encountered within the sandy clayey silt till only in Borehole 14-47. The sand contains some silt and clay and trace gravel. Thickness of the sand was about 1.1 m with the base of the layer at elevation 192.3 m.

Standard Penetration Test (SPT) conducted within the sand deposit produced an 'N' value of 39, indicating a dense relative density. The measured natural moisture content of the sand sample was about 9%.

Result of grain size analysis conducted on a sand sample is presented in Figure B3, and indicates that the sand contains 7% gravel, 69% sand and 24% fines.

5.6 Sand and Silt (Till)

A layer of brown sand and silt till was encountered beneath the sandy clayey silt till in Boreholes 14-44 and 14-48. The sand and silt till contains trace clay and gravel, and occasional cobbles and shale fragments. Borehole 14-44 was terminated upon refusal on probable bedrock at elevation 178.1 m. Thickness of the sand and silt till layer in Boreholes 14-44 and 14-48 were 5.2 m and 3.9 m with the base of the layer at elevations 178.1 m and 181.4 m, respectively.

Standard Penetration Tests (SPT) conducted within the sand and silt till deposit produced 'N' values ranging from 43 blows per 0.3 m penetration to 100 blows for 0.075 m penetration, indicating dense to very dense relative densities. The measured natural moisture contents of the sand and silt till samples ranged from about 4 to 10%.

Results of grain size analyses conducted on selected sand samples are presented in Figure B4, and are summarized as follows:

Gravel	6 to 18%
Sand	39 to 51%
Silt	26 to 40%
Clay	6 to 10%

Glacial tills inherently contain cobbles and boulders.

5.7 Sand and Gravel

A layer of sand and gravel was encountered underlying the sand and silt till in Borehole 14-48. Thickness of the sand and gravel was about 2.9 m with the base of the layer at elevation 178.5 m.

Standard Penetration Tests (SPT) conducted within the sand and gravel deposit produced 'N' values of 83 blows per 0.3 m penetration and 100 blows for 0.1 m penetration, indicating a very dense relative density. The measured natural moisture content of the sand

and gravel sample was 10%.

Result of grain size analysis conducted on sand sample is presented in Figure B5, and indicates that the sand and gravel contains 38% gravel, 40% sand and 22% fines.

5.8 Lower Clayey Silt (Till)

A layer of reddish brown clayey silt till was encountered beneath the sand and gravel layer in Borehole 14-48 before contacting shale bedrock. The clayey silt till contains some sand to sandy and shale fragments. Thickness of the layer was approximately 3.0 m with the base of the layer at elevation 175.5 m.

Standard Penetration Tests (SPT) conducted within the layer produced ‘N’ values of 100 blows for 0.075 m and 0.125 m penetration, indicating hard consistency. The measured natural moisture contents of the sand samples ranged from about 8 to 13%.

5.9 Shale Bedrock

Shale bedrock (Queenston Formation) was encountered in Boreholes 14-44 and 14-48 below the clayey silt till. Bedrock was proven by 3 m coring in Borehole 14-48. The bedrock encountered in Borehole 14-48 was generally described as heavily to slightly weathered, thinly bedded, fine grained, reddish brown shale with frequent hard grey limestone interbeds up to 100 mm in thickness.

One Standard Penetration Test (SPT) conducted within the heavily weathered shale produced an ‘N’ value of 100 blows for 0.075 m penetration.

Total Core Recovery (TCR) of the bedrock ranged from 92 to 100%. Rock Quality Designation (RQD) values ranged from 51 to 84%, indicating fair to good rock quality. Fracture Index (FI) of the rock cores typically ranged from 0 to 7 with two values greater than 10 recorded in Run 2. The following table summarizes the depths to bedrock and the bedrock surface elevations encountered in the current and previous boreholes.

Foundation Element	Borehole	Depth to Bedrock below Ground Surface (m)	Bedrock Elevation (m)
West Abutment	14-44	15.4	178.1 (Inferred)
	10-401	15.9	177.3
East Abutment	10-402	20.7	180.5 (Inferred)
	14-48	18.9	175.5

The unconfined compressive strengths (UCS) of the intact rock cores, estimated from the results of point load tests (PLT) conducted on the rock core samples, range between 11.7 and 24.7 MPa, indicating weak intact rock. The UCS values are included on the Record of Borehole sheets in Appendix A.

5.10 Groundwater Conditions

Groundwater conditions were observed in the open boreholes upon completion of drilling. The measured groundwater levels in the open boreholes and the standpipe piezometers are presented in the table below.

Borehole	Date	Conditions	Groundwater Level	
			Depth (m)	Elevation (m)
14-45	July 25, 2014	Piezometer	2.7	191.7
	July 28, 2014		3.2	191.2
	Aug. 11, 2014		2.4	192.0
14-46	July 24, 2014	Piezometer	1.6	192.7
	July 25, 2014		1.1	193.2
	July 28, 2014		1.4	192.9
	Aug. 11, 2014		1.1	193.2
14-48	July 25, 2014	Piezometer	1.4	193.5
	July 28, 2014		1.7	193.2
	Aug. 11, 2014		1.3	193.6

The high water level in the Oakville Creek was reported to be at elevation 194.25. It should be noted that all groundwater observations at this site are short term. The groundwater levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

The drilling and sampling equipment was supplied and operated by Walker Drilling Ltd. of Barrie, Ontario and Altech Drilling & Investigation Services Ltd. Of Waterloo, Ontario. Walker Drilling Ltd. supplied a track-mounted D-50 drill rig and Altech Drilling & Investigation Services Ltd. supplied a track-mounted D-120 drill rig for the duration of the investigation.

Traffic protection during the drilling operation was provided by Direct Traffic Management Inc. of Hamilton, Ontario.

The field work was supervised on a full time basis by Mr. George Azzopardi, Mr. Luke Gilarski and Mr. Justin Gray of Thurber Engineering Ltd. Overall supervision of the field program was conducted by Mr. Mark Farrant, P.Eng.

The report was prepared by Mr. Keli Shi, P. Eng., and reviewed by Mr. Alastair Gorman, P.Eng. and 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 GENERAL

This report provides an interpretation of the geotechnical data in the factual report and presents preliminary foundation design recommendations to assist the design team in the selection and design of a suitable foundation system for the Highway 401 Oakville Creek West Branch crossing.

The preliminary General Arrangement (GA) drawing provided by AECOM indicates that the replacement bridge will be a 44 m long single span bridge with integral abutments founded on driven piles. The existing approach embankments will be widened towards the north and south to accommodate the replacement bridge with a total roadway width of about 78.9 m. Reinforced soil systems (RSS) wing walls are proposed for the widened approach embankments.

The proposed replacement bridge will essentially maintain the existing road grade. Fill heights at the proposed abutments will be in the order of 8.5 m and 7.5 m at the west and east, respectively. It is understood that the replacement structure will be constructed in stages in order to maintain traffic flow on the existing Highway 401.

The discussion and recommendations presented in this report are based on the information provided by AECOM and on the factual data obtained during the course of the current investigation.

8 STRUCTURE FOUNDATION

In general, the soil stratigraphy at the bridge site consists of a surficial soft to stiff clayey silt to silty clay layer underlain by a relatively thick layer of very stiff to hard sandy clayey silt till which overlies interbedded layers of very dense sand and silt till, sand and gravel and hard clayey silt till over reddish brown Queenston shale bedrock. The highest groundwater level measured in the standpipe piezometers was at about elevation 193.6. This level is controlled by the creek level and will be higher after periods of wet weather. The reported high water level in the creek is at elevation 194.25.

Based on the subsurface conditions, initial consideration was given to supporting the replacement bridge on spread footings on native soil or engineered fill, driven steel H-piles, or augered caissons. A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix F.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is recommended.

8.1 Spread Footings on Native Soil

Based on the subsurface conditions encountered at this site, the use of spread footings to support the abutments is considered feasible from a geotechnical perspective. Spread footings, however, must be protected from scour and erosion. Spread footings should be founded on undisturbed, very stiff to hard native clayey silt till. Table 8.1 summarizes the recommended founding elevations at the abutment locations and the recommended geotechnical resistances assuming a minimum 2 m wide footing subjected to vertical concentric loading.

Table 8.1 – Recommended Founding Elevations and Geotechnical Resistances for Spread Footings

Foundation Element	Borehole No.	Recommended Highest Founding Elevation (m)	Bearing Stratum	Factored ULS (kPa)	SLS (kPa)
West Abutment	14-44	190.3	Hard Clayey Silt Till	600	400
	10-401	189.5			
East Abutment	10-402	192.0			
	14-48	192.0			

The exposed native material at the subgrade level should be protected from disturbance such as construction traffic and weathering. Any topsoil and soft/loose fill or native material should be stripped from the footprint of the footing. The footing subgrade must be protected from deterioration by a 100 mm thick concrete working slab.

Where eccentric or inclined loads are applied, the resistances used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The geotechnical resistance at SLS is based on an estimated settlement not exceeding 25 mm. This settlement will be essentially complete by the end of construction.

The lateral resistance developed along the base of concrete footings founded on the above soils may be computed using an ultimate friction coefficient of 0.45.

Excavation and backfilling for the footings must be in accordance with OPSS 902.

Construction of a footing will be below the prevailing creek water level. Dewatering will likely be required during footing construction to maintain an unwatered condition. Design of the dewatering system is the responsibility of the contractor. However, one possible

system would be pumping within a cofferdam.

The spread footing option is not recommended at this site due to the need for a relatively deep excavation requiring dewatering close to the creek and the need to provide scour protection.

8.2 Spread Footings on Engineered Fill

The founding levels may be raised by placing the footings on engineered fill constructed over the native very stiff to hard clayey silt till. The base of the engineered fill pad must be placed at or below the founding levels provided in Table 8.1. The engineered fill must 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 footing.

Provided a minimum footing width of 2 m is maintained, footings bearing on an engineered fill pad at least 2.0 m thick may be designed for the following values:

- Factored Geotechnical Resistance at ULS = 900 kPa
- Geotechnical Resistance at SLS = 350 kPa

The lateral resistance of footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6.

Construction of an engineered fill pad will be below the creek water level at the recommended founding levels. Dewatering will be required during construction of the engineered fill pad to maintain an unwatered condition.

For the same reason outlined above for spread footings on native soils, spread footings on engineered fill is not recommended at this site.

8.3 Steel H-Pile Foundations

The ground conditions at the site are considered to be suitable for the use of driven steel H-piles.

8.3.1 Axial Resistance

While some piles driven at this site are expected to reach the weathered shale bedrock, others may achieve refusal in the overlying hard to very dense till. Accordingly, pile resistances consistent with bearing in the overburden are recommended for design.

The anticipated pile tip elevations and design geotechnical resistances for HP 310x110 piles are presented in Table 8.2.

Table 8.2 – Anticipated Pile Tip Elevation and Recommended Geotechnical Resistance for H-Piles

Foundation Element	Borehole No.	Anticipated Pile Tip Elevation	Bearing Stratum	Factored ULS (kPa)	SLS (kPa)
West Abutment	14-44	178.1	Weathered Shale Bedrock or Hard Clayey Silt Till	1,800	1,600
	10-401	177.3			
East Abutment	10-402	180.5	Very Dense Sand and Gravel		
	14-48	179.0			

The pile tips must be driven to the anticipated elevations or deeper if this is necessary to develop the geotechnical resistance.

8.3.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage. The tips of all driven H-piles must be fitted with driving shoes in accordance with OPSD 3000.100.

8.3.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving must be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and an ultimate pile resistance should be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design pile tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile. “R” must have a minimum value of twice the design load at ULS but a maximum of 3,600 kN.

8.3.4 Pile Lateral Resistance

The geotechnical lateral resistance acting on a pile in cohesionless soil may be calculated using 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 along pile (m)
	D	=	pile width or diameter (m)
	n_h	=	coefficient related to soil density (kN/m^3)
	γ'	=	effective unit weight (kN/m^3)
	K_p	=	coefficient of passive lateral earth pressure

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

follows:

$$k_s = 67 S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 S_u \quad (\text{kPa})$$

Where S_u = undrained shear strength (kPa)

D = pile width or diameter (m)

The above equations and recommended parameters in Table 8.3 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 8.3 – Soil Parameters for Lateral Pile Resistance

Location	Soil Unit	Elevation (m)		γ' (kN/m ³)	n_h (kN/m ³)	K_p	S_u (kPa)
		Top	Bottom				
West Abutment	Fill	196.0 *	193.5	21	2,500	3.0	-
	Clayey Silt/Silty Clay	193.5	190.3	10	-	-	80
	Clayey Silt Till	190.3	183.3	11	-	-	200
	Sand and Silt Till	183.3	178.1	12	7,500	4.2	-
East Abutment	Fill	196.0 *	194.3	21	2,500	3.0	-
	Clayey Silt/Silty Clay	194.3	192.0	10	-	-	80
	Clayey Silt Till	192.0	185.3	11	-	-	200
	Sand and Silt Till	185.3	175.5	12	7,500	4.2	-

Note: * Assumed underside of pile cap elevation.

The above equations and recommended parameters 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.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), 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 geotechnical failure of the pile occurs and will not support any additional load at greater displacement.

According to the CHBDC Clause C6.8.7.1 and Table C6.4, lateral resistance for steel HP310 x 110 piles embedded in stiff cohesive soil should be limited to 160 kN and 65 kN under ULS (factored) and SLS conditions, respectively.

The coefficient 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 8.4. Intermediate values may be obtained by linear interpolation.

Table 8.4 – 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

Consideration may be given to the use of battered piles if lateral pile capacities higher than the available geotechnical lateral resistances are required.

8.4 Caissons / Drilled Shafts

Caisson installation at this site would extend below the clayey silt till into cohesionless sand and silt till and sand and gravel below groundwater level and require the use of a permanent liner to support the caisson sidewalls. Sealing of the caisson liner to prevent inflow of water and cohesionless soils from the base will be problematic. Caissons may encounter cobbles and boulders, and cleaning and base inspection will be difficult. The use of caissons is therefore not recommended and the design recommendations have not been developed.

8.5 Downdrag

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

8.6 Recommended Foundation

From a geotechnical and cost perspective, driven steel H-piles are the preferred foundation option for this site. Spread footings are feasible but not recommended in view of the requirement for dewatering and scour protection.

8.7 Frost Cover

The depth of frost penetration at this site is approximately 1.2 m. The base of footings or pile caps must be provided with a minimum of 1.2 m of earth cover as protection against frost action.

9 EXCAVATION AND DEWATERING

All excavations must be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and native clayey silt to silty clay within the depth of excavation may be classified as Type 3 soils. The clayey silt till within the depth of excavation may be classified as Type 2 soil. Flatter slopes may be required at locations where water seepage affects stability of an excavation.

The selection of the method of excavation is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a

hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and cobbles and boulders in the till.

It is understood that bridge rehabilitation will be carried out in stages to maintain the highway traffic at all times. Roadway protection will be required to facilitate staging. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

The design of any roadway protection or dewatering system that may be required is the responsibility of the Contractor. All shoring systems should be designed by a professional engineer experienced in such design.

10 RETAINED SOIL SYSTEMS (RSS)

Based on the preliminary design information provided by AECOM, both abutments will have RSS wingwalls. The RSS walls will be stepped up at a slope of 2H: 1V along the approaches away from the abutments.

In general, RSS walls used in conjunction with the new abutments must 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.

To provide an acceptable foundation performance, the RSS mass must be founded on competent soils or well compacted engineered fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

It is recommended that the RSS walls be founded on well-compacted granular fill or soft to very stiff native clayey silt to silty clay. Walls founded on the above materials should be designed for a Factored Geotechnical Resistance at ULS of 225 kPa and a Geotechnical Reaction at SLS of 150 kPa.

The above geotechnical resistance values are estimated for a horizontal ground surface in front of the wall and may have to be reduced for ground surface sloping down in front of the wall.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7. The resistance values assume that the RSS wall reinforcement will extend a distance behind the wall face of approximately 70% of the wall height.

A minimum 500 mm thick layer of bedding material conforming to OPSS Granular “A” requirements should be provided under the RSS mass to provide a uniform subgrade condition. Engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip. Any topsoil and soft/loose fill or native material should be stripped from the footprint of the RSS. All disturbed and new embankment fill must be compacted in accordance

with OPSS 501. Suggested text for a NSSP addressing these issues is included in Appendix E.

The reinforced earth block must also be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on native clayey silt to silty clay and engineered granular fill may be estimated using ultimate friction coefficients of 0.35 and 0.55, respectively. The internal stability of the RSS wall should be analysed by the supplier/designer of the proprietary product selected for this site.

In view of the soil conditions at this site, the estimated foundation settlement beneath RSS walls is expected to be less than 25 mm and will be essentially complete at the end of construction.

11 EMBANKMENT WIDENING

The approach embankments will be widened by approximately 21 m each to the north and south of the existing embankments.

Global stability of the proposed widening of the approach embankments is not considered to be an issue with permanent embankment slopes inclined at 2H: 1V or flatter for earth or granular embankment.

Settlement induced by the proposed fill placement is anticipated to be less than 40 mm and will be essentially complete at the end of construction.

When placing new fill against the existing embankment, benching will be required for the existing embankment slopes in accordance with OPSD 208.010. A mid-height berm comprising a 2 m wide bench should be incorporated along the length of embankments with fill heights exceeding 8 m.

12 LATERAL EARTH PRESSURES

Backfill to the abutment walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type II material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the backfill. 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 = coefficient of lateral earth pressure (see Table 12.1)

γ = unit weight of retained soil (see Table 12.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 12.1.

The coefficients provided in Table 12.1 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 C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

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.

Table 12.1 – Coefficients of Lateral Earth Pressure

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or 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 Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.31	0.47*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design in accordance with the CHBDC for a design earthquake with 475-year return period:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Ground Acceleration 0.04 g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

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. For the design of retaining walls, the coefficients of lateral earth pressure in Table 13.1 may be used.

Table 13.1 – Coefficient of Lateral Earth Pressure for Seismic Loading

Loading Condition	Earth Pressure Coefficient (K_E) for Seismic Loading			
	OPSS Granular A or 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 Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.29	0.42	0.32	0.51
At-rest (K_{OE})**	0.46	-	0.51	-
Passive (K_{PE})*	3.5	-	3.1	-

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

** After Woods (1973).

Based on review on the SPT data, seismically-induced liquefaction of foundation soils is not anticipated under the design earthquake.

14 SCOUR AND EROSION PROTECTION

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

Erosion protection should be provided along any soil surfaces that may be in contact with the creek flow. In particular, erosion protection must be provided to prevent undermining of the RSS walls at the abutments.

15 CONSTRUCTION CONCERNS

During construction, the Contract Administrator (CA) should retain an experienced geotechnical engineer to observe foundation construction activities and to provide advice to the CA regarding any issues that need to be referred to the design team.

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

Protection of the Existing Structure and Roadway Remaining in Service

During the staged replacement of the existing structure, portions of the existing structure and travelled lanes must remain in service. The Contractor must provide adequate protection, e.g. shoring, to ensure that the performance of the existing foundations is not compromised and the roadway is protected.

Pile Installation

If piles are meeting refusal at higher elevations than anticipated, the issue should be referred to the contract administrator (CA) for comment and guidance.

Excavation and Dewatering

Any excavation carried out below the prevailing groundwater level runs a significant risk of being destabilized due to the inflow of groundwater. Adequate shoring and groundwater control

measures must be in place to maintain the stability of the excavation and to prevent loss of ground under the structure or embankment.

Monitoring of Existing Structure

Settlement and vibration monitoring of the existing structure should be carried out during pile driving or excavation for footing construction.

16 INVESTIGATION FOR DETAIL DESIGN

During the detail design phase, the designers must review the available geotechnical information to determine if it is adequate to support the proposed design. If there are information gaps at the final foundation locations or in the approach embankments, additional investigation must be carried out in accordance with MTO standards.

17 CLOSURE

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

THURBER ENGINEERING LTD.

Keli Shi, P.Eng.
Geotechnical Engineer



Alastair E. Gorman, P.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.
Review Principal



Appendix A
Record of Borehole Sheets

RECORD OF BOREHOLE No 14-44

1 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 636.0 E 276 428.4 ORIGINATED BY LPG/SP
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.17 - 2014.07.17 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				w _p w w _L				
193.5							20	40	60	80	100					
0.0	TOPSOIL: (50mm)															
192.9	Clayey SILT, some sand, trace gravel Very Stiff Brown Dry (FILL)		1	SS	25								○			
0.6																
	Clayey SILT, with SAND, trace gravel Stiff to Firm Brown Moist		2	SS	15								○			
			3	SS	13								○			4 38 42 16
			4	SS	5								○			
190.3	Clayey SILT, Sandy to with SAND, trace gravel Hard Brown Moist (TILL) Slow augering from 3.0m to 6.7m		5	SS	52								○			
3.2																
			6	SS	36								○			4 36 43 17
			7	SS	41								○			
			8	SS	61								○ H			2 29 56 13
								</								

Continued Next Page

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

RECORD OF BOREHOLE No 14-44

2 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 636.0 E 276 428.4 ORIGINATED BY LPG/SP
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.17 - 2014.07.17 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
Continued From Previous Page							<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div> <div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div><div>W_P W W_L</div><div>WATER CONTENT (%)</div><div>204060</div></div>							
183.3														
10.2	SAND and SILT , trace clay, trace gravel, occasional cobbles and shale fragments Dense to Very Dense Brown Dry to Moist (TILL)		11	SS	45		183							11 39 40 10
							182							
				12	SS	55		181						
								180						
				13	SS	100/ 0.225		179						6 51 37 6
178.1			14	SS	100/									
15.4	END OF BOREHOLE AT 15.4m. REFUSAL ON PROBABLE BEDROCK (SHALE)				0.200									

+³, ×³: Numbers refer to Sensitivity
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(%) STRAIN AT FAILURE

METRIC

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+³, ×³: Numbers refer to Sensitivity

ONTMT4S 3896A.GPJ 2012TEMPLATE(MTO).GDT 8/14/14

RECORD OF BOREHOLE No 14-45

2 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 683.8 E 276 465.1 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.19 - 2014.07.19 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2014.07.25 2.7 191.7 2014.07.28 3.2 191.2 2014.08.11 2.4 192.0													

RECORD OF BOREHOLE No 14-46

1 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 588.8 E 276 477.2 ORIGINATED BY GA
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.22 - 2014.07.22 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) w _p w w _L				GR	SA	SI	CL		
194.3								20	40	60	80	100									
0.0	TOPSOIL: (125mm)																				
0.1	Silty CLAY , some sand to sandy, trace gravel, trace organics Stiff to Firm Brown to Grey Moist		1	SS	9		194							○							
			2	SS	6		193							○							
			3	SS	10									○							
192.1																					
2.2	Clayey SILT , with SAND to sandy, trace gravel Hard Brown Moist (TILL)		4	SS	30		192							○				2	38	44	16
			5	SS	36		191							○							
			6	SS	37		190							○							
			7	SS	31		188							○	H			5	38	41	16
			8	SS	44		187							○							
			9	SS	61		185							○				9	31	43	17
184.5																					
9.8	END OF BOREHOLE AT 9.8m.																				

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+³, ×³: Numbers refer to
Sensitivity

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15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 14-46

2 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 588.8 E 276 477.2 ORIGINATED BY GA
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.22 - 2014.07.22 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page																
	BOREHOLE OPEN TO 9.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2014.07.24 1.6 192.7 2014.07.25 1.1 193.2 2014.07.28 1.4 192.9 2014.08.11 1.1 193.2																

METRIC

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+³, ×³: Numbers refer to Sensitivity

ONTMT4S 3896A.GPJ 2012TEMPLATE(MTO).GDT 7/29/14

RECORD OF BOREHOLE No 14-47

2 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 704.8 E 276 484.6 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.20 - 2014.07.20 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.													

RECORD OF BOREHOLE No 14-48

1 OF 3

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 641.3 E 276 518.1 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Hollow Stem Augers/HQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.07.20 - 2014.07.22 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
194.4								20	40	60	80	100					
0.0	TOPSOIL: (120mm)																
0.1	Silty CLAY, sandy, trace rootlets Very Stiff Brown Moist (FILL)		1	SS	18									○			
193.7																	
0.7	Silty CLAY, sandy, trace gravel, trace rootlets Stiff to Firm Brown Moist		2	SS	15									○			5 25 42 28
			3	SS	5									○			
192.1																	
2.3	Clayey SILT, with SAND to sandy, trace gravel, occasional cobbles and shale fragments Hard to Very Stiff Reddish Brown Moist (TILL)		4	SS	73									○			
			5	SS	40									○			10 37 39 14
	Brownish Grey		7	SS	20									○			6 35 42 17
	Grey		8	SS	25									○			
185.3																	
9.1	SAND and SILT, trace clay, some gravel, occasional cobbles Very Dense Brown Moist (TILL)		9	SS	43									○			18 47 26 9

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 14-48

2 OF 3

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 641.3 E 276 518.1 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Hollow Stem Augers/HQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.07.20 - 2014.07.22 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
	Continued From Previous Page							20 40 60 80 100								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
								20 40 60 80 100								
181.4																
			10	SS	87											
			11	SS	100/ 0.075											
13.0	SAND and GRAVEL , some silt Very Dense Reddish Brown Wet															
			12	SS	83											
	no recovery		13	SS	100/ 0.100											
178.5																
15.9	Clayey SILT , some sand to sandy, trace gravel, with shale fragments Hard Reddish Brown Moist (TILL)		14	SS	100/ 0.075											
			15	SS	100/ 0.125											
175.5																
18.9	SHALE , heavily weathered Reddish Brown															
174.7																
19.7	SHALE , moderately weathered, fine grained, laminated to thinly bedded		16	SS	100/ 0.075											

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

METRIC



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RECORD OF BOREHOLE No 14-49

1 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 656.9 E 276 535.9 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.20 - 2014.07.20 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _P	W						W _L
195.6																				
0.0	Silty CLAY , with SAND Soft to Firm Brown Moist		1	SS	3															
			2	SS	7															
194.2																				
1.4	Clayey SILT , sandy to with SAND , occasional limestone and shale fragments Stiff to Hard Brown to Grey Moist (TILL)		3	SS	14															
					4	SS	21													
					5	SS	29													
					6	SS	55													
			7	SS	55															
			8	SS	40															
			9	SS	30															
185.8																				
9.8	END OF BOREHOLE AT 9.8m.																			

Continued Next Page

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

RECORD OF BOREHOLE No 14-49

2 OF 2

METRIC

GWP# 2188-10-00 LOCATION Oakville Creek West Branch Bridge N 4 823 656.9 E 276 535.9 ORIGINATED BY JAG
 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.07.20 - 2014.07.20 CHECKED BY MEF

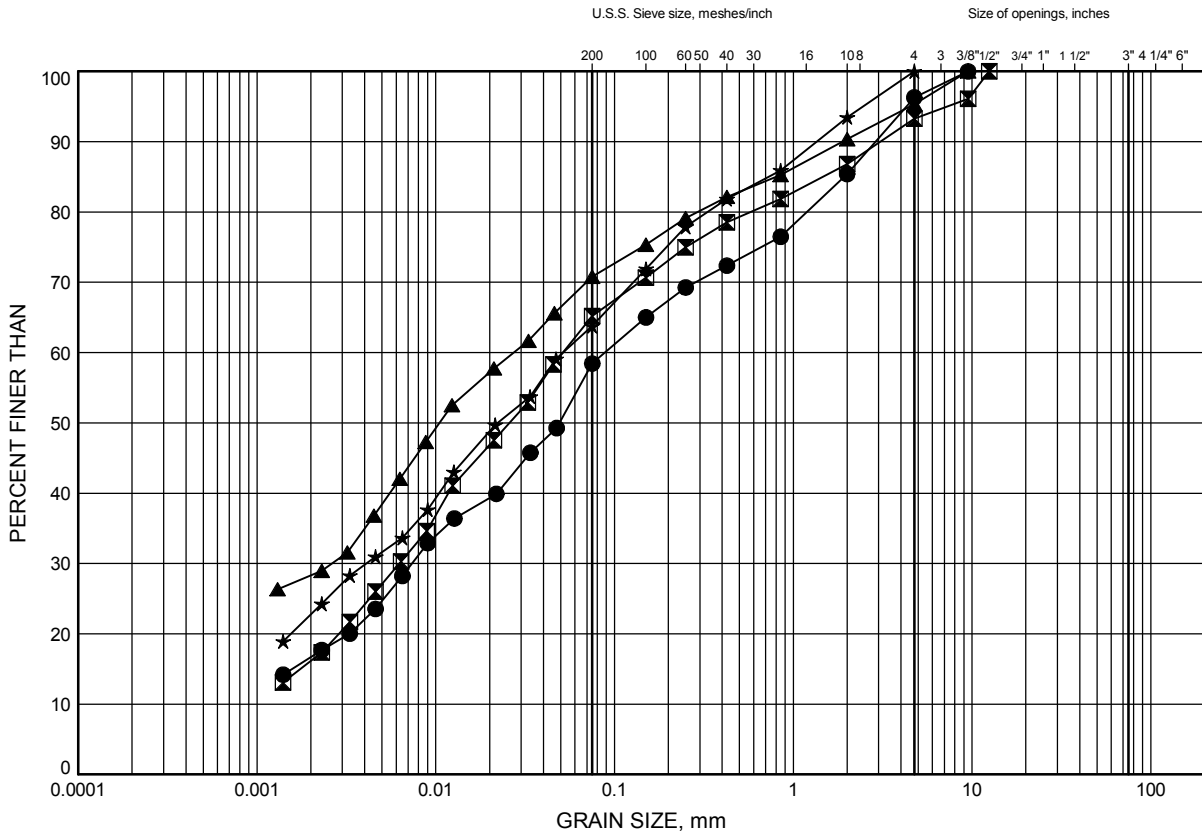
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.													

Appendix B
Laboratory Test Results

Oakville Creek West Branch Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

Clayey Silt to Silty Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-44	1.83	191.67
⊠	14-47	0.30	196.10
▲	14-48	1.07	193.33
★	14-49	1.07	194.53

Date July 2014
GWP# 2188-10-00

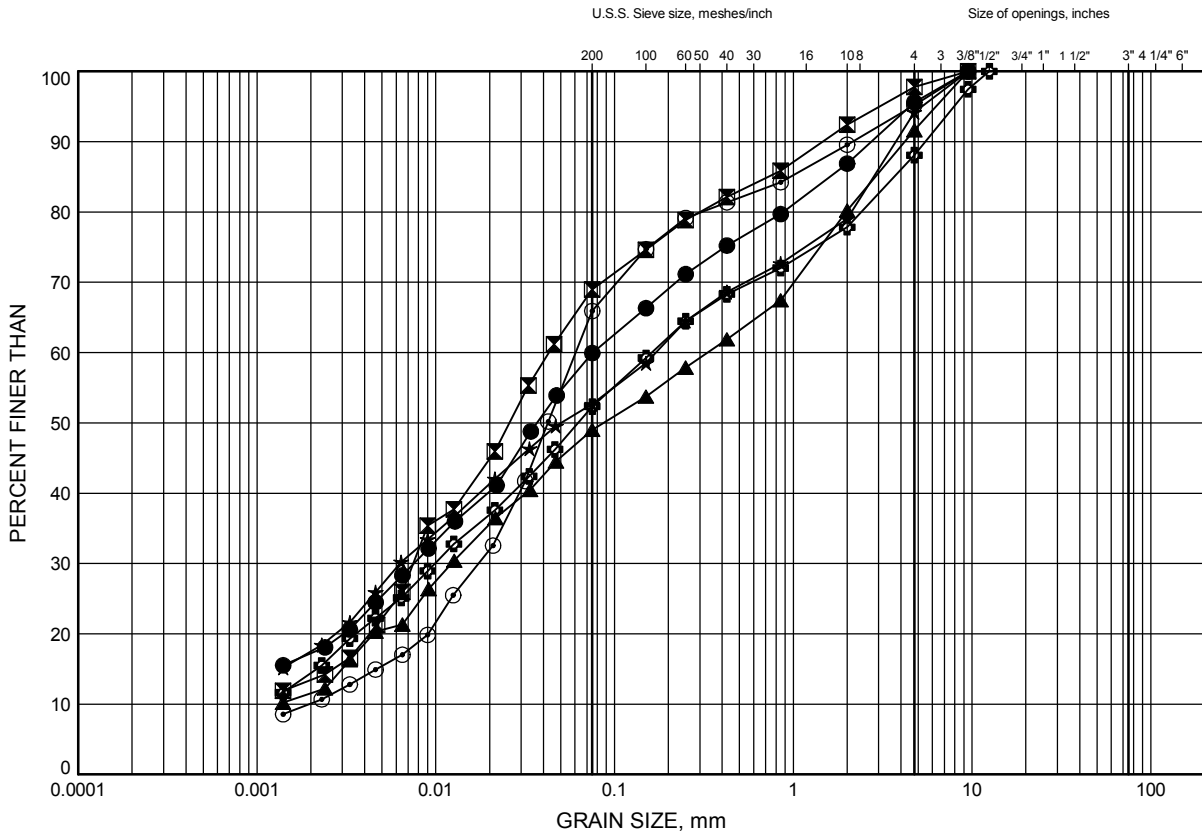


Prep'd MFA
Chkd. KS

Oakville Creek West Branch Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2a

Sandy Clayey Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-44	4.11	189.39
⊠	14-44	6.40	187.10
▲	14-45	1.07	193.33
★	14-45	2.59	191.81
⊙	14-45	4.88	189.52
⊕	14-45	9.45	184.95

Date July 2014

GWP# 2188-10-00



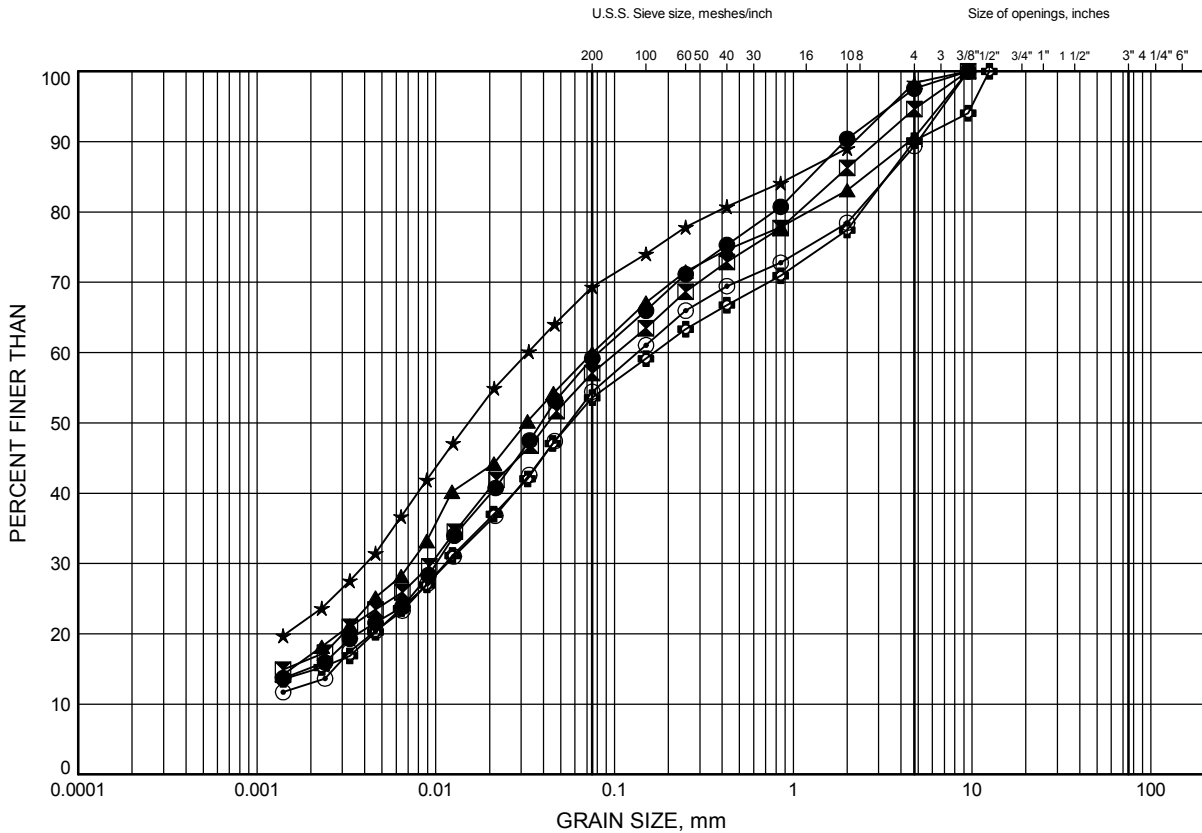
Prep'd MFA

Chkd. KS

Oakville Creek West Branch Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2b

Sandy Clayey Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-46	2.59	191.71
⊠	14-46	6.40	187.90
▲	14-46	9.45	184.85
★	14-47	1.83	194.57
⊙	14-47	9.45	186.95
⊕	14-48	3.35	191.05

Date July 2014

GWP# 2188-10-00



Prep'd MFA

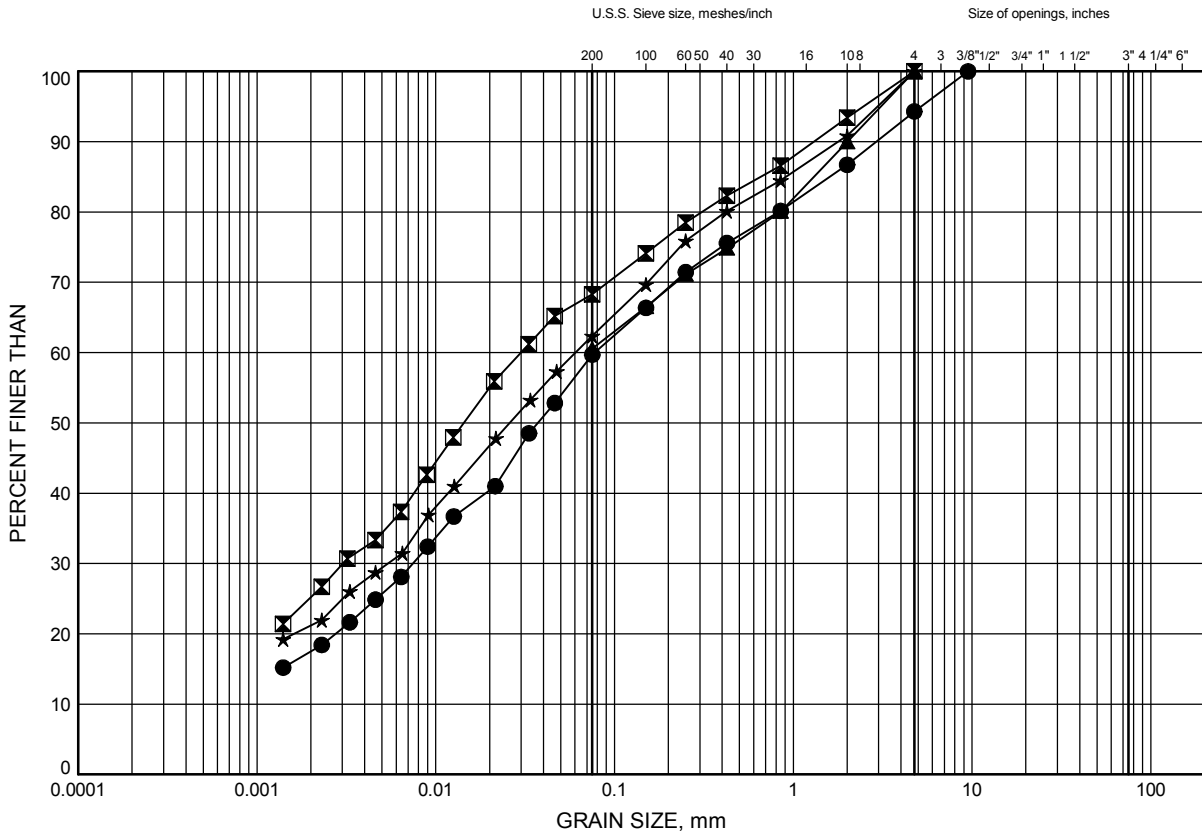
Chkd. KS

Oakville Creek West Branch Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2c

Sandy Clayey Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-48	6.40	188.00
⊠	14-49	3.35	192.25
▲	14-49	6.40	189.20
★	14-49	9.45	186.15

Date July 2014

GWP# 2188-10-00



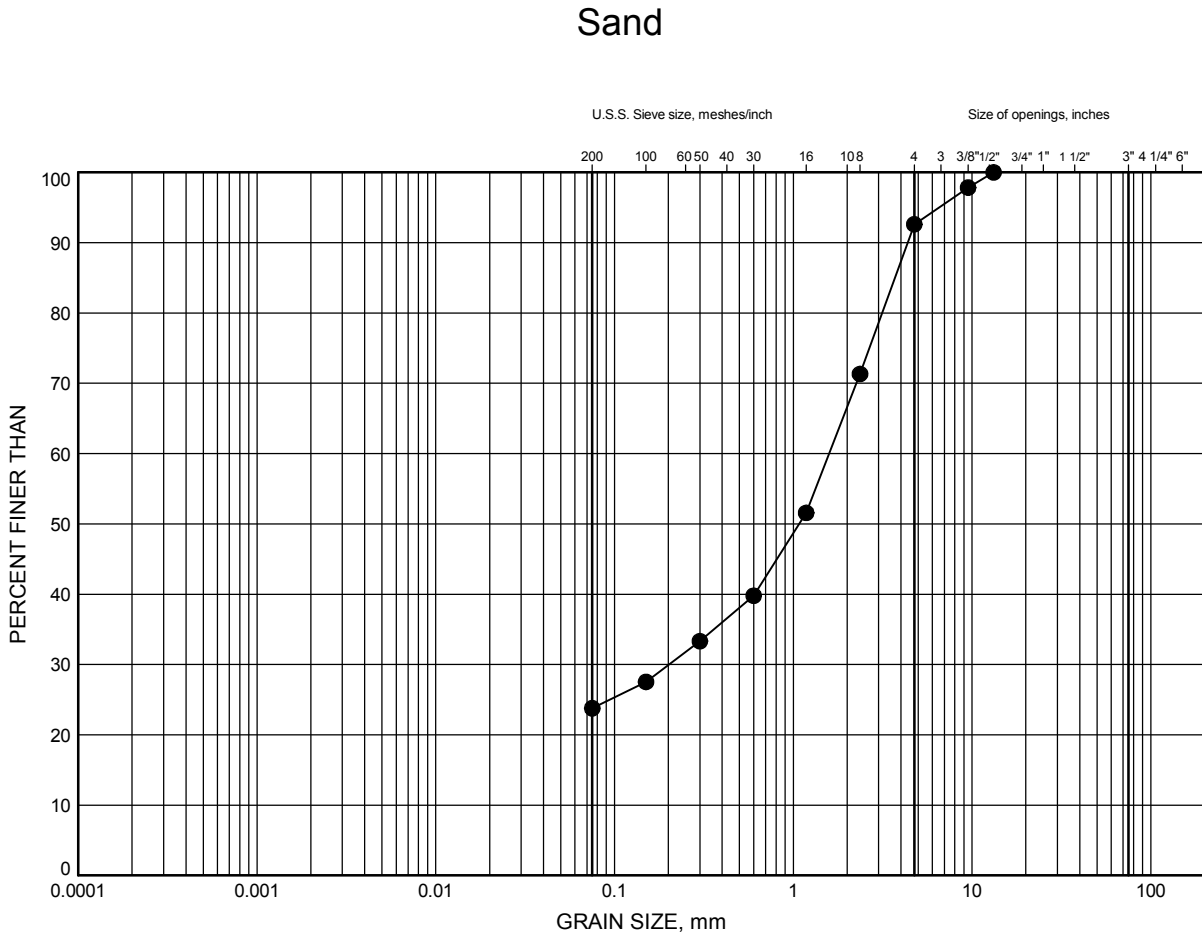
Prep'd MFA

Chkd. KS

Oakville Creek West Branch Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-47	3.35	193.05

Date July 2014
GWP# 2188-10-00

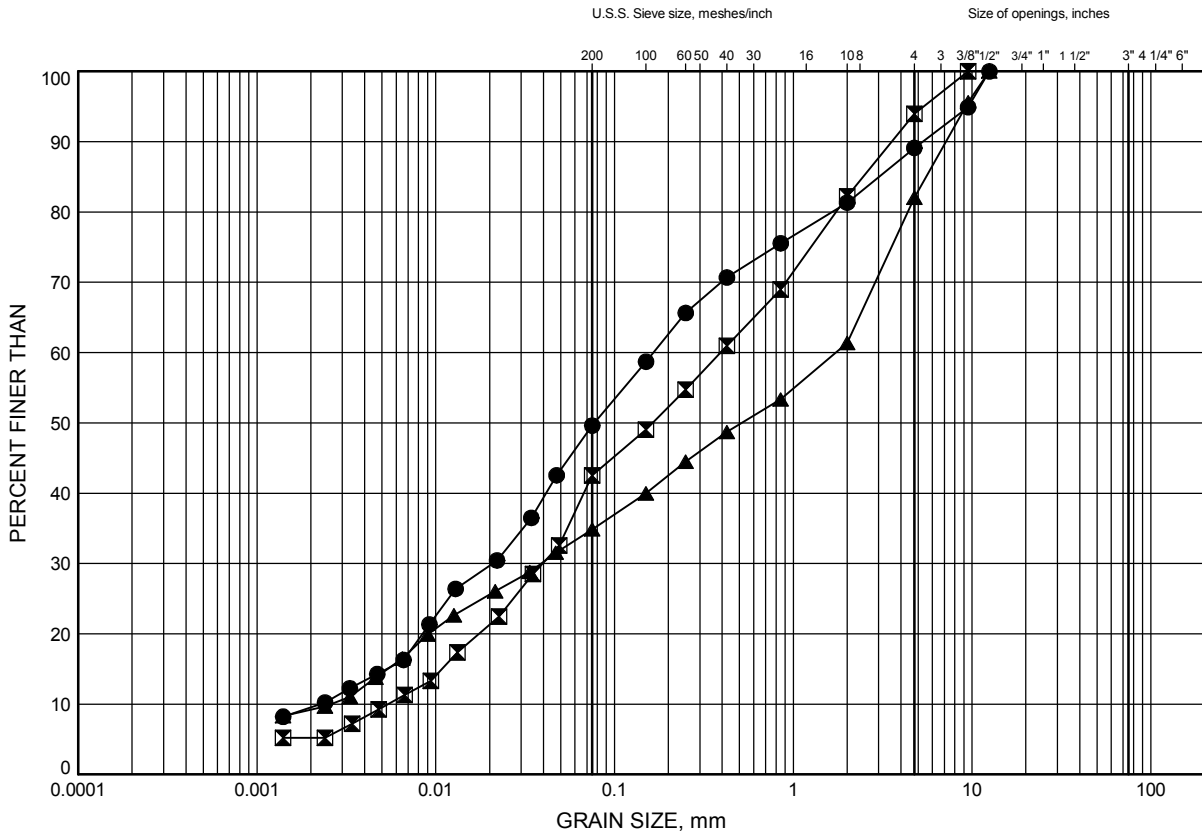


Prep'd MFA
Chkd. KS

Oakville Creek West Branch Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4

Sand and Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-44	10.97	182.53
⊠	14-44	13.90	179.60
▲	14-48	9.45	184.95

Date July 2014
GWP# 2188-10-00

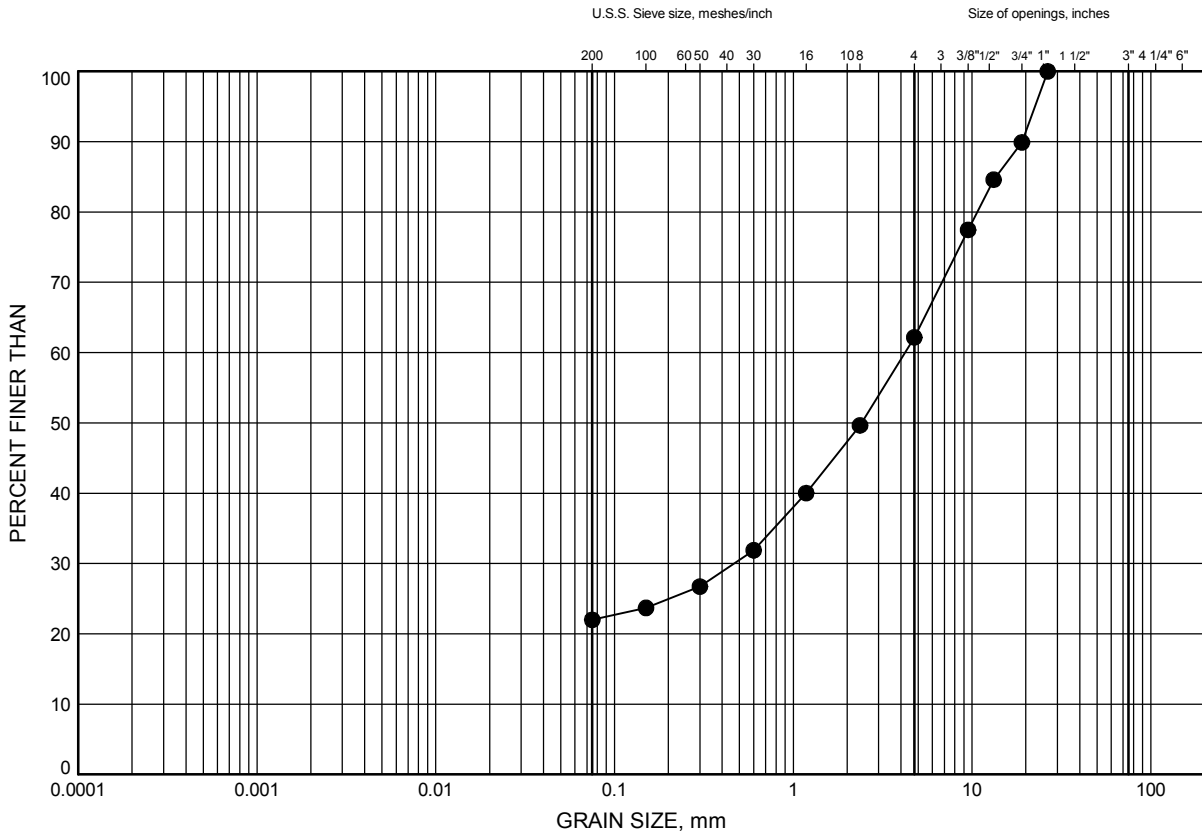


Prep'd MFA
Chkd. KS

Oakville Creek West Branch Bridge GRAIN SIZE DISTRIBUTION

FIGURE B5

Sand and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-48	14.02	180.38

Date July 2014
GWP# 2188-10-00

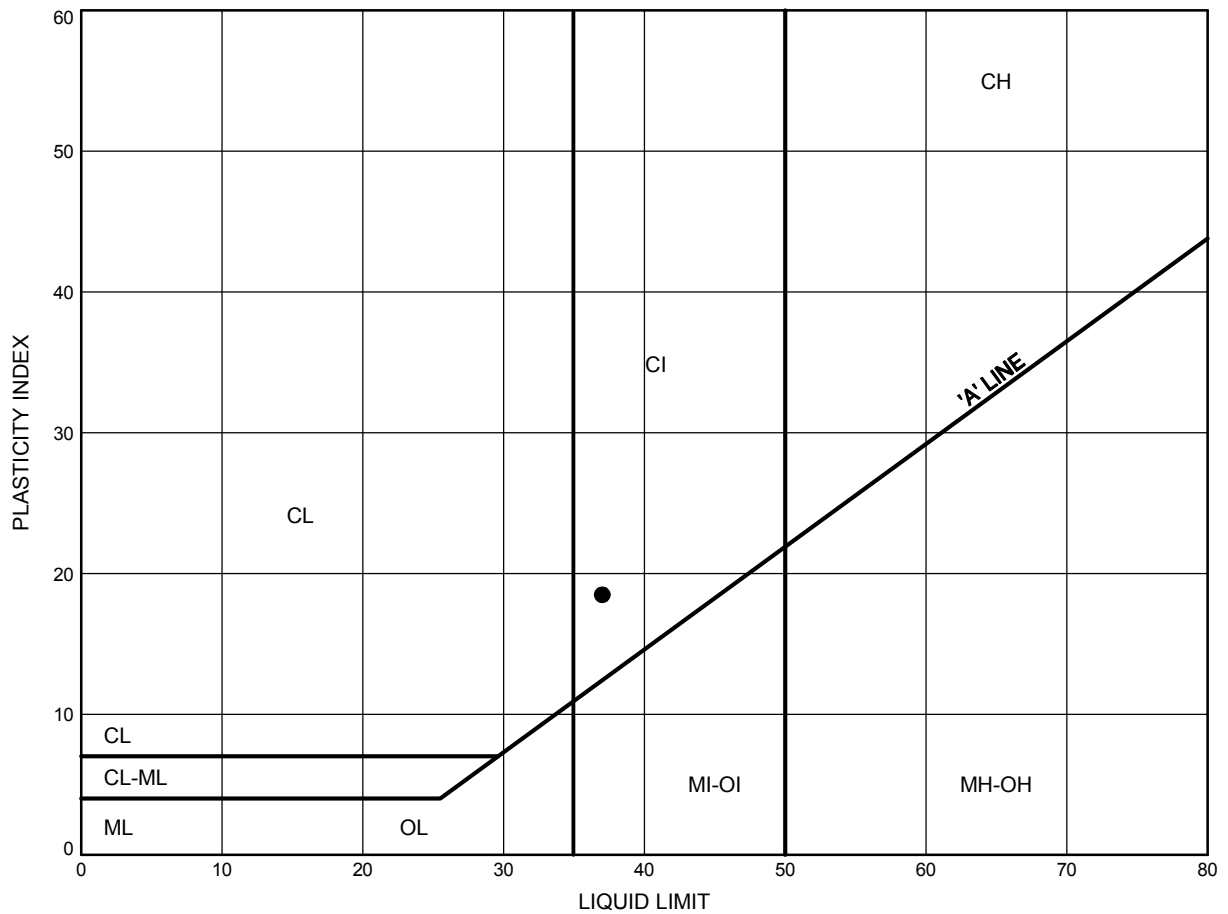


Prep'd MFA
Chkd. KS

Oakville Creek West Branch Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Silty Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-48	1.07	193.33

Date July 2014
 GWP# 2188-10-00

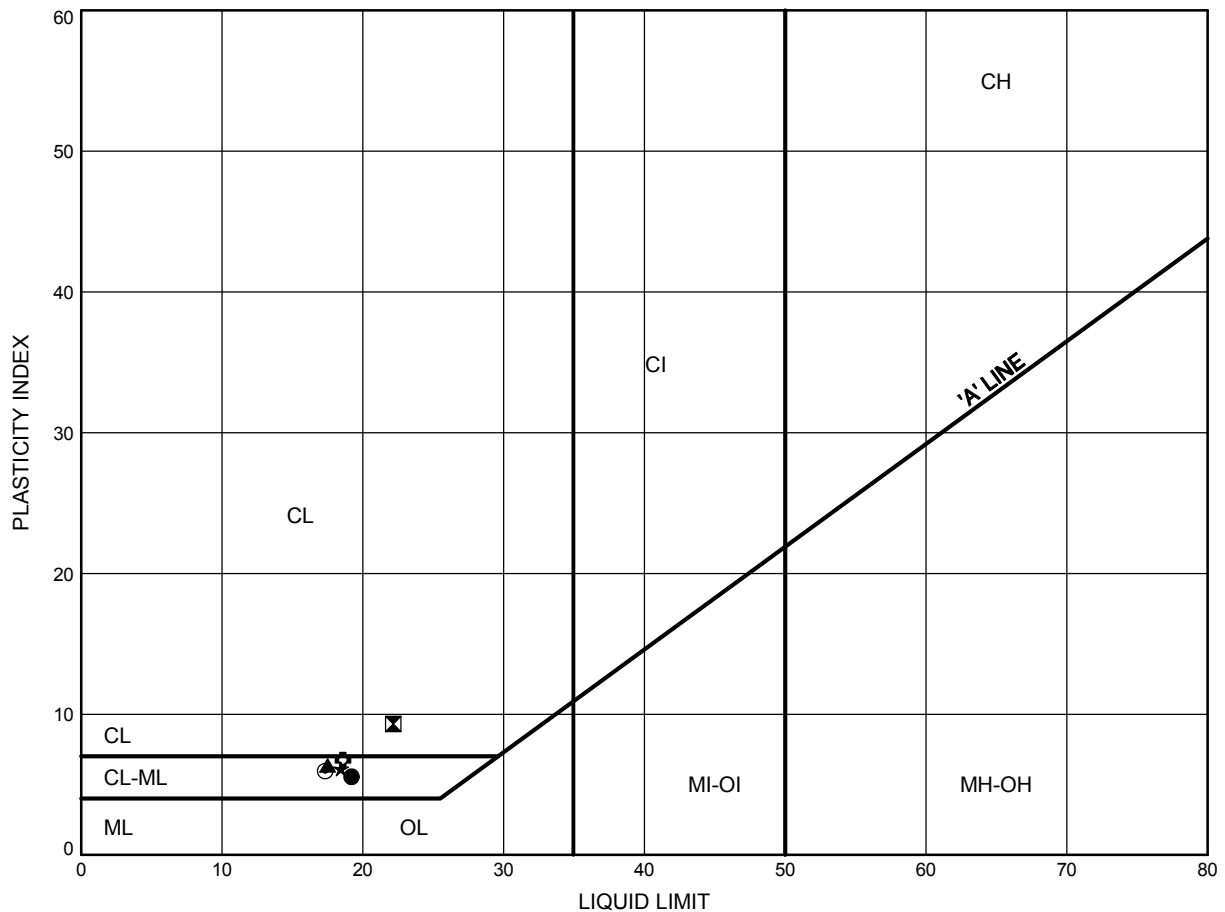


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 Chkd. KS

Oakville Creek West Branch Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B7a

Sandy Clayey Silt (TILL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-44	6.40	187.10
⊠	14-45	2.59	191.81
▲	14-45	9.45	184.95
★	14-46	6.40	187.90
⊙	14-47	9.45	186.95
✱	14-48	6.40	188.00

Date July 2014
 GWP# 2188-10-00

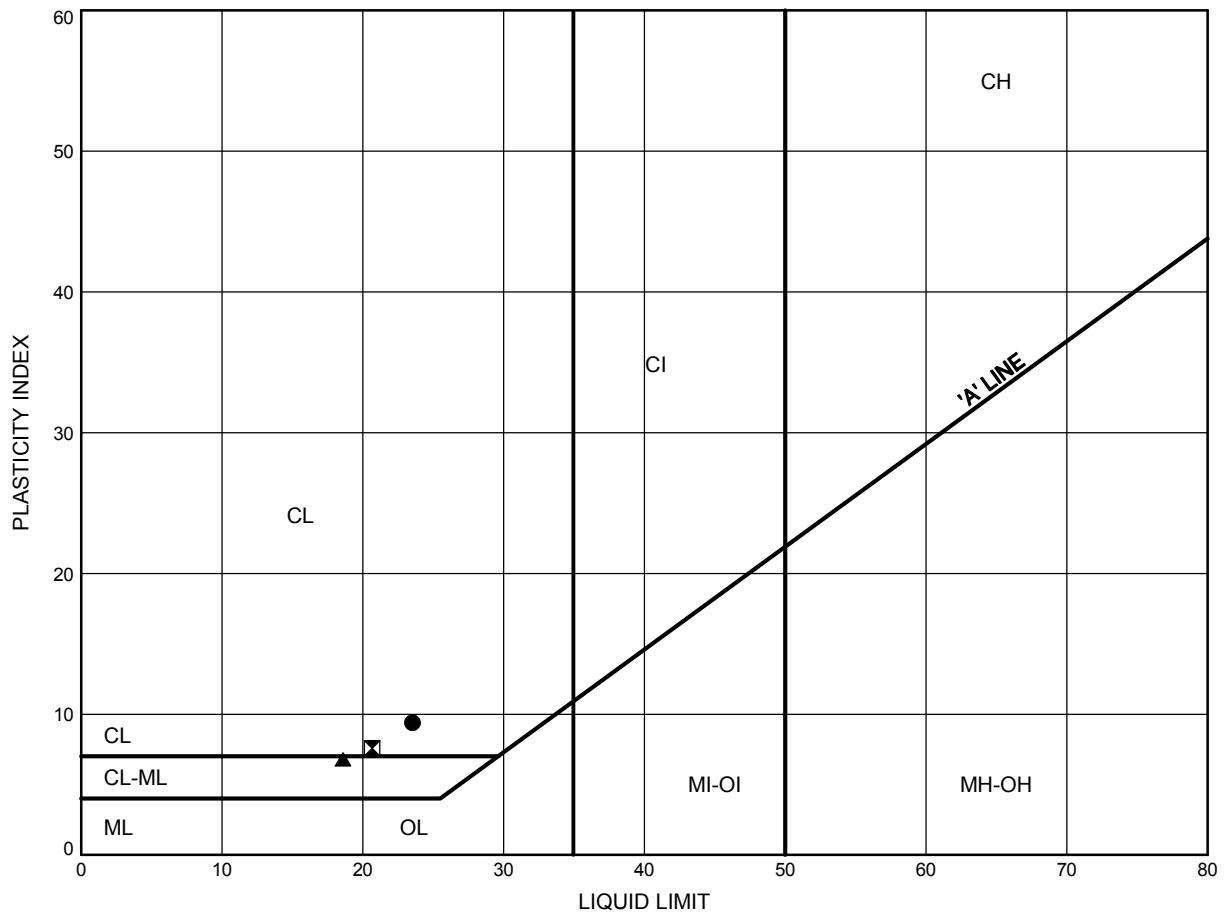


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 Chkd. KS

Oakville Creek West Branch Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B7b

Sandy Clayey Silt (TILL)



LEGEND

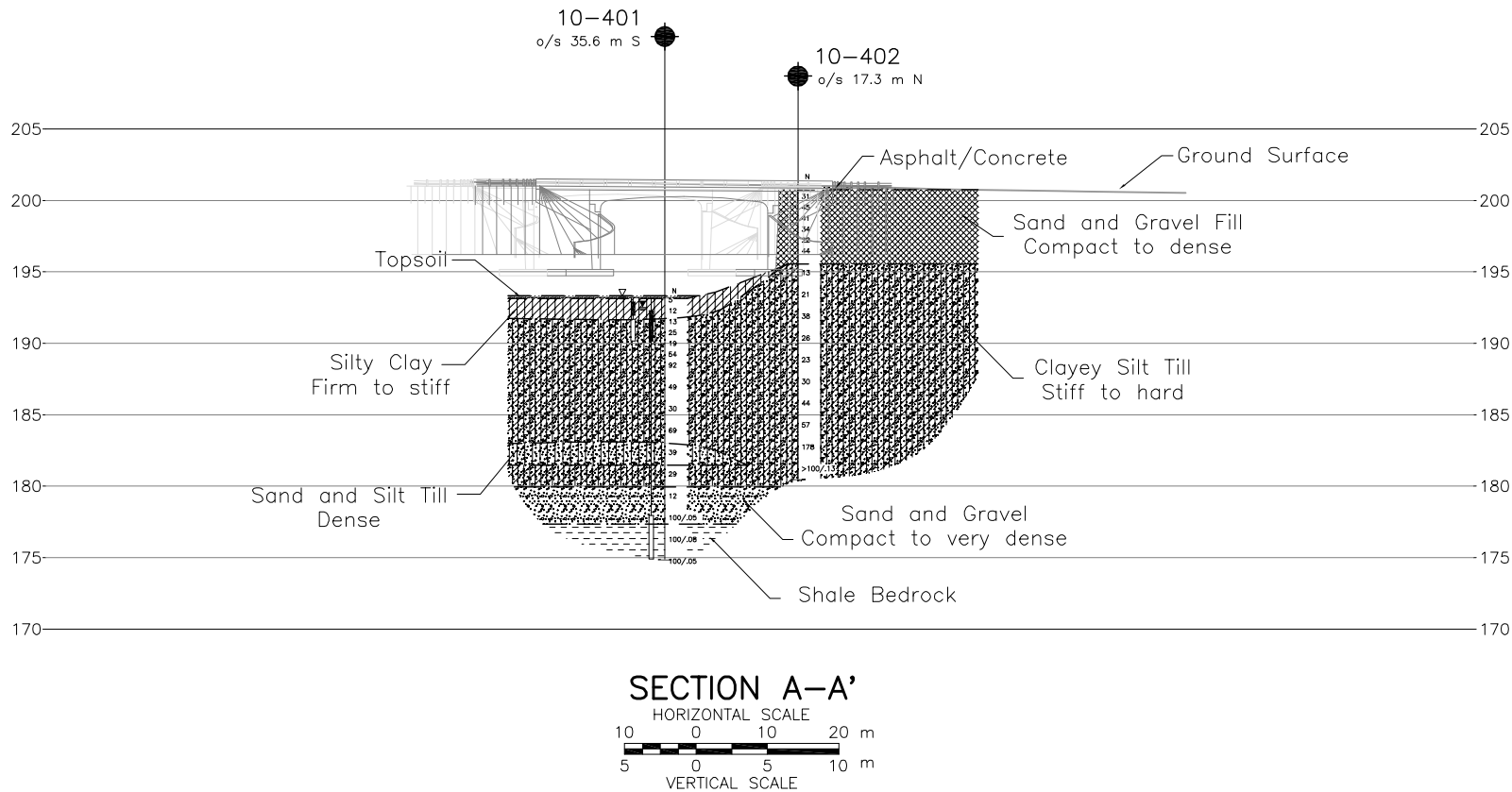
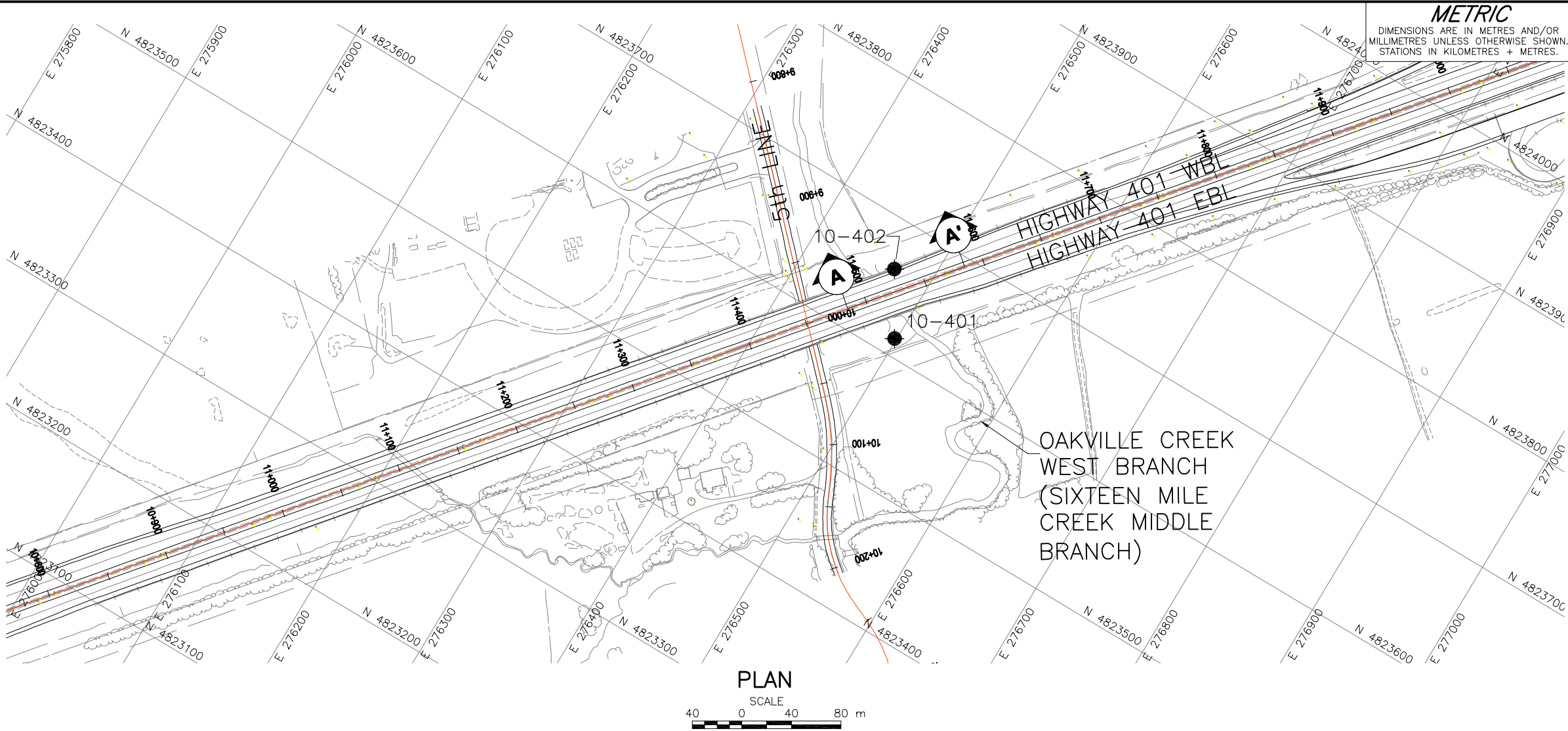
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-49	3.35	192.25
⊠	14-49	6.40	189.20
▲	14-49	9.45	186.15

Date July 2014
 GWP# 2188-10-00



Prep'd MFA
 Chkd. KS

Appendix C
Historical Borehole Information

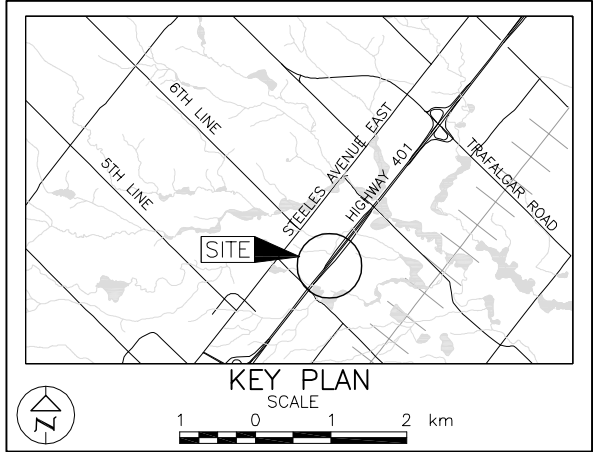


CONT No.
WO No. 07-20024

OAKVILLE CREEK WEST BRANCH (SIXTEEN
MILE CREEK MIDDLE BRANCH) BRIDGE
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND				
	Borehole - Current Investigation			
N	Standard Penetration Test Value			
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)			
	WL upon completion of drilling			
	WL in piezometer, measured on April 21, 2011			

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
10-401	193.2	4823614.1	276498.5
10-402	201.2	4823662.1	276469.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the design configuration as shown elsewhere in the Preliminary Design Project.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS. (Drawing File X-Align_401.dwg, received March 10, 2011, X-contour.dwg, received August, 2010, and X-Base, received September, 2010.



NO.	DATE	BY	REVISION
Geocres No. 30M12-328			
HWY. 401	PROJECT NO. 09-1111-6036		DIST.
SUBM'D. AM	CHKD. LCC	DATE: 10/21/2011	SITE:
DRAWN: JFC/CD	CHKD. AM	APPD. LCC	DWG. 1

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-401		1 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4823614.1 ; E 276498.5</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Track-Mounted CME 55, 108 mm Internal Diameter Hollow Stem Augers</u>		COMPILED BY <u>CS</u>	
DATUM <u>Geodetic</u>		DATE <u>December 15, 2010</u>		CHECKED BY <u>LCC</u>	

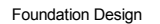
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
193.2	GROUND SURFACE													
0.9	TOPSOIL													
	SILTY CLAY with sand, trace gravel, containing rootlets, and organics Firm to stiff Brown Moist		1	SS	5									
			2	SS	12									
191.8														
1.5	CLAYEY SILT with sand to some sand, trace to some gravel (TILL) Stiff to hard Brown to grey Moist becoming wet below a depth of 2.3 m		3	SS	13									
			4	SS	25									
			5	SS	19									
			6	SS	54									
			7	SS	92									
			8	SS	49									
			9	SS	30									
			10	SS	69									
183.0														
10.2	SAND and SILT, some gravel, trace clay (TILL) Dense Brown Wet		11	SS	39									
181.5														
11.7	CLAYEY SILT, some sand and gravel (TILL) Very stiff Brown Wet		12	SS	29									
179.9														
13.3	SAND and GRAVEL, trace to some silt, trace clay Compact to very dense Brown Wet		13	SS	12									

Continued Next Page

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

MIS-MTO-001 09-1111-6036.GPJ GAL-MISS.GDT 6/20/11 CD

PROJECT 09-1111-6036				RECORD OF BOREHOLE No 10-401				2 OF 2 METRIC								
W.O. 07-20024				LOCATION N 4823614.1 ; E 276498.5				ORIGINATED BY MS								
DIST Central HWY 401				BOREHOLE TYPE Track-Mounted CME 55, 108 mm Internal Diameter Hollow Stem Augers				COMPILED BY CS								
DATUM Geodetic				DATE December 15, 2010				CHECKED BY LCC								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100					
177.3	SAND and GRAVEL, trace to some silt, trace clay Compact to very dense Brown Wet		14	SS	100/05											
15.9	SHALE (BEDROCK) Reddish-brown		15	SS	100/08											
174.9																
18.3	END OF BOREHOLE		16	SS	100/05											
NOTES: 1. Shallow piezometer installed in separate borehole 2. Water level in piezometers measured as follows: Deep piezometer Date Depth (m) Elev. (m) Dec. 15, 2010 0.3 192.9 Feb. 3, 2011 0.2 193.0 Apr. 21, 2011 0.1 a.g.s.* 193.3 *Above ground surface Shallow piezometer Date Depth (m) Elev. (m) Dec. 15, 2010 1.3 191.9 Feb. 3, 2011 1.3 191.9 Apr. 21, 2011 0.7 192.5																



1 OF 2 **METRIC**

— CHECKED BY _____ LCC

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

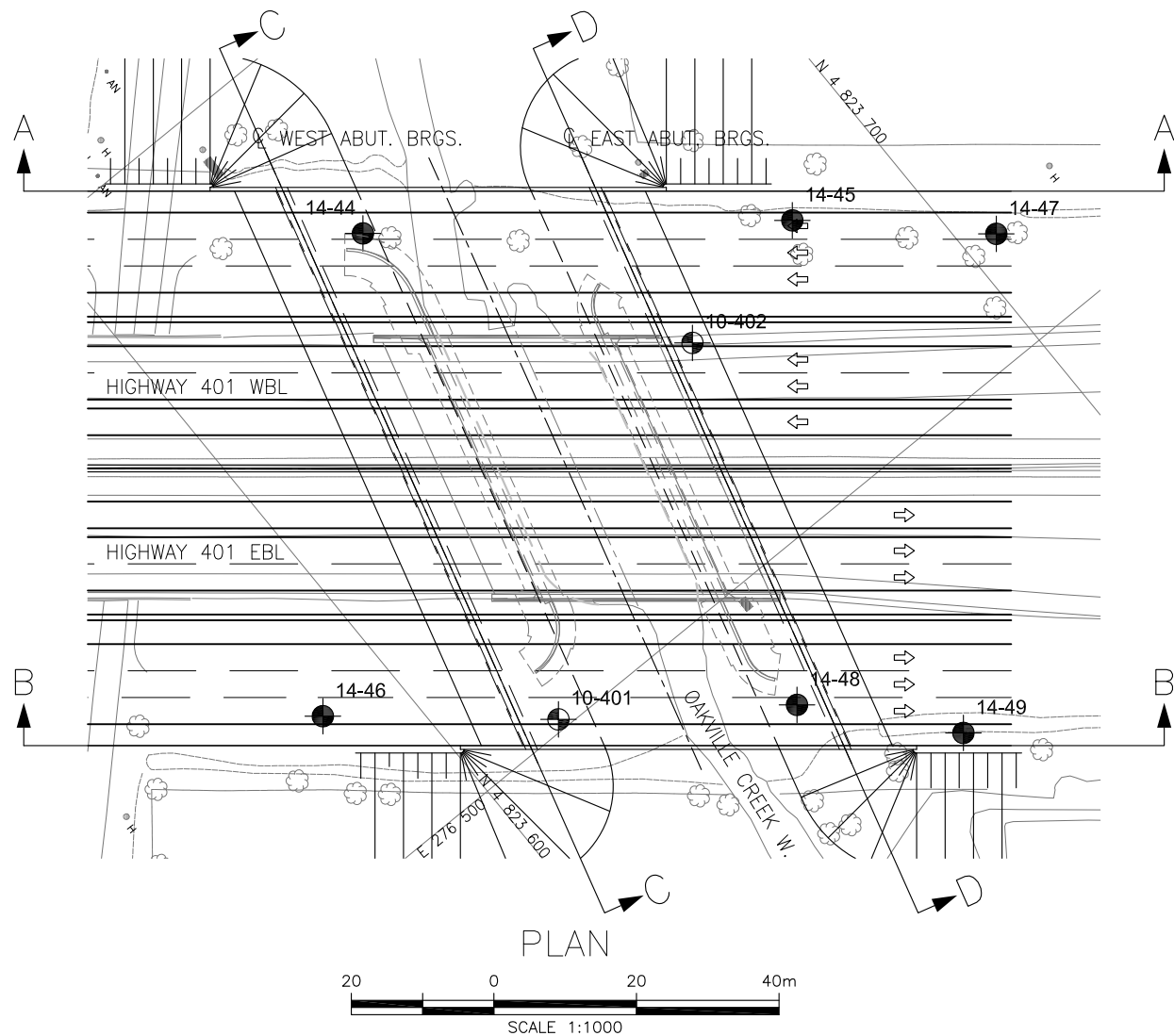
MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 6/20/11 CD

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-402		2 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4823662.1 ; E 276469.5</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Power Auger</u>		COMPILED BY <u>CS</u>	
DATUM <u>Geodetic</u>		DATE <u>November 29, 2010</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
180.5 20.7	CLAYEY SILT with sand, trace to some gravel (TILL) Stiff to hard Brown to grey Wet		13	SS	44															
			14	SS	57															
			15	SS	178															
			16	SS	100/.13															

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

Appendix D
“Borehole Locations and Soil Strata” Drawings



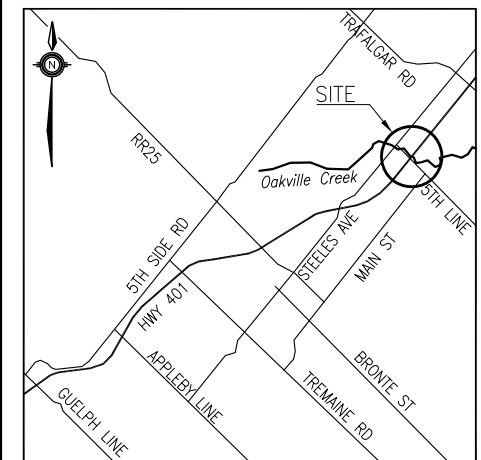
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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No

HIGHWAY 401
OAKVILLE CREEK WEST BRANCH
BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

AECOM

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

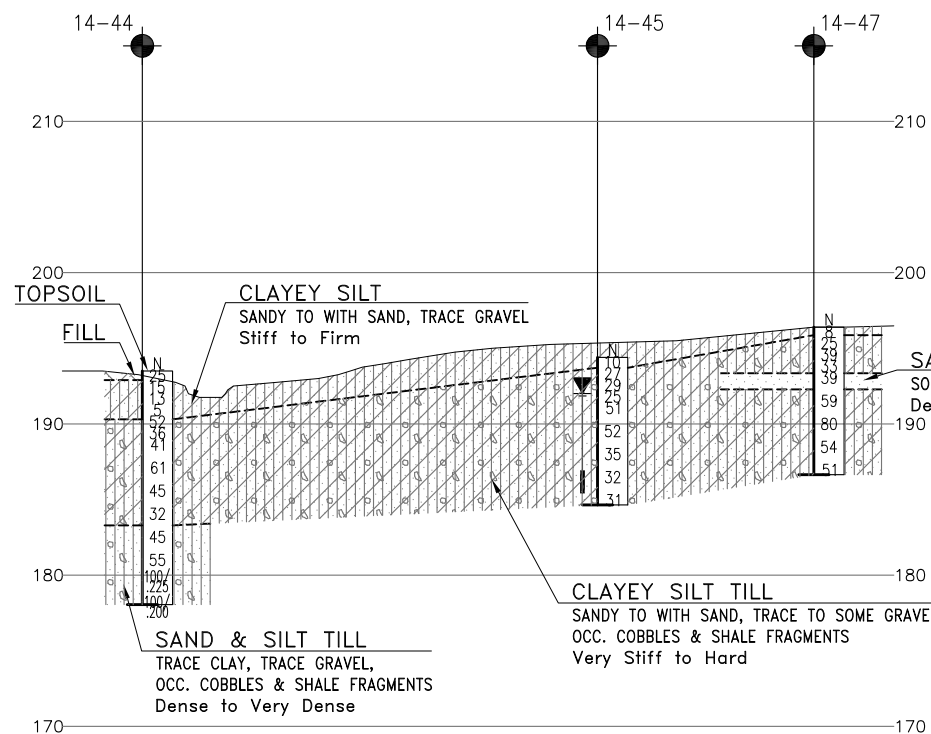
●	Borehole (Current Investigation)
⊙	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⬇	Head Artesian Water
⬆	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
14-44	193.5	4 823 636.0	276 428.4
14-45	194.4	4 823 683.8	276 465.1
14-46	194.3	4 823 588.8	276 477.2
14-47	196.4	4 823 704.8	276 484.6
14-48	194.4	4 823 641.3	276 518.1
14-49	195.6	4 823 656.9	276 535.9
10-401	193.2	4 823 614.1	276 498.5
10-402	201.2	4 823 662.1	276 469.5

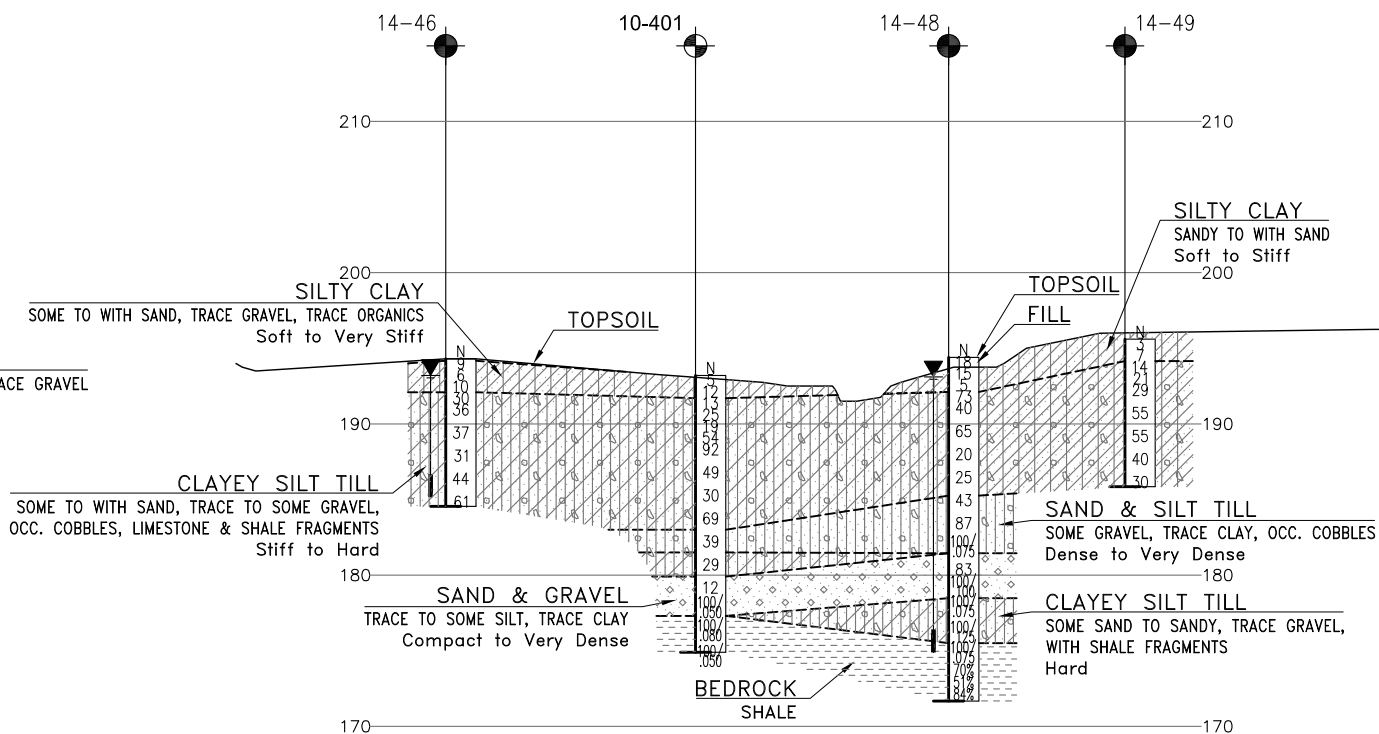
NOTES-

- 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. 30M12-374



PROFILE A-A

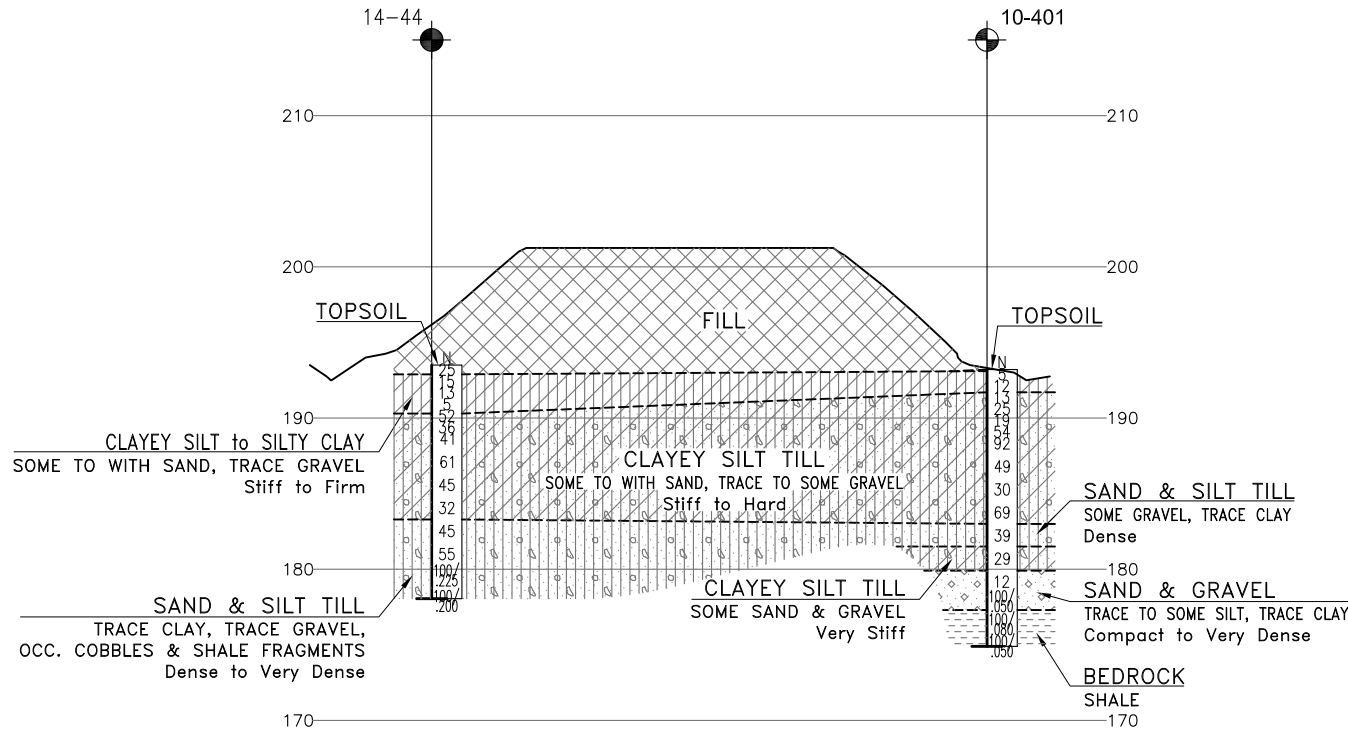


PROFILE B-B

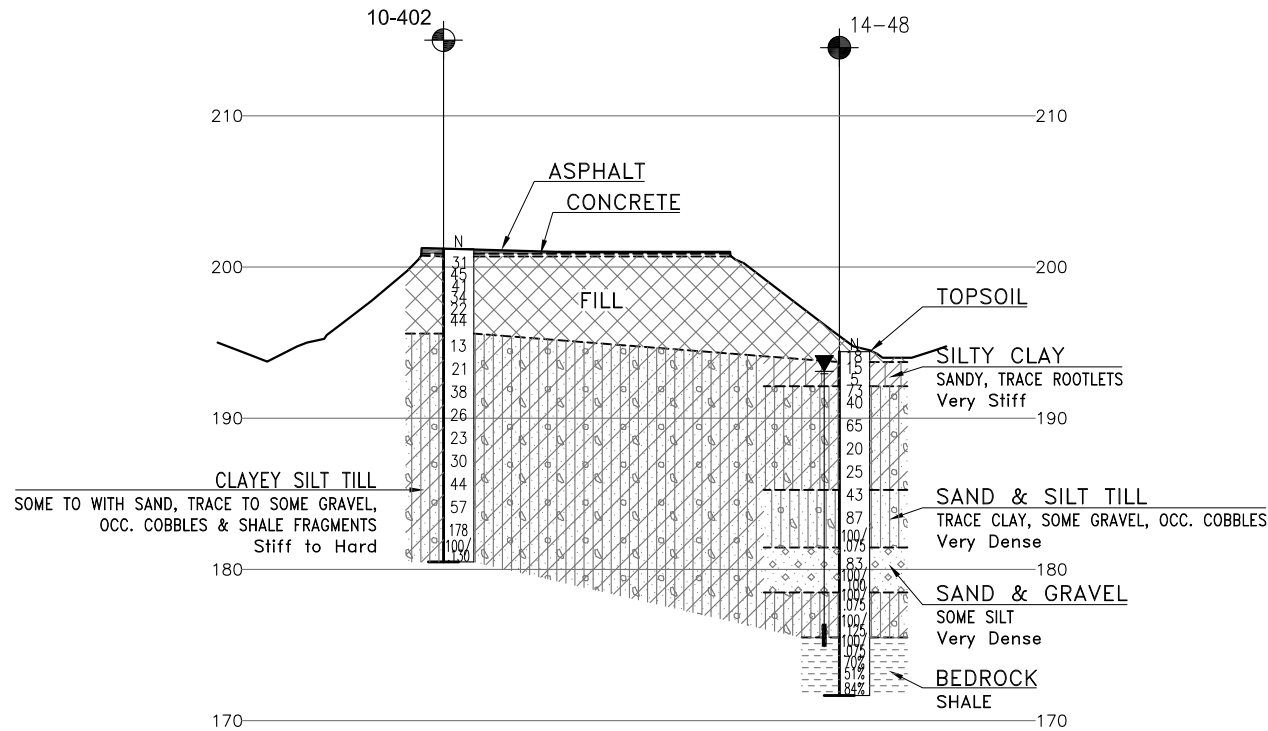
H 1:1000
V 1:500

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MEF	CHK PKC	CODE
DRAWN	MFA	CHK MEF	SITE
			LOAD
			STRUCT
			DWG 1
			DATE JUL. 2014

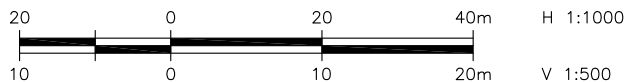
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



SECTION C-C



SECTION D-D

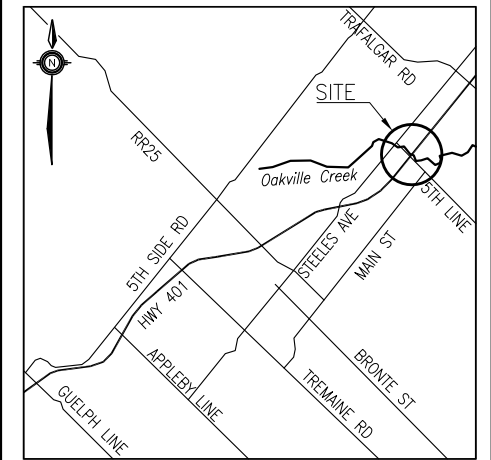


CONT No
WP No

HIGHWAY 401
OAKVILLE CREEK WEST BRANCH
BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

AECOM



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
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
14-44	193.5	4 823 636.0	276 428.4
14-45	194.4	4 823 683.8	276 465.1
14-46	194.3	4 823 588.8	276 477.2
14-47	196.4	4 823 704.8	276 484.6
14-48	194.4	4 823 641.3	276 518.1
14-49	195.6	4 823 656.9	276 535.9
10-401	193.2	4 823 614.1	276 498.5
10-402	201.2	4 823 662.1	276 469.5

-NOTES-

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

GEOCRES No. 30M12-374

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MEF	CHK PKC	CODE
DRAWN	MFA	CHK MEF	SITE
			LOAD
			DATE JUL. 2014
			STRUCT
			DWG 2

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010
- OPSD 208.010
- OPSD 3000.100

2. Suggested Text for NSSP on “Subgrade preparation and Engineered Fill Pad for RSS”

Any topsoil, soft/loose native soil or disturbed fill should be stripped from the footprint of the RSS. A minimum 500 mm thick layer of bedding material conforming to OPSS Granular “A” requirements should be provided under the RSS mass to provide a uniform subgrade condition. Engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

Appendix F
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Driven H-Piles	Caissons / Drilled Shafts
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Lower cost than deep foundations. iii. High geotechnical resistance available on very stiff to hard till. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Excavation may require temporary shoring. ii. Potential disturbance to the existing footings under service. iii. Dewatering may be required, depending on depth of excavation and surface drainage conditions. iv. Footing base potential for scour near the creek flow. <p style="text-align: center;">FEASIBLE BUT NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundations. ii. Allows use of perched abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement. ii. Potential disturbance to the existing footings under service. iii. Dewatering may be required, depending on depth of excavation and surface drainage conditions. iv. Footing base potential for scour near the creek flow. <p style="text-align: center;">NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Requires less excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Possibility that cobbles and boulders may be encountered in existing fill and native soils. iii. Individual piles within a group may encounter refusal at varying elevations. <p style="text-align: center;">RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded in bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than footings. ii. Temporary liners will be required to install caissons through cohesionless gravels/sands/silts below groundwater level. iii. Difficulty in sealing liners at base. iv. Possibility of cobbles and boulders being encountered during augering and liner installation. v. Difficulty in cleaning and inspecting bases. <p style="text-align: center;">NOT RECOMMENDED</p>