PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HWY 403/Q.E.W E-N RAMP
QUEEN ELIZABETH WAY/HIGHWAY 403 IMPROVEMENTS
OAKVILLE, ONTARIO

W.0. 09-20007

Geocres Number: 30M5-292

Report to

McCormick Rankin

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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 connect westbound (WB) Queen Elizabeth Way (QEW) to eastbound (EB) Highway 403 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, a 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 detailed 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, under the Ministry of Transportation Ontario (MTO) Work Order Number 09-20007.

2 SITE DESCRIPTION

The proposed structure is to be located approximately 270 m east of Highway 403 (25 m east of the on ramp from Upper Middle Road to EB Highway 403) and will carry traffic from WB QEW to EB Highway 403. The east abutment, Pier #1, and Pier #2 will be located between Upper Middle Road and the WB QEW off ramp to Ford Drive. Pier #3 and the west abutment will be located west of Upper Middle Road.

The lands immediately adjacent to the proposed structure site consist primarily of undeveloped lands within the MTO right-of-way. Some commercial buildings are located to the north of the west abutment. The land immediately adjacent to the site has a gently rolling topography sloping down to the south.

Visual inspection of the site indicates that the north approach fill may have been partially constructed under a past contract.

The site lies on the southern fringe of the South Slope physiographic region, characterized by glacially deposited overburden overlying shale bedrock of the Queenston and Dundas Formations of the upper Ordivician 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 between May 8 and 28, 2013. A total of five boreholes were drilled and sampled at this site, identified as 13-03 to 13-07. Borehole 13-03 was drilled next the proposed west abutment and Borehole 13-07 was drilled near the proposed east abutment. Boreholes 13-04 to 13-06 were drilled for Pier #3, Pier #2, and Pier #1, respectively. The borehole depths ranged from 5.5 m to 6.1 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 drawings 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 Occupancy Permit was obtained for unloading and loading the drill rig on the shoulder of Upper Middle Road for accessing Boreholes 13-03 and 13-04.

The drilling was carried out using a CME 55 track-mounted drill rig. A combination of solid stem augers and NQ coring techniques 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 supervisor logged the boreholes and processed the recovered soil and bedrock samples for transport to Thurber’s laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes during the drilling operations. Standpipe piezometers, consisting of a 25 mm diameter PVC pipe with slotted screen, were installed in three boreholes at this structure site. The installation details for the piezometers and borehole completion details for boreholes without a piezometer installation are summarized in Table 3.1.

Table 3.1 –Piezometer Installation and Borehole Completion Details

Borehole	Tip Position		Piezometer Installation and Borehole Completion Details
	Depth (m)	Elev. (m)	
13-03	5.5	147.1	Sand filter from 5.5 m to 3.7 m then bentonite holeplug to surface.
13-04	None installed		Backfilled with bentonite holeplug to surface.
13-05	6.1	145.7	Sand filter from 6.1 m to 4.3 m then bentonite holeplug to surface.
13-06	None installed		Backfilled with bentonite holeplug to surface.
13-07	6.1	145.2	Sand filter from 6.1 m to 4.3 m then bentonite holeplug to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and 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 portion 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 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 Drawings 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 typically consists of a thin layer of topsoil overlying silty clay fill (re-worked native silty clay) which is underlain by shale bedrock at a shallow depth. At one location some shale fill was encountered below the topsoil layer. More detailed descriptions of the individual strata encountered at the proposed structure site are presented below.

5.1 Topsoil

A thin layer of topsoil was encountered at the surface at all five borehole locations. The thickness of the topsoil ranged from 25 to 125 mm. The topsoil thickness may vary between and beyond the borehole locations.

5.2 Shale Fill

Reddish brown shale fill containing occasional rootlets was encountered below the topsoil in Borehole 13-03.

The thickness of the shale fill in Borehole 13-03 was 0.7 m with the lower boundary encountered at elevation 151.9.

A single SPT N-value of 21 blows for 0.3 m penetration was recorded in the shale fill, indicating a compact relative density. The moisture content of the shale fill was 10%.

5.3 Silty Clay Fill

This silty clay has been identified as fill since all of the land in this area would have been reworked during construction of the existing QEW/Highway 403 interchange.

The silty clay fill was encountered below the topsoil in Boreholes 13-04 to 13-07 and below the shale fill in Borehole 13-03. The silty clay fill was brown to reddish brown with trace sand, trace gravel, and occasional rootlets near the surface.

The thickness of the silty clay fill layer ranged from 0.6 to 1.4 m. The base of the silty clay fill was encountered at depths of 0.9 to 1.4 m (elevation 150.4 to 151.3).

SPT N-values recorded in the silty clay fill typically ranged from 7 to 21 blows for 0.3 m penetration, indicating a firm to very stiff consistency. An SPT N-value of 45 blows for 0.3 m penetration (hard) was recorded in the silty clay encountered below the shale fill in Borehole 13-03. The moisture content of samples of the silty clay fill ranged from 8 to 18%.

Three samples of the silty clay fill were selected for laboratory grains size analysis testing. The results of these tests are summarized below and are presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are plotted on Figure B1 of Appendix B.

Gravel%	0 to 1
Sand%	0 to 8
Silt%	53 to 68
Clay%	27 to 41

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5.4 Shale Bedrock

Highly weathered reddish-brown shale bedrock was encountered below the silty clay fill in all of the boreholes drilled for this structure. In general, the upper 1 to 2 m of the shale bedrock was highly weathered and solid stem augers were used to advance the boreholes through this material. Bedrock coring was initiated once more competent bedrock was encountered. The depths and elevations where bedrock was encountered in the boreholes are summarized in Table 5.1. These depths and elevations correspond to the highly weathered bedrock surface.

Table 5.1 – Depths and Elevations of Bedrock Surface

Foundation Element	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
East Abutment	13-07	0.9	150.4
Pier #1	13-06	0.9	150.9
Pier #2	13-05	0.9	150.9
Pier #3	13-04	1.4	150.8
West Abutment	13-03	1.3	151.3

The bedrock was described as thinly laminated reddish brown shale containing frequent hard grey limestone interbeds up to 200 mm in thickness. Frequent horizontal fractures and occasional vertical fractures, highly 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 47 to 100%, indicating a poor to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, typically ranged from 1 to 4 with occasional highly fractured zones with FI of greater than 10.

The average estimated unconfined compression strength (UCS) of the shale, interpreted from point load tests conducted on intact cores, ranged from 4 to 17 MPa, indicating a very weak to weak rock strength classification. In most boreholes there were limestone interbeds that were sufficiently thick for representative point load testing. The UCS of the limestone interbeds, interpreted from point load tests, ranged from 26 to 110 MPa, indicating a medium strong to very strong rock strength classification.

5.5 Water Levels

No groundwater was encountered in the boreholes prior to starting the coring operations. Standpipe piezometers were installed in the shale bedrock in three of the boreholes drilled for this structure. The water levels measured in the open boreholes and piezometers are as follows.

Table 5.2 – Groundwater Depths and Elevations

Borehole	Date of Reading	Water Level		Comment
		Depth (m)	Elev. (m)	
13-03	May 8, 2013	Dry		Open borehole – prior to coring
	May 30, 2013	4.5	148.1	Piezometer
	June 26, 2013	4.4	148.2	Piezometer
13-04	May 9, 2013	Dry		Open borehole – prior to coring
13-05	May 27, 2013	Dry		Open borehole – prior to coring
	May 30, 2013	3.7	148.1	Piezometer
	June 26, 2013	3.8	148.0	Piezometer
13-06	May 27, 2013	Dry		Open borehole - prior to coring
13-07	May 27, 2013	Dry		Open borehole - prior to coring
	May 30, 2013	3.9	147.4	Piezometer
	June 26, 2013	3.6	147.7	Piezometer

It should be noted that ground water level is susceptible to seasonal fluctuations. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

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. of Ajax, Ontario supplied a track-mounted CME 55 drill rig 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.

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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 structure proposed for the Highway 403 and QEW E-N (403/QEW E-N) Ramp.

The 403/QEW E-N Ramp will cross over Upper Middle Road and the QEW EB to Ford Dr N-S Ramp. The proposed finished grade of the 403/QEW E-N structure will be near elevations 160.6 and 157.4 at the west and east abutments, respectively. The existing ground surface within the proposed structure is near Elevations 152 at Upper Middle Road and around 151.5 at the centreline of the E-N/S ramp. The approach fill heights are approximately 8 m at the west abutment and 6 m at the east abutment.

The preliminary GA drawing (dated July 2013 from McCormick Rankin (MRC)) shows that the proposed 403/QEW E-N Ramp is a four span structure supported by two abutments and three piers. The ramp structure spans a distance of approximately 169.0 m and the bridge deck is approximately 18.2 m wide.

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

8 STRUCTURE FOUNDATIONS

The stratigraphy identified in the preliminary investigation consisted primarily of topsoil and shale/silty clay fill overlying shale bedrock at shallow depth. The short term groundwater level measured in the piezometers was 3.6 m to 4.5 m below the ground surface (Elevations 147.7 to 148.2).

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

- Spread footings bearing on bedrock
- Spread footings on engineered fill
- Driven steel H-piles
- Augered caissons socketed into bedrock

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix C.

8.1 Spread Footings on Bedrock

Since all earth overlying the bedrock appears to consist of fill, footings bearing on earth are not recommended. Spread footings founded 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 the elevations summarized in Table 8.1.

Table 8.1 – Bearing Resistances for Spread Footings

Element	Borehole	Depth (m)	Elev.	ULS_f (kPa)	Soil/Rock
West Abutment	13-03	2.4	150.2	1000	Shale
Pier #1	13-04	2.0	150.2	1000	Shale
Pier #2	13-05	1.8	150.0	1000	Shale
Pier #3	13-06	1.8	150.0	1000	Shale
East Abutment	13-07	1.8	149.5	1000	Shale

The bearing resistances in Table 8.1 are for vertical, concentric loading and include a resistance factor of 0.5 as per Table 6.1 of the CHBDC. The SLS condition will not govern the design of footings founded on bedrock. In the case of eccentric or inclined loading, the bearing resistance must be adjusted as shown in the CHBDC (2006) Clause 6.7.3 and Clause 6.7.4.

Founding elevations presented in Table 8.1 are generally above the groundwater level observed during the investigation. However, if temporary excavations are required to construct these footings below the water table, local groundwater control will be required to construct the footings in the dry and prevent disturbance of the footing base.

8.2 Spread Footings on Engineered Fill

The use of spread footings bearing on engineered fill pads is considered to be feasible for the construction of perched abutment foundations.

For preliminary design purposes, it is recommended that the engineered fill be assumed to bear on bedrock, i.e. all existing fill overburden must be stripped prior to constructing the engineered fill.

If a footing on engineered fill bearing on bedrock is used, it may be designed on the basis of the following concentric, vertical geotechnical resistances:

- 900kPa at factored ULS
- 350kPa at SLS

The engineered fill must be founded on the top of the undisturbed shale bedrock at or below the elevations given in Table 8.1 The engineered fill must consist of OPSS Granular "A" or Granular "B" Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content and generally conforming to the geometry illustrated in Figure 1.

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

Dewatering is not expected to be required for the excavation and construction of the engineered fill. However, any water accumulating in the excavation must be pumped out prior to placing the engineered fill.

8.3 Steel H-Piles

In view of the preliminary grade of the ramp and the bedrock elevations at this site, driven steel H-piles may be feasible at the west abutment, where the grade will be 8 m above the bedrock surface. At the east abutment, the grade will be in the order of 5 m above the bedrock surface and driven piles are not considered to be feasible. However, if a steel pile solution is favoured, the piles could be concreted into sockets in the bedrock.

It is recommended that the piles be driven to refusal in the shale bedrock at the depths and tip elevations shown in Table 8.2 or placed into sockets drilled to the elevation shown in Table 8.2

Table 8.2 – Estimated Pile Tip Elevation

Element	Pile Tip Depth (m)	Pile Tip Elevation
West Abutment (BH 13-03)	2.0 (below OGL)	150.6
East Abutment (BH 13-07)	3.8 (below OGL)	147.5

For preliminary design, the vertical, axial, factored geotechnical resistance at ULS for two pile sections when installed in the bedrock are presented in Table 8.3. The SLS condition will not govern for piles driven to bedrock.

Table 8.3 – Axial Resistance of Two Pile Sections Founded on Bedrock

Pile Section	Factored Geotechnical Resistance at ULS (kN)
HP 310 X 110	2,000
HP 310 X 132	2,400

The resistance values in Table 8.3 include a geotechnical resistance factor of 0.4 as per the CHBDC. 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.

These are preliminary recommendations and may change during detail design based on the final alignment, final bridge arrangement and the results of the investigation and field testing to be completed at that time.

8.4 Augered Caissons

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 conventional or semi-integral abutment design, principally due to the shallow depth to bedrock. Based on geotechnical considerations and the preliminary grade for the ramp, integral abutment design could also be considered.

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 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 footing bearing on undisturbed, shale bedrock for the piers. At the abutments, footings bearing on engineered fill are recommended. Driven piles may also be feasible at the west abutment.

9 DEWATERING

Excavations for footings to depths indicated in Table 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 of water are expected to be small. Similarly, some 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.

10 APPROACH EMBANKMENTS

Based on the current boreholes drilled at the site, the 6 to 8 m high approach embankments will be constructed over firm to hard silty clay fill or highly weathered compact shale fill over shale bedrock. The embankment foundation soils are considered to provide adequate stability if the approach embankments constructed at side slopes of 2H:1V 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 foundations 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 detailed design phase. Additionally, permanent drainage and slope protection requirements must be addressed during the detailed design.

11 ROADWAY PROTECTION

Excavation support systems may be required for temporary roadway protection during foundation construction where stable slopes cannot be constructed. 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 detailed design.

12 CONSTRUCTION CONCERNS

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

- The shale bedrock exposed in foundations must be concreted within 24 hours once the bedrock surface has been properly prepared and is free of loose debris to prevent softening and deterioration. Shale exposed below engineered fill is less critical but fill must be placed within one week of completing excavation.

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 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. Lukasz Gilarski, 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.

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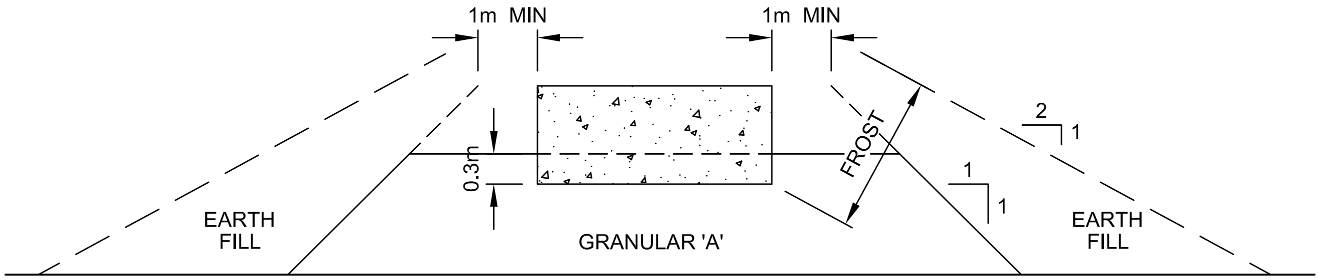
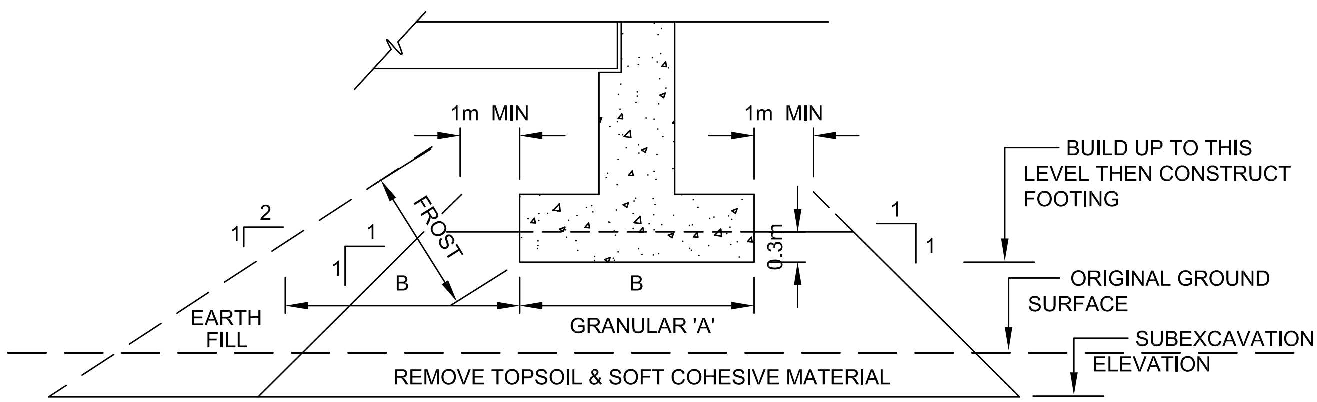


CROSS-SECTION



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 : LPG	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

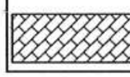
ROCK WEATHERING CLASSIFICATION

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)		APPROX. SPT[(1)] "N" VALUE
Very Soft	< 10		< 2
Soft	10 to 25	(POCKET PEN)	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

RECORD OF BOREHOLE No 13-03

METRIC

W.P. ______ LOCATION N 4 818 025.2 E 290 735.9 ORIGINATED BY GA

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

DATUM Geodetic DATE 2013.05.08 - 2013.05.08 CHECKED BY LRB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
152.6 / 0.0	**TOPSOIL:** (25mm)							
	SHALE, highly weathered, trace rootlets, Compact, Reddish Brown, Damp (FILL)	1	SS	21	152	○		
151.9 / 0.7	Silty **CLAY**, trace sand, occasional shale fragments, Hard, Reddish Brown (FILL)	2	SS	45		○		0 5 68 27
151.3 / 1.3	**SHALE**, highly weathered, thinly bedded, reddish brown	3	SS	39	151	○		
	Start coring at 2.4m. Slightly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds. Limestone interbeds (25mm thick) at 2.6m, 2.9m, 3.5m, 3.6m. Horizontal fracture at 2.4m, 2.5m, 2.6m, 2.9m, 3.0m, 3.2m, 3.4m, 3.5m, 3.6m, 3.7m, 3.8m	1	RUN		150, 149		FI: 3, 3, 2, 4, 3	RUN #1 TCR=100% SCR=92% RQD=47% UCS=4MPa (Average)
	Limestone interbeds (25mm thick) at 4.1m, 4.7m, 5.1m. Limestone interbed (25mm thick) at 4.6m, 4.8m, 5.3m. Horizontal fracture at 4.1m, 4.4m, 4.5m, 4.6m, 4.7m, 4.8m, 5.1m, 5.2m, 5.3m	2	RUN		148		1, 3, 4, 1, 3	RUN #2 TCR=100% SCR=92% RQD=63% UCS=14MPa (Average)
147.1 / 5.5	END OF BOREHOLE AT 5.5m. BOREHOLE OPEN TO 5.5m AND WATER LEVEL AT 0.6m UPON COMPLETION OF CORING. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.							

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
May 30/13	4.5	148.1
Jun 26/13	4.4	148.2

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

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

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

RECORD OF BOREHOLE No 13-04

METRIC

W.P.		LOCATION	N 4 818 045.5 E 290 756.2
HWY	403/QEW	BOREHOLE TYPE	Solid Stem Augers/NQ Coring
DATUM	Geodetic	DATE	2013.05.09 - 2013.05.09
ORIGINATED BY	GA	COMPILED BY	AN
CHECKED BY	LRB		

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
152.2 / 0.0	**TOPSOIL:** (25mm)					152		
	Silty **CLAY**, trace sand, trace gravel, occasional shale fragments Firm to Very Stiff Reddish Brown (FILL)	1	SS	7				
		2	SS	21	∇	151		1 8 53 38
150.8 / 1.4	**SHALE**, highly weathered, thinly bedded, reddish brown	3	SS	74		150		
	Start coring at 2.4m						FI	
	Slightly weathered to fresh, thinly bedded, reddish brown, occcasional limestone interbeds Horizontal fractures at 2.4m, 2.6m, 2.7m, 2.9m, 3.0m, 3.2m, 3.4m, 3.5m, 3.6m, 3.7m, 3.8m Clay seam at 2.9m	1	RUN			149	2 3 2 3 3	RUN #1 TCR=100% SCR=90% RQD=57% UCS=6MPa (Average)
	Limestone interbeds (75mm) at 3.3m, (25mm) at 3.7m and 3.9m, (200mm) at 4.3, (50mm) at 4.9m and 5.4m Horizontal fractures at 4.1m, 4.3m, 4.4m, 4.5m, 4.6m, 4.9m, 5.3m	2	RUN			148 147	1 4 1 2 1	RUN #2 TCR=100% SCR=100% RQD=73% UCS=17MPa (Average)
146.8 / 5.5	END OF BOREHOLE AT 5.5m. BOREHOLE OPEN TO 5.5m AND WATER LEVEL AT 0.9m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

+ 3, × 3: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 13-05

METRIC

W.P. ______ LOCATION N 4 818 066.3 E 290 824.0 ORIGINATED BY GA

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

DATUM Geodetic DATE 2013.05.27 - 2013.05.28 CHECKED BY LRB

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.8 / 0.0	**TOPSOIL:** (110mm)							
0.1	Silty **CLAY**, trace sand, occasional rootlets, Stiff, Brown to Reddish Brown (FILL)	1	SS	11		○		0 0 59 41
150.9 / 0.9	**SHALE**, highly weathered, thinly bedded, reddish brown	2	SS	30	151	○		
		3	SS	50/ 0.150	150	○		
	Start coring at 3.1m. Slightly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds. Clay seam (100mm) at 3.2m. Horizontal fracture at 3.1m, 3.3m, 3.6m, 4.0m, 4.4m. Highly broken zone (75mm) at 3.9m. Limestone interbeds (25mm) at 4.1m, 4.2m, 4.4m, 4.7m, 4.8m, 4.9m, 5.9m and (100mm) at 3.6m	1	RUN		149, 148		FI: 2, 2, >5, 1, 2	RUN #1 TCR=100% SCR=87% RQD=80% UCS=11MPa (Average)
	Horizontal fracture at 4.9m, 5.2m, 5.5m, 5.8m	2	RUN		147, 146		0, 1, 1, 1, 1	RUN #2 TCR=100% SCR=100% RQD=100% UCS=13MPa (Average)
145.7 / 6.1	END OF BOREHOLE AT 6.1m. BOREHOLE OPEN TO 6.1m AND WATER LEVEL AT 2.4m UPON COMPLETION OF CORING. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.							

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
May 30/13	3.7	148.1
Jun 26/13	3.8	148.0

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

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

ONTMT4S 1184.GPJ 2012TEMPLATE(MTO).GDT 7/24/13

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THURBER

RECORD OF BOREHOLE No 13-06

METRIC

W.P. ______ LOCATION N 4 818 089.0 E 290 852.6 ORIGINATED BY GA

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

DATUM Geodetic DATE 2013.05.27 - 2013.05.27 CHECKED BY LRB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³ / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.8									
0.0	**TOPSOIL:** (125mm)								
0.1	Silty **CLAY**, trace sand Stiff Reddish Brown (FILL)	1	SS	10			○		
150.9 / 0.9	**SHALE**, highly weathered, thinly bedded, reddish brown	2	SS	80		151	○		
		3	SS	50/ 0.150	∇	150	○		
	Start coring at 3.1m					149			
	Slightly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds Horizontal fractures at 3.0m, 3.1m, 3.2m, 3.6m, 3.7m, 4.3m, 4.5m Highly broken zone (25mm) at 3.0m, 3.1m and (250mm) at 3.3m	1	RUN			148		FI 3 >5 1 0 3	RUN #1 TCR=100% SCR=77% RQD=63% UCS=17MPa (Average)
	Limestone interbeds (25mm) at 3.6m, 3.8m, 4.0m, 4.3m, 4.4m, 4.5m, 5.4m, 5.7m, 6.0m and (50mm) at 4.5m Horizontal fractures at 4.6m, 4.9m, 5.4m Limestone interbed at 5.6m Soft zone (25mm) at 6.1m	2	RUN			147 146		2 3 1 0 0	RUN #2 TCR=100% SCR=97% RQD=78% UCS=8MPa (Average)
145.7 / 6.1	END OF BOREHOLE AT 6.1m. BOREHOLE OPEN TO 6.1m AND WATER LEVEL AT 1.9m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

+³, ×³: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 13-07

METRIC

W.P.		LOCATION N 4 818 101.2 E 290 887.1	ORIGINATED BY GA
HWY 403/QEW		BOREHOLE TYPE Solid Stem Augers/NQ Coring	COMPILED BY AN
DATUM Geodetic		DATE 2013.05.26 - 2013.05.27	CHECKED BY LRB

SOIL PROFILE | 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	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT / FI	REMARKS
151.3						
0.0	**TOPSOIL:** (125mm)					
0.1	Silty **CLAY**, trace sand, occasional rootlets Stiff Brown (FILL)	1	SS	14		
150.4 / 0.9	**SHALE**, highly weathered, reddish brown	2	SS	18		
		3	SS	50/ 0.150		
	Start coring at 3.1m Highly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds Highly broken zone (0.6m) at 3.0m Soft zone (305mm) at 3.0m Horizontal fracture at 3.8m, 4.1m	1	RUN		FI >10 >5 1 1 0	RUN #1 TCR=100% SCR=67% RQD=60% UCS=12MPa (Average)
	Limestone interbeds (25mm) at 3.8m, 3.9m, 4.0m, 4.1m, 4.4m, 4.6m, 4.7m, 5.5m, 5.8m, 6.0m Horizontal fracture at 4.6m, 4.7m, 4.9m, 5.1m, 5.2m, 5.3m, 5.5m	2	RUN		2 2 2 1 0	RUN #2 TCR=100% SCR=100% RQD=93% UCS=16MPa (Average)
145.2 / 6.1	END OF BOREHOLE AT 6.1m. BOREHOLE OPEN TO 6.1m AND WATER LEVEL AT 2.1m UPON COMPLETION OF CORING. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

Elevation scale: 151, 150, 149, 148, 147, 146

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
May 30/13	3.9	147.4
Jun 30/13	3.6	147.7

+³, ×³: Numbers refer to Sensitivity

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

Appendix B

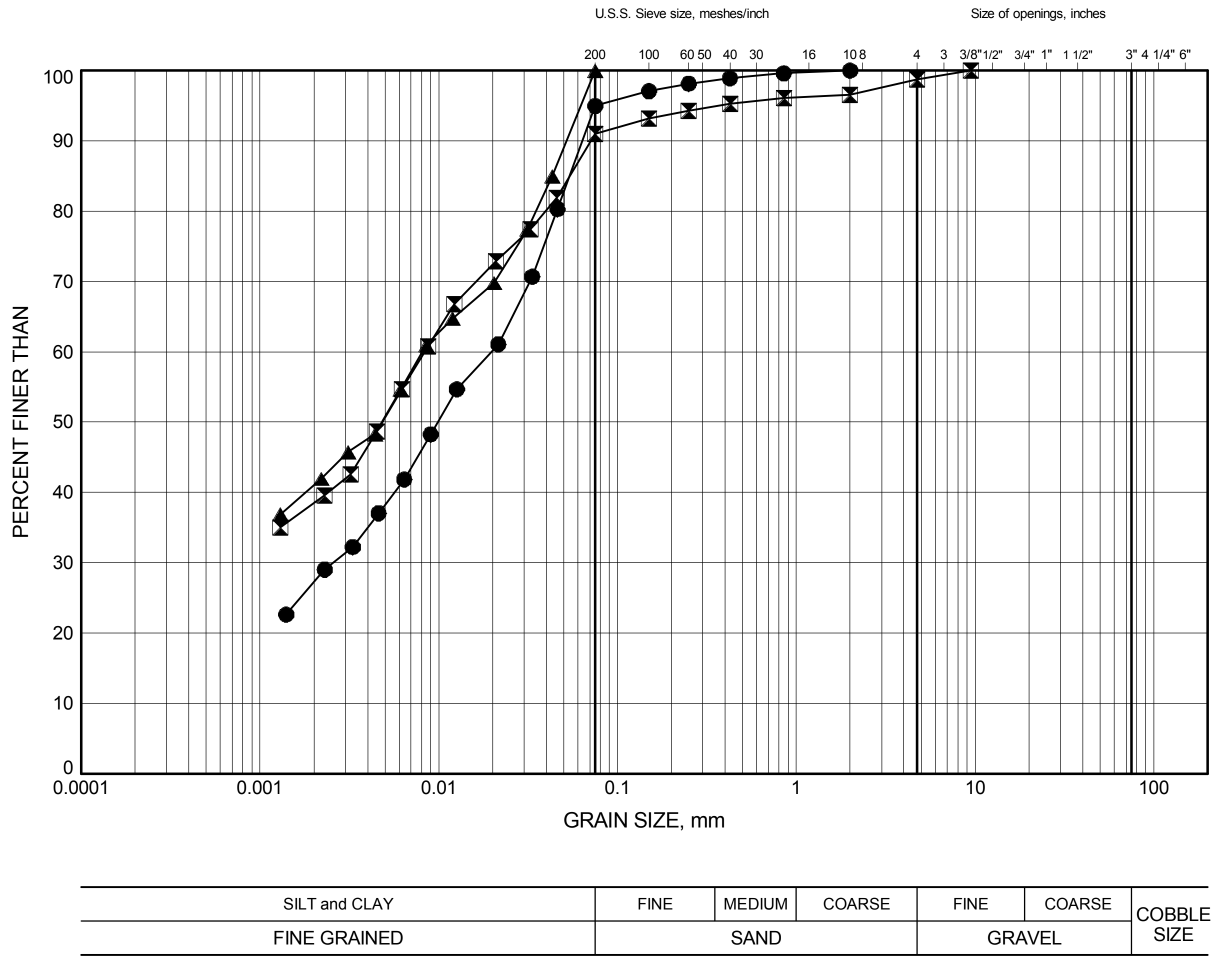
Laboratory Test Results

QEW and Hwy 403

GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty CLAY FILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-03	0.99	151.59
⧗	13-04	0.99	151.25
▲	13-05	0.38	151.45

GRAIN SIZE DISTRIBUTION - THURBER 1184.GPJ 7/22/13

Date July 2013

W.P.

THURBER

Prep'd AN

Chkd. LPG

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Spread Footing on Bedrock	Spread Footing on Engineered Fill	Steel H-Piles Driven to Shale Bedrock
Advantages: i. Economical to install. ii. Higher geotechnical resistance than footing on native soil.	***Advantages:*** i. Economical to install ii. Accommodates perched abutment.	***Advantages:*** i. High geotechnical resistance available by driving piles to bedrock. ii. Comparatively short abutment possible iii. Permits integral abutment design.
Disadvantages: i. Dewatering may be required depending on depth of excavation ii. Ineffective for resistance to uplift.	***Disadvantages:*** i. Dewatering may be required depending on depth of excavation ii. Lower geotechnical resistance than spread footing on bedrock iii. Ineffective for resistance to uplift.	***Disadvantages:*** i. Higher unit cost compared to footings.
RECOMMENDED FOR PIERS	**RECOMMENDED FOR ABUTMENTS**	**FEASIBLE FOR WEST ABUTMENT**

Appendix D

Site Photographs

Photograph 1: Looking east from Upper Middle Road along the proposed alignment of the HWY403-QEW E-N ramp (orange stakes).

Photograph 2: Looking west on Upper Middle Road at the location of the proposed ramp structure.

Photograph 3: Looking west along Upper Middle Road at the location of the proposed west abutment.

Appendix E

Borehole Locations and Soil Strata Drawing

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

CONT No
WP No

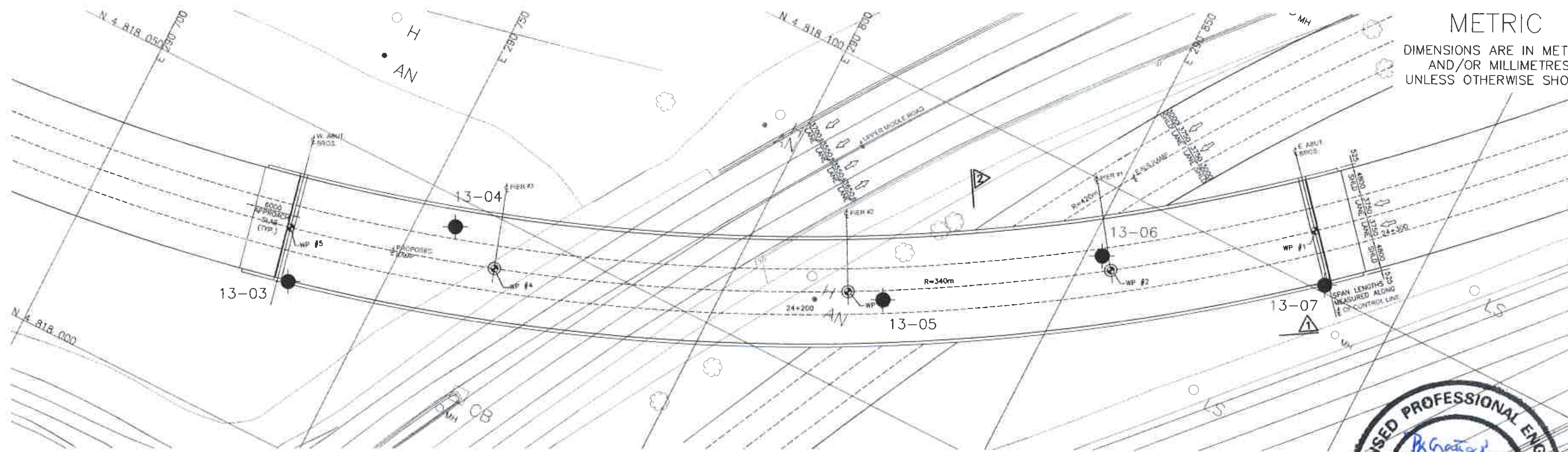
HWY 403/Q.E.W.
E–N RAMP

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McCORMICK RANKIN
A member of MMM GROUP

THURBER ENGINEERING LTD.

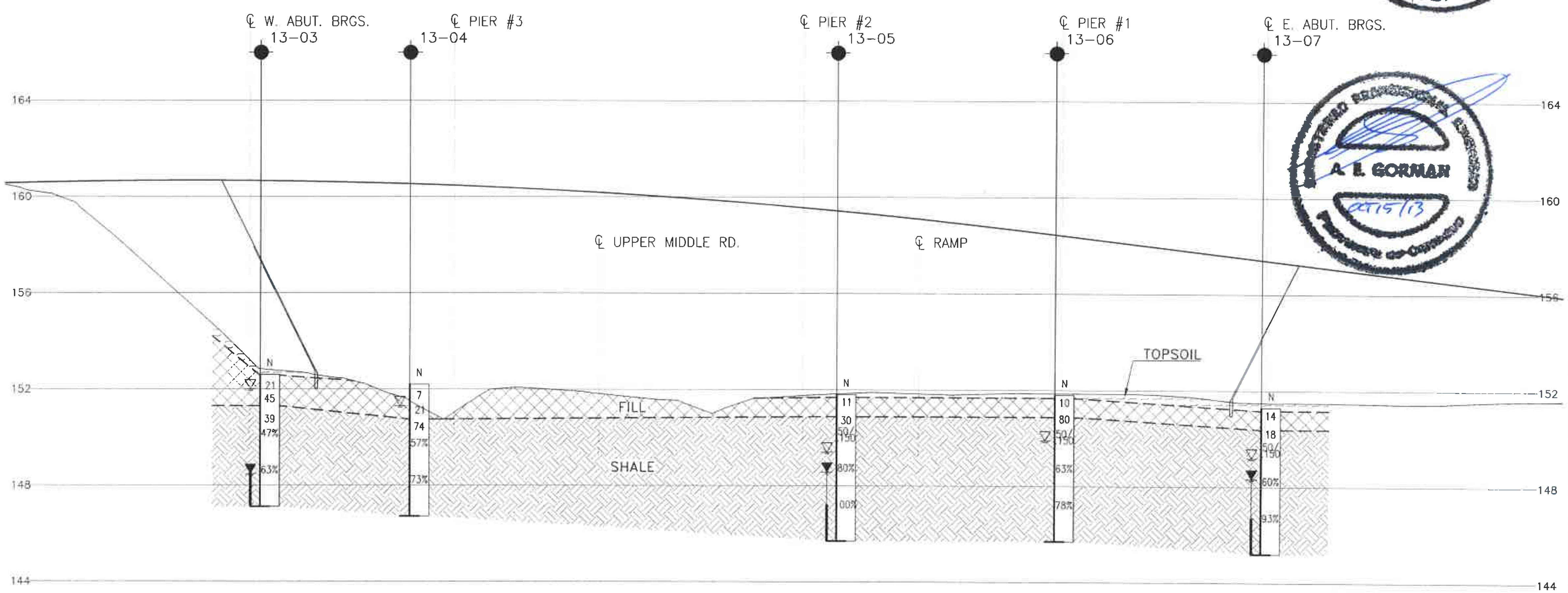


PLAN

16 0 16 32m
SCALE 1:800

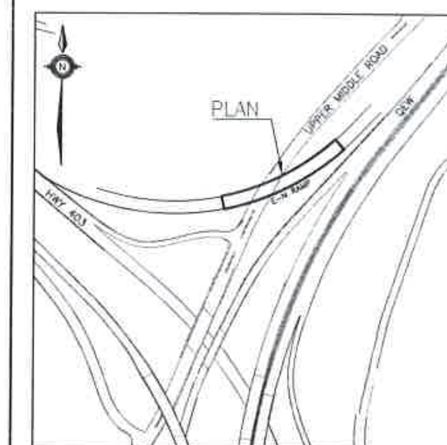
LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 15/13
PROVINCE OF ONTARIO

A. E. GORMAN
Oct 15/13



PROFILE ALONG ℄ OF E–N RAMP

16 0 16 32m H 1:800
4 0 4 8m V 1:200



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
13–03	152.6	4 818 025.2	290 735.9
13–04	152.2	4 818 045.5	290 756.2
13–05	151.8	4 818 066.3	290 824.0
13–06	151.8	4 818 088.9	290 852.6
13–07	151.3	4 818 101.2	290 887.1

-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. 30M5-292

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION			
DESIGN LPG	CHK AEG	CODE	LOAD	DATE JUL 2013		
DRAWN MFA	CHK LPG	SITE	STRUCT	DWG 1		

FILENAME: H:\Drafting\19\1351\184\ted184–Plan&Profile–321(Hwy403–QEW E–N Ramp).dwg
PLOTDATE: 10/11/2013 5:09 PM