PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
QEW N-E RAMP OVER FORD DRIVE
QUEEN ELIZABETH WAY/HIGHWAY 403 IMPROVEMENTS
OAKVILLE, ONTARIO

W.O. 09-20007

Geocres Number: 30M5-296

Report to

McCormick Rankin

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

October 17, 2013
File: 19-1351-184

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted for the proposed structure which will carry traffic from southbound (SB) Ford Drive to eastbound (EB) Queen Elizabeth Way (QEW) in the Town of Oakville, Ontario. This investigation is part of the QEW/Highway 403 Improvements project, from Trafalgar Road to Winston Churchill Boulevard.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

The information collected in the course of this investigation and presented in this report is intended for preliminary design purposes only. Additional site investigation, field testing and engineering analysis may be required at the detail design phase. The extent of the additional investigation will depend on the final location and General Arrangement (GA) of the structure.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin (MRC), under the Ministry of Transportation Ontario (MTO) Work Order Number 09-20007.

A previous foundation investigation report was completed in 1977 for the existing ramp structure located approximately 5 m north of the proposed structure. The title of the reports is as follows:

> Foundation Investigation Report For W-N Ramp HWY 403 Structure over Ford Drive, QEW/Ford Drive/403 Interchange, District 4 (Hamilton), W.P. 125-66-16, (Geocres 30M5-106), Site 10-287, dated May 1977

The Record of Borehole sheets for two of the boreholes (BH 1 and 2) drilled during the previous investigation are included in Appendix A, for reference purposes.

2 SITE DESCRIPTION

The proposed structure site is located at Ford Drive and the QEW in Oakville, Ontario. The proposed structure will be located approximately 5 m south of the existing ramp structure at Ford Drive that carries the QEW – Highway 403 W-N ramp over Ford Drive. In general, the lands in the vicinity of the site slope gently to the south (construction west) towards Joshua Creek, which is located approximately 150 m to the south. The lands immediately adjacent to the site consist of undeveloped areas of the highway right-of-way. To the east, there is a residential area and to the west and south of the QEW, lies the Ford Motors Canada Complex.

The site lies within the South Slope physiographic region, characterized by glacially deposited overburden overlying shale bedrock of the Queenston and Dundas Formations of the upper Ordovician age.

Photographs included in Appendix D show the site of the proposed structure.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on May 22 and June 4, 2013. Two boreholes, identified as 13-21 and 13-22, were drilled and sampled at this site. Borehole 13-21 was drilled near the proposed east abutment while Borehole 13-22 was drilled near the proposed west abutment. The borehole depths ranged from 4.9 m to 7.6 m. The Record of Borehole sheets are included in Appendix A.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing included in Appendix E. The coordinates and elevations of the boreholes are given on the drawing and on the individual Record of Borehole sheets.

The borehole locations were marked in the field and utility clearances were obtained prior to commencement of drilling operations. A Region of Halton Road Cut Permit was obtained for drilling Borehole 13-21 on Ford Drive and for unloading and loading the drill rig on Ford Drive for accessing Borehole 13-22.

Borehole 13-21 was drilled using a CME 75 truck-mounted drill and Borehole 13-22 was drilled using a CME 55 track-mounted drill rig. A combination of solid-stem auger drilling techniques and NQ coring methods were used to advance the boreholes. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The recovered soil and bedrock samples were logged in the field and processed for transport to Thurber's laboratory in Oakville, Ontario for further examination and testing.

Groundwater conditions were observed in the open boreholes prior to the start of the coring operations. A standpipe piezometer, consisting of 19mm diameter PVC pipe with slotted screen, was installed in Borehole 13-22. The installation details of the piezometer are summarized in Table 3-1 along with the borehole completion details for the borehole with no piezometer installation.

Table 3-1. Borehole Completion and Piezometer Installation Details

Borehole	Tip Position		Borehole Completion and Piezometer Installation Details
	Depth (m)	Elev. (m)	
BH13-21	None installed		Borehole backfilled with bentonite holeplug to 0.3 m, then concrete from 0.3 to 0.15 m, and asphalt patch to surface.
BH13-22	7.6	120.4	Sand filter from 7.6 m to 5.8 m and bentonite holeplug from 5.8 m to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determinations. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were conducted on selected portions of the rock cores. The UCS values of the rock were assessed from the point load data and these values are reported on the borehole logs (as average UCS per run).

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A and the Borehole Locations and Soil Strata Drawing included in Appendix E. An overall description of the stratigraphy based on the conditions encountered in the boreholes is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

The stratigraphy encountered at this site generally consists of a thin layer of topsoil or asphalt overlying sand to sand and gravel fill, which is underlain by silty clay (at the west abutment) and shale bedrock. More detailed descriptions of the individual strata encountered at the proposed structure site are presented below.

5.1 Topsoil

A thin layer of topsoil (125 mm thick) was encountered at the surface in Borehole 13-22. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.2 Asphalt

A layer of asphalt (150 mm) was encountered at the surface of Borehole 13-21 drilled on Ford Drive in the northbound (NB) lanes.

5.3 Sand Fill

Sand fill was encountered below the topsoil in Borehole 13-22 while sand and gravel fill was encountered below the asphalt in Borehole 13-21. The sand to sand and gravel fill was brown in colour and contained some silt.

The sand fill in Borehole 13-22 was 0.7 m thick, with the lower boundary encountered at a depth of 0.8 m (Elev. 127.3 m). The sand and gravel fill in Borehole 13-21 was 0.6 m thick, with the lower boundary at a depth of 0.8 m (Elev. 124.3 m).

SPT N-values recorded in the fill ranged from 13 to 38 blows for 300 mm of penetration, indicating a compact to dense relative density. In general, the fill below the asphalt was dense while the fill below the topsoil was compact. The moisture content of samples of the fill ranged from 4 to 7%.

Laboratory grain size distribution analysis was carried out on one sample of the granular fill. The results of this test are presented on the corresponding Record of Borehole sheet included in Appendix A and the grain size distribution curve is presented in Figure B1 of Appendix B.. The results are summarized below:

Gravel %	57
Sand %	33
Silt and Clay %	10

5.4 Silty Clay

Reddish brown silty clay containing trace sand and trace gravel was encountered below the sand fill in Borehole 13-22. The silty clay was 1.6 m thick and the lower boundary of this layer was encountered at a depth of 2.4 m (Elev. 125.7 m).

SPT N-values of 12 and 20 blows for 300 mm of penetration were recorded in the silty clay, indicating a stiff to very stiff consistency. Moisture contents ranged from 9 to 19%.

Laboratory grain size distribution analysis was performed on one sample of the silty clay. The results of this test are presented on the corresponding Record of Borehole sheet in

Appendix A and the grain size distribution curve is plotted on Figure B2 of Appendix B. The results are summarized below:

Gravel %	2
Sand %	11
Silt %	53
Clay %	34

5.5 Shale Bedrock

Shale bedrock was encountered below the silty clay in Borehole 13-22 and below the sand and gravel fill in Borehole 13-21. The depths and elevations at which bedrock was encountered at the borehole locations are summarized in Table 5.1.

Table 5.1 – Depths and Elevations of Bedrock Surface

Foundation Element	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
East Abutment	BH13-21	0.8	124.3
	2(1)	2.3	128.6
West Abutment	BH13-22	2.4	125.6
	1(1)	2.4	127.2

Note: (1) Geocres 30M5-106, Site 10-287

The bedrock was described as thinly bedded grey shale with frequent hard limestone interbeds up to approximately 0.5 m thick. The bedrock was generally described as weathered at the soil-bedrock interface and described as slightly weathered to fresh within 1 to 1.5 m of the soil-bedrock interface. Frequent horizontal fractures, occasional vertical fractures, broken zones, and clay seams were observed in the bedrock cores.

Total Core Recovery (TCR) in the bedrock was 100%. The Rock Quality Designation (RQD) values ranged from 52 to 100%, indicating a fair to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 5.

The average estimated unconfined compression strength (UCS) of the shale with hard limestone interbeds, interpreted from point load tests conducted on intact cores, ranged from 33 to 73 MPa, indicating a medium strong to strong rock strength classification.

5.6 Groundwater Levels

Water levels were observed in the open boreholes prior to the start of the coring operations. A standpipe piezometer was installed in Borehole 13-22 within the bedrock. The water levels measured in the open boreholes and piezometer are as follows:

Table 5.3 – Groundwater Depths and Elevations

Borehole	Date of Reading	Water Level		Comment
		Depth (m)	Elevation (m)	
BH13-21	May 22, 2013	Dry	N/A	Prior to coring
BH13-22	June 4, 2013	Dry	N/A	Prior to coring
	June 7, 2013	5.4	122.6	Piezometer
	June 26, 2013	6.0	122.0	Piezometer

It should be noted that the recorded groundwater levels are short term and are susceptible to seasonal fluctuations. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of significant and/or prolonged precipitation.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors from MMM Group provided co-ordinates and the ground surface elevations at the boreholes drilled.

DBW Drilling Ltd. from Ajax, Ontario supplied both truck and track mounted CME 55 drill rigs and conducted the drilling, sampling and in-situ testing operations.

Overall planning and supervision of the field program was conducted by Ms. Lindsey Blaine, P.Eng. The field investigation was supervised by Mr. George Azzopardi of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Interpretation of the data and preparation of the report were carried out by Ms. Lindsey Blaine, P.Eng. and Mr. Alastair Gorman, P.Eng.. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Lindsey Blaine, P.Eng.
Geological Engineer

Alastair Gorman, P.Eng.
Senior Foundations Engineer

A. E. GORMAN
OCT 17/13

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 17/13
PROVINCE OF ONTARIO

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

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

Our understanding of the project, based on the GA, consists of:

- The proposed structure will carry traffic on southbound Ford Drive to eastbound Queen Elizabeth Way (QEW)
- the proposed overpass structure will comprise of a single 40.0 m deck span, flanked by RSS Walls and carry a single lane of traffic
- the proposed pavement elevation of QEW at the west and east abutment will be 130.2 and 131.2 m, respectively
- Ford Drive will be at approximate elevation 123.4 m
- An integral abutment design is preferred

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

8 STRUCTURE FOUNDATIONS

The stratigraphy identified in the preliminary investigation consisted of a thin layer of topsoil or asphalt overlying sand to sand and gravel fill, which is underlain by silty clay and shale bedrock. The short term groundwater level measured in the piezometers was at Elev. 122.0 m. This elevation may be higher than the true groundwater level due to drilling water not having fully dissipated into the low permeable bedrock.

In the preparation of the preliminary foundation recommendations, consideration was given to the following foundation types:

- Spread footings bearing on shale bedrock
- Spread footings bearing on engineered fill
- Steel H-piles socketed into bedrock
- Augered caissons socketed into bedrock

Spread footings on native soil were not considered since shallow bedrock was encountered in both boreholes. A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix C.

8.1 Spread Footings on Bedrock

Ford Drive is constructed in a cut at the proposed structure location. Due to the shallow depth of overburden, spread footings on shale bedrock are considered feasible to support the structural loads.

As interpreted from the boreholes, spread footings should be founded on undisturbed shale bedrock at or below elevation 125.2 and 123.8 m at the west and east abutments, respectively. The elevations presented are the highest recommended founding elevation and must be reviewed during the detail design based on the final bridge arrangement and results of the site investigation and field testing to be completed at that time.

For preliminary design, footings founded on undisturbed shale should be designed using a factored geotechnical resistance at ULS of 1,000 kPa. This value includes a resistance factor of 0.5 as per Table 6.1 of the CHBDC. The SLS condition will not govern design of footings founded on bedrock.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be adjusted as shown in the CHBDC (2006) Clause 6.7.3 and 6.7.4. During detail design, the geotechnical resistance must also be reviewed taking account of the position of the footing relative to the forward slope.

8.2 Spread Footings on Engineered Fill

If higher founding elevations are required, than those provided in Section 8.1, spread footings could be constructed on an engineered fill pad consisting of Granular “A” material. This option would be suitable for abutment footings which may be perched within the approach embankment and above the existing bedrock surface elevation.

For preliminary design, footings founded on engineered fill should be designed using a factored resistance at ULS of 900 kPa and a SLS of 350 kPa.

The engineered fill must bear on undisturbed shale at or below elevations provided in Section 8.1. The Granular "A" pad must be placed in 150 mm lifts and compacted to 100% standard proctor maximum dry density (SPMDD) at optimum moisture content ±2%. The geometry of the fill pad must conform to the general requirements shown in Figure 1.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be adjusted as shown in the CHBDC (2006) Clause 6.7.3 and 6.7.4. During detail design, the geotechnical resistance must also be reviewed taking account of the position of the footing relative to the forward slope.

8.3 Steel H-Piles Socketed into Bedrock

Since bedrock is shallow at this site, driven H-piles would typically not be considered cost effective or practical from a foundation point of view. However, piles socketed into the bedrock could be used to provide axial geotechnical resistance and to accommodate the design of an integral abutment, if required.

In the case of an integral abutment, excavation of bedrock will be required within the abutment footprint and special considerations must be given to the details of the pile installation in order to provide the required flexibility in the upper 3.0 m length. Preliminary recommendations are provided but must be reviewed during detail design based on the final alignment, final bridge arrangement and the results of the site investigation and field testing to be completed at that time.

For HP 310x110 steel H-piles placed in rock sockets, a factored axial geotechnical resistance at ULS of 2,000 kN is recommended. This value includes a geotechnical resistance factor of 0.4 as per the CHBDC. The SLS condition will not govern for piles socketed into bedrock.

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

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

8.4 Augered Caissons Socketed into Bedrock

Drilled shaft foundations socketed into shale bedrock are not considered appropriate for this site and have not been developed further.

8.5 Abutment Design Considerations

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional or semi-integral abutment design, principally due to shallow depth to bedrock.

However, if other design and/or maintenance issues favour the use of integral abutment design, this can be accommodated through excavation of shale bedrock within the abutment area to accommodate the use of steel H-pile foundations.

8.6 Frost Cover

The design depth of frost penetration at this site is 1.2 m. It is recommended that all footings be provided with a minimum of 1.2 m of earth cover above the underside of the pile cap or footing. Frost protection is also required for footings founded on shale bedrock.

8.7 Recommended Foundation

From a geotechnical perspective, and based on current information, the recommended foundation consists of spread footings bearing on undisturbed shale bedrock.

9 DEWATERING

Excavations for spread footings at the elevations given in Section 8.1 are not expected to penetrate below the groundwater level. However, if deeper excavations are required they may penetrate below the groundwater level and some seepage into the excavation may occur. However, due to the relatively low permeability of the shale, the volumes are expected to be small. Similarly, minor seepage from the fill may be encountered and surface water flow may enter the excavations.

Given the small volumes of water that are expected, it is considered that pumping from sumps will be adequate for dewatering excavations at this site. The exposed shale at the base of the foundation excavation must be protected from deterioration within 24 hours of completion of the excavation.

In the case of sockets drilled in the bedrock for deep foundations, pumping accumulated water from the socket prior to concreting will be required in conjunction with cleaning all loosened material from the socket.

10 BRIDGE APPROACHES AND EMBANKMENTS

Based on the current and previous boreholes drilled at this site, the approach embankments will be constructed over foundation soils consisting of stiff native silty clay and shale bedrock. The foundation soils are considered to provide adequate stability for approach embankments if constructed at a side slope of 2H:1V or RSS wall using SSM or granular fill.

Constructing the approach embankments with cohesive fill may be possible but will be dependent on the mechanical properties of the material. An embankment constructed of cohesive material will typically not perform as well as an embankment constructed using SSM or granular fill and will require flatter side slopes which will extend the footprint of the embankment.

Preliminary analysis indicates that settlement of the foundation soils under the imposed embankment loading is expected to be less than 25 mm. Considering the competency of the foundation soils the settlement will be essentially completed when construction of the fill is completed.

Further settlement analysis and the global, internal and surficial stability of the approach embankment fills should be further evaluated during the detail design phase. Additionally, permanent drainage and slope protection requirements must be addressed during the detail design.

11 ROADWAY PROTECTION

Excavation support systems may be required for temporary roadway protection during foundation construction where stable slopes cannot be maintained. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. In general, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. The feasibility of installing protection systems should be assessed once further subsurface investigation is carried out during detail design.

12 CONSTRUCTION CONCERNS

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

- The shale bedrock exposed in foundations must be concreted within 24 hours after the bedrock surface has been properly prepared and is free of loose debris to prevent softening and deterioration.
- Excavations must not undermine the footings of the existing QEW-Ford Drive overpass.

13 INVESTIGATION FOR DETAIL DESIGN

During the detail design phase of the project, additional site investigation and field testing may be required. The scope and results of this investigation must be reviewed at that time based on the final GA to determine if they meet the current Ministry requirements and if additional investigation and analysis is necessary.

14 CLOSURE

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

Thurber Engineering Ltd.

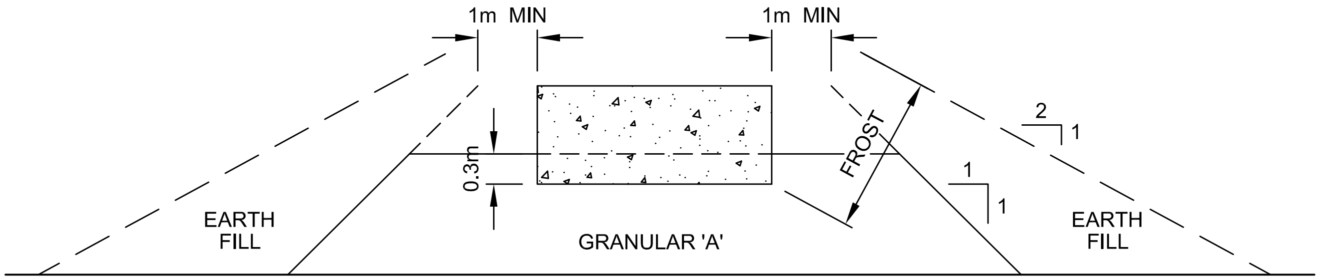
Report prepared by:
Stephen Peters, P.Eng.
Project Engineer

Alastair Gorman, P.Eng.
Senior Foundations Engineer

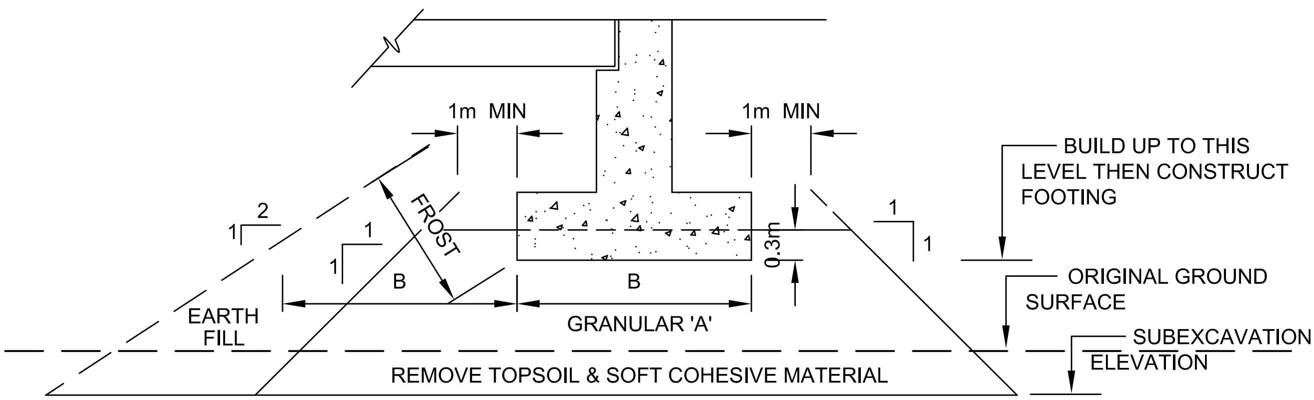
REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
OCT 17/13
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 17/13
PROVINCE OF ONTARIO



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 : SBP	DRAWN : MFA	APPROVED : AEG
DATE : OCTOBER 2013	SCALE : N.T.S.	DRAWING No. FIGURE 1

Appendix A

Record of Borehole Sheets

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

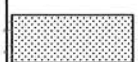
DISCONTINUITY SPACING

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

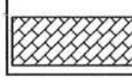
SYMBOLS

 CLAYSTONE

 SILTSTONE

 SANDSTONE

 COAL

 BEDROCK

STRENGTH CLASSIFICATION

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

TERMS

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

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

TEXTURAL CLASSIFICATION OF SOILS

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

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

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	(POCKET PEN)	APPROX. SPT[1] "N" VALUE
Very Soft	< 10		< 2
Soft	10 to 25		2 to 4
Firm	25 to 50	(0.5-1)	4 to 8
Stiff	50 to 100	(1-2)	8 to 15
Very Stiff	100 to 200	(2-4)	15 to 30
Hard	> 200	(>4)	> 30

(1) Standard Penetration Test – the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m

TERMS DESCRIBING DENSITY(COHESIONLESS SOILS)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

HIERARCHY OF SOIL STRENGTH PREDICTION

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT Value
5) Pocket Penetrometer

LEGEND FOR TEST HOLE LOGS

■ Shelby Tube   A – Casing   ⊠ SPT   ▥ Grab/Auger sample   ◫ Core   ☒ No Recovery

• MC – Moisture Content (% by Weight) as determined by sample

▼	Water Level
C_{vane}	Shear Strength Determination by Field Insitu Vane
C_{pen}	Shear Strength Determination by Pocket Penetrometer
C_{lab}	Shear Strength Determination using a Laboratory Vane Apparatus
C_U	Undrained Shear Strength determined by Unconfined Compression Test
AS/GS/BS	Auger Sample/Grab Sample/ Block Sample
SS	Split-spoon
SC	Soil core
AED	Oedometer test
TXL	Triaxial test

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-21

METRIC

W.P. ______ LOCATION N 4 817 230.8 E 290 835.9 ORIGINATED BY GA

HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2013.05.22 - 2013.05.22 CHECKED BY LRB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³ / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
125.1									
0.0	**ASPHALT:** (150mm)					125			
0.2	**SAND** and **GRAVEL**, some silt Dense Brown Damp (FILL)	1	SS	38			o		57 33 10 (SI+CL)
124.3 / 0.8	**SHALE** with limestone interbeds, highly weathered, thinly bedded	2	SS	71		124	o		
	Start coring from 1.8m	3	SS	50/ 0.075			o	FI	
	Slightly weathered, thinly bedded, grey, occasional limestone interbeds Limestone interbed (25mm) at 2.4m 200mm at 2.2m Horizontal fractures at 1.8m, 1.9m, 2.0m, 2.1m, 2.2m, 2.5m, 2.9m, 3.0m 150mm highly broken zone at 3.2m	1	RUN		▽	123 / 122		>5, 1, 2, 2, >5	RUN #1 TCR=100% SCR=83% RQD=52% UCS=33MPa (Average)
120.2	Clay seam at 3.2m Horizontal fractures at 3.4m, 3.5m, 3.6m, 3.7m, 3.9m, 4.1m, 4.3m, 4.7m, 4.8m Limestone interbeds (25mm) at 3.8m, 4.2m, 4.4m, 4.6m and (75mm) at 4.0m	2	RUN			121		4, 3, 2, 1, 3	RUN #2 TCR=100% SCR=93% RQD=70% UCS=56MPa (Average)
4.9	END OF BOREHOLE AT 4.9m. BOREHOLE OPEN TO 4.9m AND WATER LEVEL AT 2.1m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, CONCRETE TO 0.15m, THEN ASPHALT PATCH TO SURFACE.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-22

METRIC

W.P.		ORIGINATED BY	GA
HWY	403/QEW	COMPILED BY	AN
DATUM	Geodetic	CHECKED BY	LRB
LOCATION	N 4 817 183.5 E 290 838.7		
BOREHOLE TYPE	Solid Stem Augers/NQ Coring		
DATE	2013.06.03 - 2013.06.04		

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

ELEV / DEPTH	DESCRIPTION
128.0	
0.0	**TOPSOIL:** (125mm)
0.1	**SAND**, some silt, some gravel Compact Brown Damp (FILL)
127.3 / 0.8	Silty **CLAY**, trace sand, trace gravel Stiff Reddish Brown
125.7 / 2.4	**SHALE** with limestone interbeds, highly weathered, thinly bedded, grey, iron oxide staining Start coring at 4.5m Slightly weathered to fresh, thinly bedded, grey, occasional limestone interbeds Clay seam (50mm) at 4.5m Limestone interbeds (25mm thick) at 5.1m, 5.4m, 5.6m, 6.0m and (100mm) at 5.2m Horizontal fractures at 4.8m, 4.9m, 5.4m, 5.5m, 5.7m, 5.9m Limestone interbeds (25 to 50mm thick) at 6.1m, 6.2m, 6.4m, 6.6m, 6.8m, 7.2m and (300mm) at 7.3m Horizontal fractures at 6.2m, 6.7m, 6.9m, 7.0m, 7.1m
120.4 / 7.6	END OF BOREHOLE AT 7.6m. BOREHOLE OPEN TO 7.6m AND WATER LEVEL AT 5.4m UPON COMPLETION OF CORING. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
1	SS	13	
2	SS	12	GR 2, SA 11, SI 53, CL 34
3	SS	20	
4	SS	50/ 0.150	
5	SS	50/ 0.125	
1	RUN		RUN #1 TCR=100% SCR=83% RQD=75% UCS=49MPa (Average)
2	RUN		RUN #2 TCR=100% SCR=100% RQD=100% UCS=73MPa (Average)

FI: >5, 2, 1, 3, 1, 1, 0, 2, 2, 0

Elevation scale: 128, 127, 126, 125, 124, 123, 122, 121

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
June 7/13	5.4	122.6
June 26/13	5.2	122.8

ONTMT4S 1184.GPJ 2012TEMPLATE(MTO).GDT 11/10/13

+ ³, × ³: Numbers refer to Sensitivity

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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

WP 125-66-16 | LOCATION Co-ords N 15 803 724; E 954 067 | ORIGINATED BY CTJ

DIST 4 HWY 403 | BORING DATE March 23, 1977 | COMPILED BY CTJ

DATUM Geodetic | BOREHOLE TYPE Solid Stem Auger, BXL Core | CHECKED BY RS

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W — WATER CONTENT % 10 20 30	UNIT WEIGHT γ	REMARKS % GR SA SI CL
425.1	Ground Level									
0.0	Clayey silt, some sand, trace of gravel Very Stiff Reworked									
			1	SS	30					
			2	SS	16	420				7 22 51 20
417.2 / 7.9	(Weathered)		3	SS	31					
415.1 / 10.0	(Sound)		4	BXL	80% REC					
	Shale Bedrock (See Below)*		5	BXL	90% REC	410				RQD 9%
			6	BXL	100% REC	400				RQD 73%
393.8 / 31.3	End of Borehole					390				

*Intermittent shale, shaley limestone & limestone, fine texture, soft to hard bedding is thin and horizontal, light grey color, shale is fissile with Limestone (hard, fine texture, light grey, fossiliferous, horizontal bedding) seams from
22'0" to 22'8"
27'0" to 28'0"

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 2

WP 125-66-16 | LOCATION Co-ords. N 15 803 823; E 954 078 | ORIGINATED BY CTJ

DIST 4 HWY 403 | BORING DATE March 21, 1977 | COMPILED BY CTJ

DATUM Geodetic | BOREHOLE TYPE Solid Stem Auger, BXL Core | CHECKED BY RS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	REMARKS
429.4	Ground Level					
0.0	Clayey silt, some sand, trace of gravel occ. organic inc. Firm Reworked	1	SS	7		
		2	SS	44		
421.9 / 7.5	Hard	3	SS	79	420	
418.9 / 10.5	(Weathered)	4	SS	155/8"		
	(Sound)	5	BXL	63% REC		RQD 0%
	Shale Bedrock (See Below)*	6	BXL	98% REC	410	RQD 16%
		7	BXL	92% REC		RQD 57%
399.4 / 30.0	End of Borehole				400	
					390	

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30; UNIT WEIGHT γ; % GR SA SI C

*Intermittent shale, shaley limestone & limestone, soft to hard, fine texture, light grey colour, shale is fissile, thin bedding - horizontal with Limestone, (med. hard to hard, fine texture light grey colour, fossiliferous) seams from
13'8" to 14'4"
20'10" to 21'7"
22' 8" to 24'5"
25' 1" to 26'4"

20 15 ◇ 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Appendix B

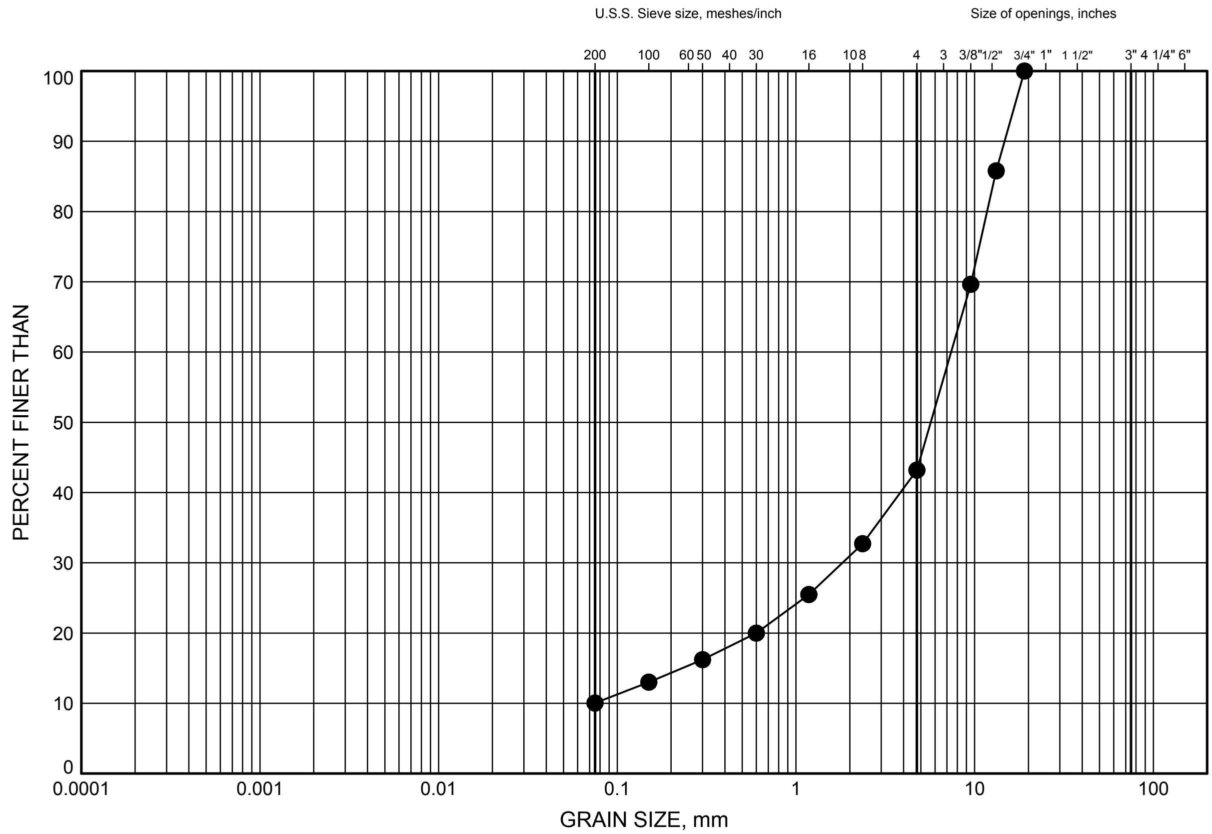
Laboratory Test Results

QEW and Hwy 403

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND and GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-21	0.38	124.73

Date August 2013

W.P.

THURBER

Prep'd SBP

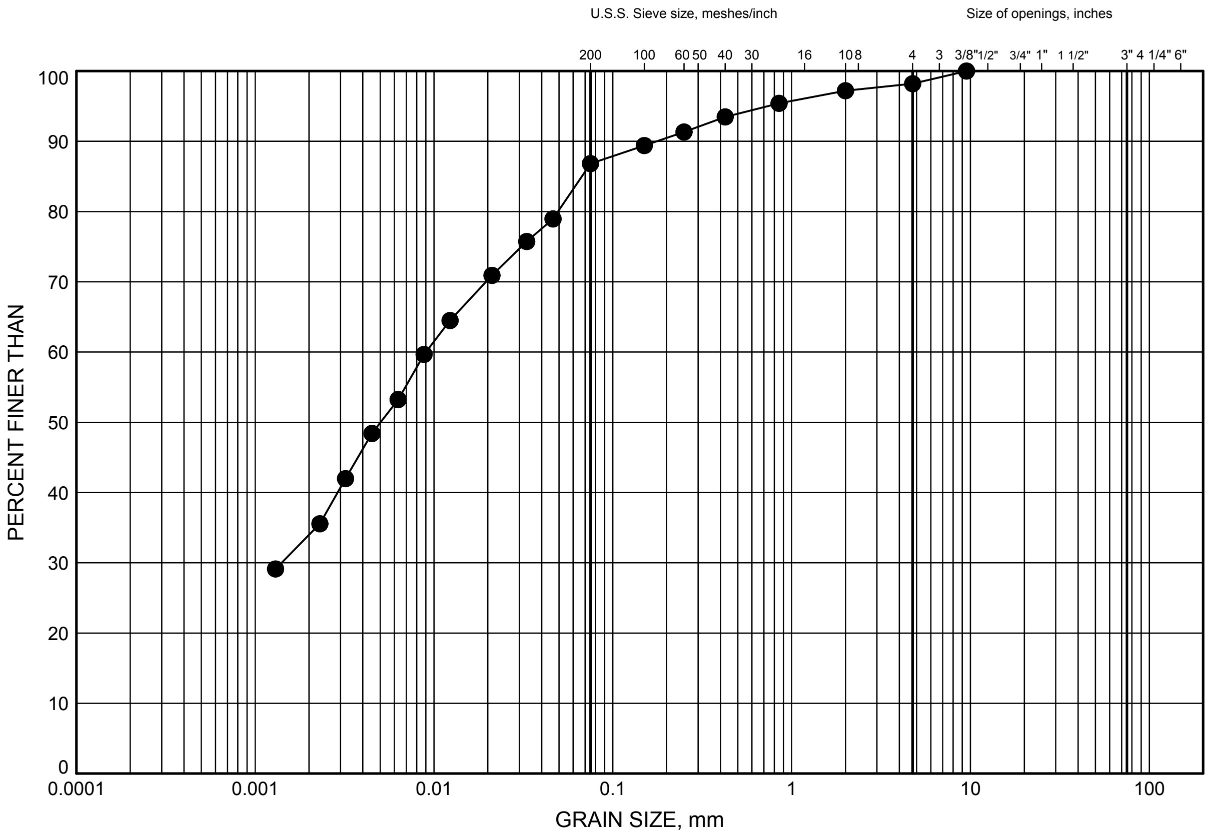
Chkd.

QEW and Hwy 403

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-22	1.07	126.97

Date August 2013

W.P.

THURBER

Prep'd SBP

Chkd.

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Spread Footing on Shale Bedrock	Spread Footing on Engineered Fill	Steel H-Piles Socketed into Shale Bedrock
Advantages: i. Generally less costly construction than deep foundation elements. ***Disadvantages:*** i. Dewatering may be required, depending on depth of excavation ii. Ineffective for resistance to uplift or overturning.	***Advantages:*** i. Economical to install ii. Accommodates perched abutment ***Disadvantages:*** i. Dewatering may be required, depending on depth of excavation. ii. Lower geotechnical resistance than spread footings on bedrock iii. Ineffective for resistance to uplift or overturning.	***Advantages:*** i. High geotechnical resistance available by socketing piles into bedrock. ii. Provide uplift and overturning resistance iii. Installation less influenced by weather and groundwater than spread footings. iv. Permits integral abutment design v. Comparatively short abutment possible ***Disadvantages:*** i. Higher unit cost compared to spread footings ii. Difficulty in unwatering, cleaning and inspecting bases iii. Pre-drilling required for installation of socketed piles. iv. Potential for difficulty in drilling through hard limestone interbeds
RECOMMENDED	**FEASIBLE**	**FEASIBLE**

Appendix D

Site Photographs

Photograph 1: Looking south along Ford Drive. The new structure is to be located south of the existing structure shown here.

Appendix E

Borehole Locations and Soil Strata Drawing

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

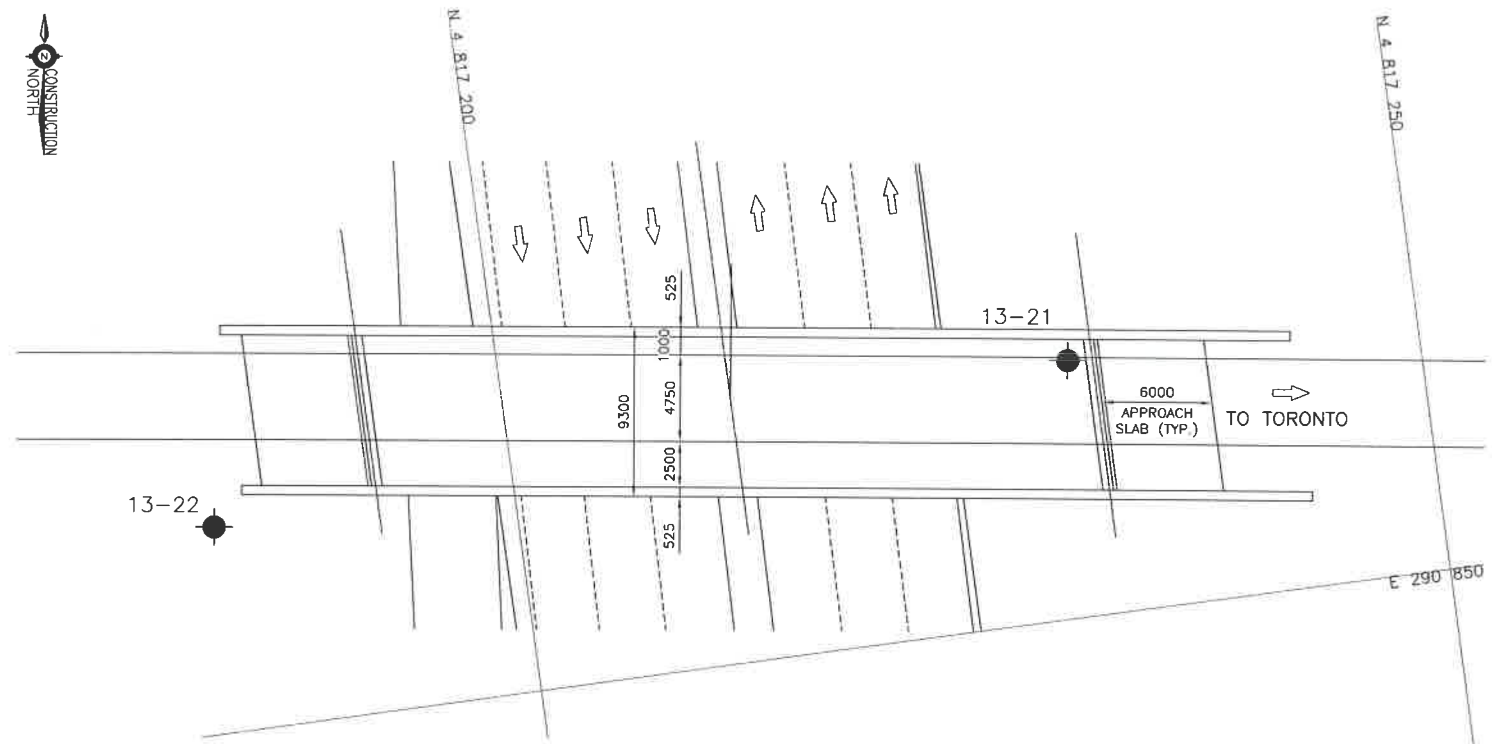
WP No

SHEET

Q.E.W.
N–E RAMP
OVER FORD DRIVE
BOREHOLE LOCATIONS AND SOIL STRATA

McCORMICK RANKIN
A member of MMM GROUP

THURBER ENGINEERING LTD.

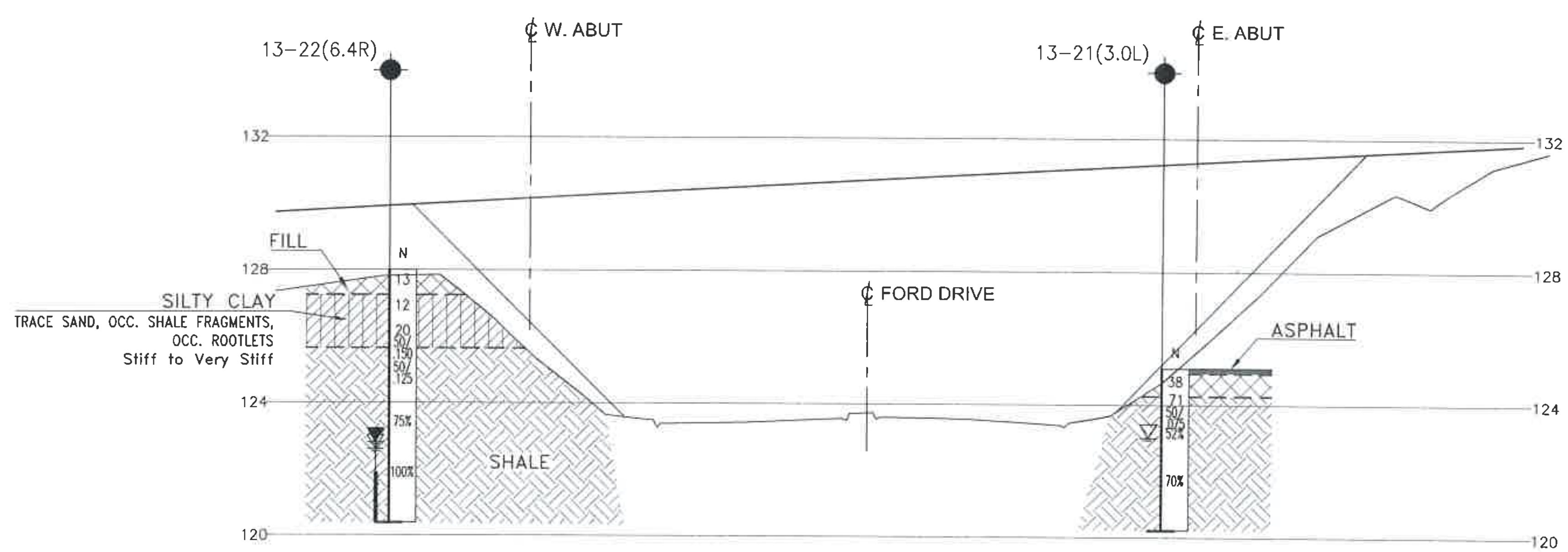


PLAN

SCALE 1:400

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 17/13
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
Oct 17/13
PROVINCE OF ONTARIO



PROFILE ALONG ℄ OF N–E RAMP

H 1:400

V 1:200

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
13–21	125.1	4 817 230.8	290 835.9
13–22	128.0	4 817 183.5	290 838.7

-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. 30M5-296

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS

DATE	BY	DESCRIPTION

DESIGN SBP	CHK AEG	CODE	LOAD	DATE AUG 2013	
DRAWN MFA	CHK SBP	SITE	STRUCT	DWG 1	

FILENAME: H:\Drafting\19\1351\184\ted1184-Plan&Profile-325(N-E Ramp over Ford Dr).dwg
PLOTDATE: 10/11/2013 5:21 PM

MINISTRY OF TRANSPORTATION, ONTARIO