PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
W-N RAMP HWY 403 UNDER QEW EASTBOUND
QUEEN ELIZABETH WAY/HIGHWAY 403 IMPROVEMENTS
OAKVILLE, ONTARIO

W.O. 09-20007, SITE No. 10-284-2

Geocres Number: 30M5-299

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 new structure which will carry eastbound Queen Elizabeth Way (QEW) over Highway 403 W-N Ramp 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 the investigation and presented in this report is intended for preliminary design purposes only. Additional site investigation, field testing and engineering analysis will 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.

A previous foundation investigation report was completed in 1978 for the existing ramp structure located 5 m north of the proposed structure. The title of the report is as follows:

> Foundation Investigation Report for W-N Ramp HWY 403 Under QEW, District 4, (Hamilton), W.P. 159-75-06, Geocres 30M05-117, Site 10-284, dated February 1978

The Record of Borehole sheets for four boreholes (BH 5, 6, 7 and 8),, drilled during the previous investigation, have been used in the preparation of this report and are included in Appendix A.

2 SITE DESCRIPTION

The site is located on the QEW approximately 500 m east of the existing Ford Drive underpass structure. At this location the QEW has both vertical and horizontal curvature and consists of 6 lanes of traffic, three EB lanes and three WB lanes. This general area has a gentle slope to the south and west towards Joshua Creek. The lands surrounding the site are primarily undeveloped fields in the adjacent MTO right-of-way. There are commercial developments further to the north east and residential developments further to the southeast.

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

A photograph included in Appendix D shows 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 21 and 22, 2013. Two boreholes were drilled at this proposed structure site, identified as 13-27 and 13-28. Borehole 13-27 was drilled near the west abutment and was 6.1 m deep while Borehole 13-28 was drilled near the east abutment and was 9.1 m deep. Four boreholes which were drilled during the previous investigation (No. 5, No. 6, No .7, and No. 8) have also been included in this report. The Record of Borehole sheets are included in Appendix A.

The approximate locations of the boreholes are shown on the attached Borehole Location 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.

The drilling was carried out using a CME 75 truck-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 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 starting the coring operations. Since water is added to the borehole for coring, water levels observed in the open boreholes after coring are likely not representative of the natural groundwater conditions. No standpipe piezometers were installed at this site as both boreholes were drilled in the travelled lane of the highway. The borehole completion details are summarised in Table 3-1.

Table 3-1. Borehole Completion Details

Borehole	Borehole Completion Details
BH13-27	Borehole backfilled with bentonite holeplug from 6.1 m to 0.3 m, concrete from 0.3 m to 0.15 m, then asphalt patch to surface.
BH13-28	Borehole backfilled with bentonite holeplug from 9.1 m to 0.3 m, concrete from 0.3 m to 0.15 m, then asphalt patch 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 per run).

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A (current investigation and previous investigation), and the Borehole Location 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 in the boreholes generally consists of a thin layer of asphalt overlying gravelly sand fill which is underlain by silty clay which in turn is underlain by shale bedrock. The boreholes drilled during the previous investigation only encountered silty clay overlying the shale bedrock. More detailed descriptions of the individual strata encountered at the proposed structure site are presented below.

5.1 Asphalt

Asphalt was encountered at the surface in both boreholes drilled during the current investigation. The asphalt layer was 150 mm thick at the west abutment and 300 mm thick at the east abutment.

5.2 Gravelly Sand Fill

Gravelly sand fill, some fines was encountered below the asphalt layer in both boreholes drilled during the current investigation. The thickness of the gravelly sand fill ranged from 0.6 m in Borehole 13-27 to 2.0 m in Borehole 13-28. The lower boundary of the fill was encountered at depths of 0.8 to 2.3 m (Elev. 143.5 to 144.7 m).

SPT N-values of 16 to 55 blows for 300 mm of penetration were recorded, indicating a compact to very dense relative density, typically compact. Moisture contents ranged from 4 to 6%.

Laboratory grain size distribution analyses were carried out on two samples of the fill. The results of these tests are summarized below and are presented on the corresponding Record of Borehole sheets included in Appendix A and the grain size distribution curves are present on Figure B1 of Appendix B:

Gravel %	33 to 34
Sand %	44 to 48
Silt and Clay %	20 to 22

5.3 Silty Clay

A layer of native silty clay, trace to some sand and trace gravel was encountered below the fill in both boreholes drilled during the current investigation and at the surface in the four boreholes drilled during the previous investigation. The thickness of the silty clay ranged from 0.8 to 2.4 m, with the lower boundary encountered at depths of 1.6 to 4.6 m (Elev. 144.3 to 142.0 m)

SPT N-values recorded in the silty clay ranged from 12 to 30 blows for 300 mm of penetration, indicating a stiff to very stiff consistency. Moisture contents ranged from 10 to 23%.

Laboratory grain size distribution analysis was completed on two samples of the silty clay (one from the current investigation and one from the previous investigation). The results of these tests are summarized below and are presented on the corresponding Record of Borehole sheets included in Appendix A and the grain size distribution curve for the samples from the current investigation is presented on Figure B2 in Appendix B.

Gravel %	0
Sand %	4 to 13
Silt %	45
Clay %	43 to 51

A single sample of the silty clay (from the previous investigation) underwent Atterberg Limits testing. The results of this test is presented on the corresponding Record of Borehole sheet included in Appendix A and indicate that the silty clay exhibits intermediate plasticity with a plastic limit of 20 and a liquid limit of 42.

5.4 Shale Bedrock

Reddish brown, thinly bedded shale bedrock was encountered below the native silty clay and was proven by coring in both boreholes drilled during the current investigation and in two boreholes from the previous investigation (No.6 and No.7). Boreholes No.5 and No.8 penetrated the bedrock with augers. The shale was observed to have frequent thin hard limestone interbeds up to 300 mm thick. The shale bedrock was highly weathered near the bedrock surface and slightly weathered to fresh within 1 to 1.5 m of the soil-bedrock interface.

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)
West Abutment	BH13-27	1.6	142.8
	No.5	2.4	142.0
	No.6	2.4	142.7
East Abutment	BH13-28	4.6	142.4
	No.7	2.4	143.7
	No.8	2.4	144.3

Total Core Recovery (TCR) in the bedrock ranged from 95 to 100%, indicating very good core recovery. The Rock Quality Designation (RQD) values ranged from 20 to 75%, indicating very poor to fair rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 1 to greater than 10. In general, the bedrock was more highly fractured near the bedrock surface.

The average estimated unconfined compression strength (UCS) of the shale, interpreted from point load tests conducted on intact cores, ranged from 1 to 15 MPa, indicating an very weak to weak rock strength classification. Some of the limestone interbeds were sufficiently thick for representative point load testing and the UCS of the limestone interbeds ranged from 93 to 210 MPa, indicating a strong to very strong rock strength classification for these layers.

5.5 Groundwater Levels

Both boreholes were observed to be dry upon completion of augering (at depths of 3.1 and 6.1 m, respectively), prior to commencement of the coring operations, where water was added to the boreholes. No standpipe piezometers were installed due to the locations of the boreholes on the existing highway.

In the boreholes drilled during the previous investigation, the water level was recorded at a depth of approximately 2 m below ground surface.

It should be noted that the 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 events.

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 truck mounted CME 75 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, C. Tech. 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 overpass.

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

- the proposed structure will carry eastbound Queen Elizabeth Way (QEW) traffic over Highway 403
- the proposed structure will carry four lanes of traffic and will comprise of 34.0, 59.0 and 34.0 m spans
- the proposed structure will have conventional abutments with concrete wing walls at the east abutment (Elev. 148.5 m) and RSS walls at the west abutment (Elev. 143.3 m)

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 FOUNDATION

The stratigraphy identified in the preliminary investigation generally consists of a thin layer of asphalt overlying gravelly sand fill followed by silty clay, all underlain by shale bedrock. The short term groundwater levels measured during the previous investigation was approximately at Elev. 144.7 to 142.5 m.

In the preparation of the preliminary geotechnical design 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

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

8.1 Spread Footings Bearing on Shale Bedrock

Highway 403 is constructed in a cut at the proposed structure location. Due to the shallow depth of overburden, spread footings founded on shale bedrock are considered feasible to support the structural loads.

As interpreted from the boreholes, spread footing should be founded on undisturbed shale bedrock at or below elevation 141.5 m at the west and east abutments. Based on the previous investigation, footings at the piers could also be founded on shale bedrock below elevation 141.5 m. However, the existing grade may be lower and all footings must be placed at least 0.5 m below the bedrock surface. The elevations presented are the highest recommended founding elevations 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 the footing founded on undisturbed shale bedrock should be designed using a factored geotechnical resistance at ULS of 1000 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) Clauses 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 native soil at or below elevation 143.5 m at the west abutment and on undisturbed shale at or below elevation 144.7 m at east abutment. 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 detailed 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. 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 the 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 310x100 steel H-piles concreted into 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 engineer.

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 conventional or semi-integral abutment design, principally due to the 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 required for footings founded on shale bedrock.

8.7 Recommended Foundation

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

9 DEWATERING

Excavation for spread footings at the elevations given in Section 8.1 are expected to penetrate 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 with a concrete slab within 24 hours of completion of the excavation.

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

10 APPROACH EMBANKMENTS

Based on the current and previous boreholes drilled at this site, the approach embankments will be constructed over foundation soils consisting of stiff to very stiff native silty clay and shale bedrock. The embankment foundation soils are considered to provide adequate stability 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 detailed design.

12 CONSTRUCTION CONCERNS

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

- The shale bedrock exposed at the footing base 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
- Excavation must not undermine the footings of any portion of the existing QEW W-N ramp structure that is still in service

13 INVESTIGATION FOR DETAIL DESIGN

During the detailed design phase of the project, additional site investigation and field testing will be required. The scope and results of this investigation must be reviewed at that time based on the final GA to determine they meet the current Ministry requirements and if additional investigation and analysis is necessary. In particular, subsurface conditions at the pier foundations must be investigated.

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.

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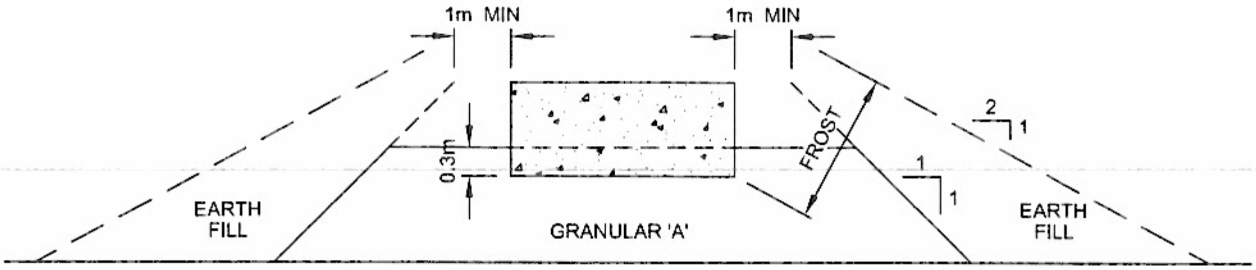
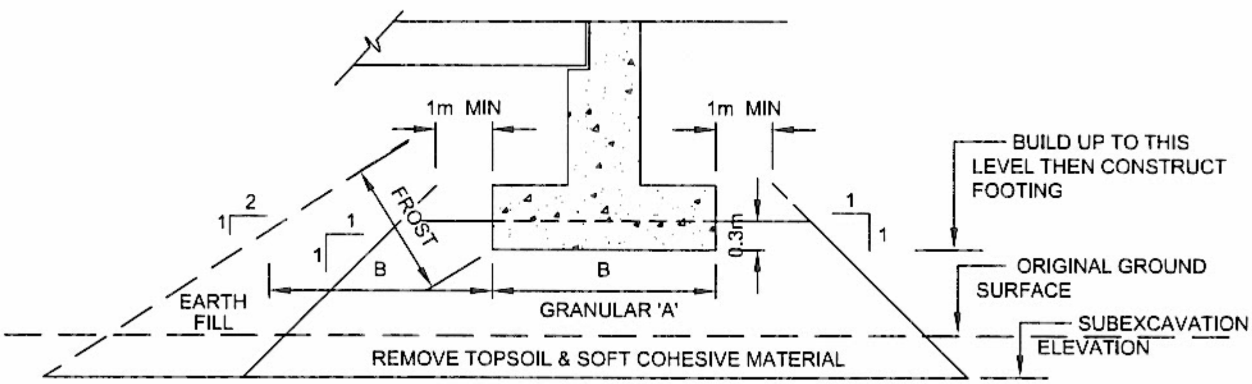


CROSS-SECTION



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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-27

METRIC

W.P. ______ LOCATION N 4 817 672.3 E 290 767.7 ORIGINATED BY GA

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

DATUM Geodetic DATE 2013.05.21 - 2013.05.22 CHECKED BY LRB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
144.4 / 0.0	**ASPHALT**: (150mm)								
0.2	Gravelly **SAND**, some silt Compact Brown Damp (FILL)	1	SS	26		144	o		34 44 22 (SI+CL)
143.5 / 0.8	Silty **CLAY**, trace sand, trace gravel Very Stiff Reddish Brown	2	SS	23		143	o		
142.8 / 1.6	**SHALE**, highly weathered, thinly bedded, reddish brown	3	SS	34	∇		o		
		4	SS	36		142	o		
	Start coring at 3.0m Highly to slightly weathered, thinly bedded, occasional limestone interbeds Frequent clay seams from 3.1m to 3.4m Horizontal fractures at 3.4m, 3.5m, 3.6m, 3.7m, 3.8m Limestone interbeds (25mm) at 3.2m, 4.0m and (75mm) at 3.6m	1	RUN			141, 140		FI >5, 3, >10, >5, >5	RUN #1 TCR=100% SCR=57% RQD=20% UCS=1MPa (Average)
138.3 / 6.1	Highly broken zone: 225mm at 3.7 300mm at 4.5m 100mm at 5.7m Horizontal fracture from 4.9m to 5.6m Limestone interbeds (25mm) at 4.6m, 5.1m, 5.3m, 5.4m, 5.7m, 5.9m, 6.0m	2	RUN			139		>10, 5, 3, 3, >5	RUN #2 TCR=100% SCR=70% RQD=48% UCS=5MPa (Average)

END OF BOREHOLE AT 6.1m. BOREHOLE OPEN TO 6.1m 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
UNIT WEIGHT γ kN/m³

$+^3$, $\times^3$: Numbers refer to Sensitivity; 15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-28

METRIC

W.P.		LOCATION	N 4 817 778.0 E 290 786.7
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		

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

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
147.0 / 0.0	**ASPHALT:**(300mm)					147		
146.7 / 0.3	Gravelly **SAND**, some silt; Compact to Very Dense; Reddish Brown; Damp; (FILL)	1	SS	30				
		2	SS	55		146		33 47 20 (SI+CL)
		3	SS	16		145		
144.7 / 2.3	Silty **CLAY**, trace to some sand, trace gravel; Very Stiff to Stiff; Brown to Reddish Brown	4	SS	18		144		
		5	SS	12		143		0 12 45 43
142.4 / 4.6	**SHALE**, highly weathered, reddish brown	6	SS	45	▽	142		
						141	FI	
	Start coring at 6.1m; Slightly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds; 175mm soft zone at 6.1m; Highly broken zone (100mm) at 6.2m, 6.5m; Limestone interbeds (25mm) at 6.4m and (150mm) at 6.8m and 7.3m	1	RUN			140	>5, >5, 3, 2, 1	RUN #1 TCR=100% SCR=83% RQD=48% UCS=9MPa (Average)
	Horizontal fractures at 6.2m, 6.4m, 6.5m, 6.8m, 6.9m, 7.0m, 7.4m, 7.7m, 7.8m, 7.9m, 8.1m, 8.2m, 8.5m, 8.9m and (150mm) at 8.6m; 50mm highly broken zone at 7.6m; Limestone interbeds (25mm) at 7.7m, 7.9m, 8.1m, 8.2m, 8.3m, 8.5m, 8.7m	2	RUN			139	>5, 2, 2, 3, 2	RUN #2 TCR=100% SCR=90% RQD=68% UCS=15MPa (Average)
137.9 / 9.1	END OF BOREHOLE AT 9.1m. BORHEOLE OPEN TO 9.1m AND WATER LEVEL AT 4.9m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m.					138		

Continued Next Page

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-28

METRIC

W.P. ______ LOCATION N 4 817 778.0 E 290 786.7 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

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Continued From Previous Page										
	CONCRETE TO 0.15m THEN ASPHALT PATCH TO SURFACE.										

ONTMT4S 1184.GPJ 2012TEMPLATE(MTO).GDT 8/8/13

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

RECORD OF BOREHOLE No 5

W P 159-75-06 LOCATION N 15 805 318 E 953 954 Co-ords. ORIGINATED BY P.J.S.
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger COMPILED BY P.J.S.
DATUM Geodetic DATE December 22, 1977 CHECKED BY R.S.

ELEV / DEPTH	SOIL PROFILE: 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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
474.0	Ground Level										
0.0	SILTY CLAY TRACE OF SAND Very Stiff To Hard										
			1	SS	30		470				
466.0 / 8.0			2	SS	87	▽					
	QUEENSTON SHALE BEDROCK		3	SS	100/13"						
			4	GS	50/1"						
	Red To Grey Red		5	SS	100/11"		460				
453.8 / 20.[illegible]			6	SS	75/3"						
	End Of Borehole										

RECORD OF BOREHOLE No 6

W P 159-75-06 LOCATION N 15 805 395 E 953 965 Co-ords. ORIGINATED BY P.J.S.
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger, BX Casing, BXL Core COMPILED BY P.J.S.
DATUM Geodetic DATE December 20, 1977 CHECKED BY R.S.

ELEV / DEPTH	SOIL PROFILE: 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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
476.2	Ground Level										
0.0	SILTY CLAY TRACE OF SAND Very Stiff To Hard		1	SS	21		470				
468.2 / 8.0			2	SS	53						
	QUEENSTON SHALE BEDROCK		3	SS	100/6"						
	Red To Grey Red Fine Texture Soft And Fissile With Thin Bedding		4	BXL Core	95% Rec		460				RQD = 38
	Including A Few Thin Beds Of Shaly Limestone Limestone Bed 31'9"-32'2'		5	BXL Core	100% Rec		450				RQD = 75
	Limestone Bed 40'6"-41'4"		6	BXL Core	95% Rec		440				RQD = 33
431.9 / 44.3	End Of Borehole										
	Note: W.L. Not Established										

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

W P 159-75-06 LOCATION N 15 805 525 E 953 990 Co-ords. ORIGINATED BY P.J.S.

DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger, BX Casing, BXL Core COMPILED BY P.J.S.

DATUM Geodetic DATE December 20, 1977 CHECKED BY R.S.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
479.4	Ground Level						
0.0	SILTY CLAY TRACE OF SAND Very Stiff To Hard	1	SS	21			0 4 45 51
471.4 / 8.0	QUEENSTON SHALE BEDROCK	2	SS	116/15"		470	0 7 63 30
		3	SS	100/12"			0 3 71 26
	Red To Grey Red Fine Texture Soft And Fissile With Thin Bedding Including A Few Shaly Limestone Beds	4	BXL Core	98% Rec		460	RQD = 50
	Shaly Limestone 15'8"-16'0" Shaly Limestone 40'8"-41'0"	5	BXL Core	100% Rec		450	RQD = 67
	Shaly Limestone 43'5"-44'5"	6	BXL Core	100% Rec		440	RQD = 54
434.4 / 45.0	End Of Borehole Note: W.L. Not Established						

Natural moisture content (%): Sample 1 ≈ 23, Sample 2 ≈ 13, Sample 3 ≈ 9 (plotted).

RECORD OF BOREHOLE No 8

W P 159-75-06 LOCATION N 15 805 601 E 954 015 Co-ords. ORIGINATED BY P.J.S.

DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger COMPILED BY P.J.S.

DATUM Geodetic DATE December 21, 1977 CHECKED BY R.S.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
481.3	Ground Level						
0.0	SILTY CLAY TRACE OF SAND Very Stiff To Hard	1	SS	16		480	
473.3 / 8.0	QUEENSTON SHALE BEDROCK	2	SS	56	▽		
		3	SS	100/7"		470	
	Red To Grey Red	4	SS	100/5"			
		5	SS	75/3"			
463.6 / 17.7	End Of Borehole	6	SS	75/1"			

Column headings (both records): 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity   15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Appendix B

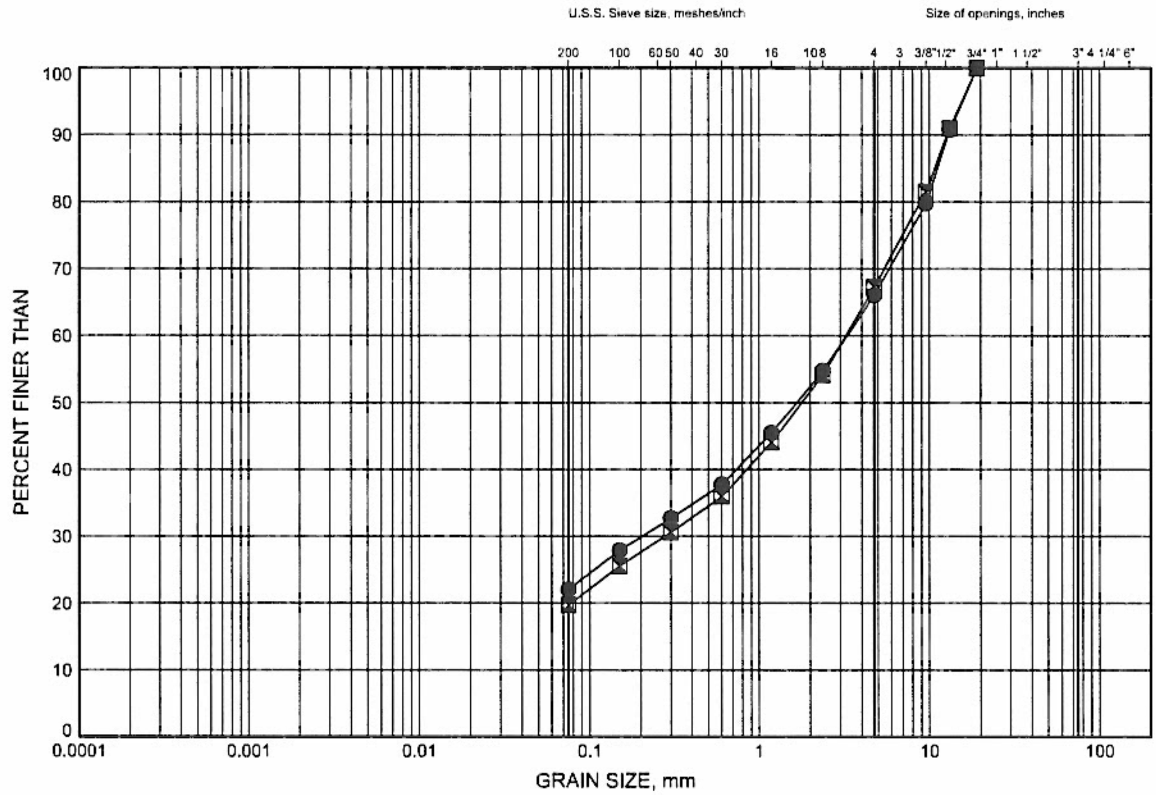
Laboratory Test Results

QEW and Hwy 403

GRAIN SIZE DISTRIBUTION

FIGURE B1

Gravelly SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-27	0.38	143.99
⊠	13-28	1.07	145.95

Date August 2013

W.P.

THURBER

Prep'd SBP

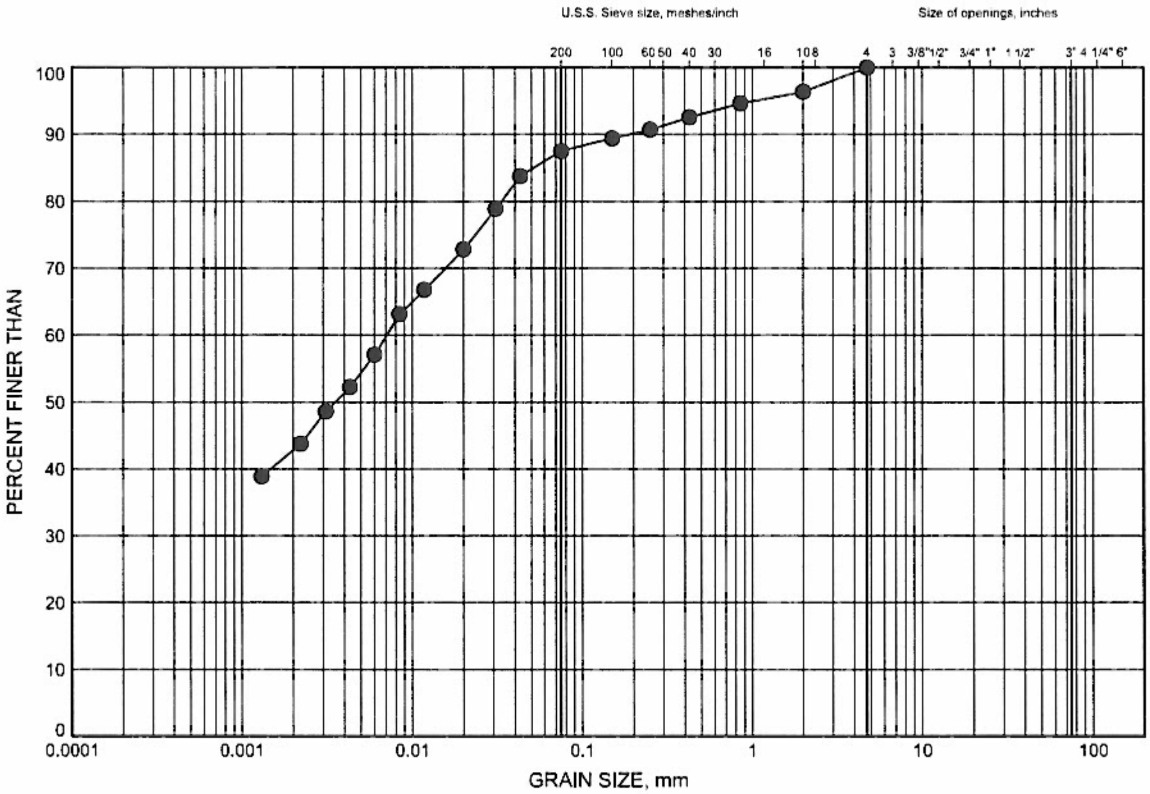
Chkd.

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)
●	13-28	3.35	143.66

Date August 2013

W.P.

THURBER

Prep'd SBP

Chkd.

Appendix C

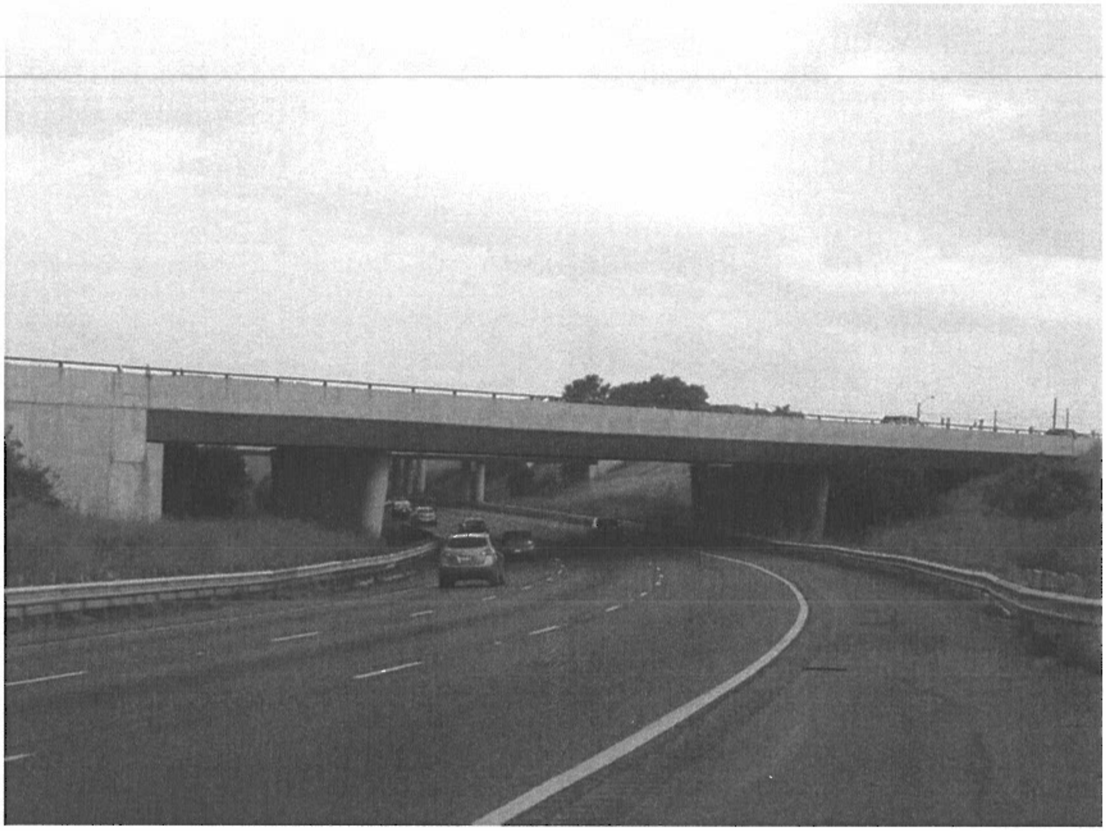
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Spread Footing on Shale Bedrock	Spread Footing on Engineered Fill	Steel H-Piles on Shale Bedrock
Advantages: i. Generally less costly construction than deep foundation elements.	*Advantages:* i. Economical to install ii. Accommodates perched abutment.	*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. Dewatering may be required, depending on depth of excavation ii. Ineffective for resistance to uplift or overturning.	*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.	*Disadvantages:* i. Higher unit cost compared to spread footings. ii. Difficultly 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**	**NOT RECOMMENDED**

Appendix D

Site Photographs



Photograph 1 – Existing structure carrying EB QEW over HWY 403

Appendix E

Borehole Locations and Soil Strata Drawing

W–N RAMP HWY 403 UNDER Q.E.W. EASTBOUND
BOREHOLE LOCATIONS AND SOIL STRATA

CONT No
WP No
SHEET

McCORMICK RANKIN
A member of MMM GROUP

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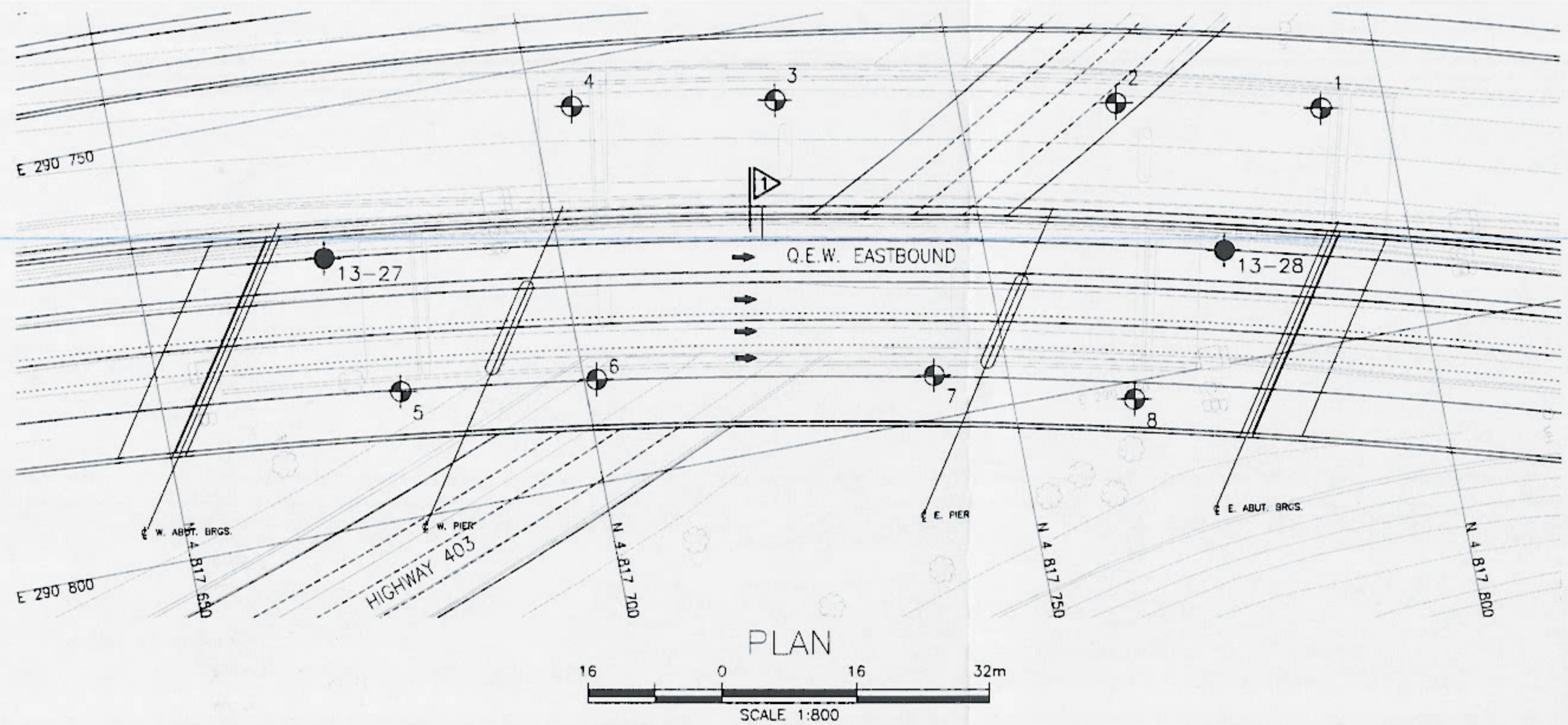
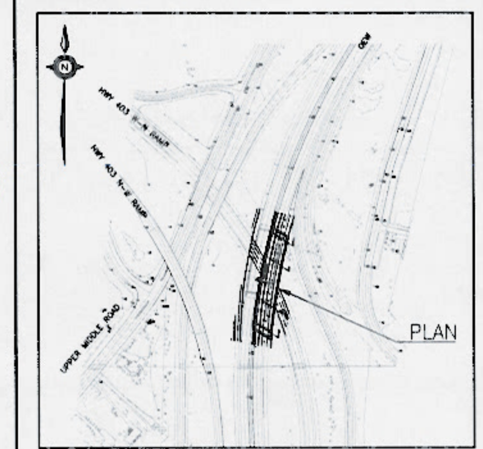
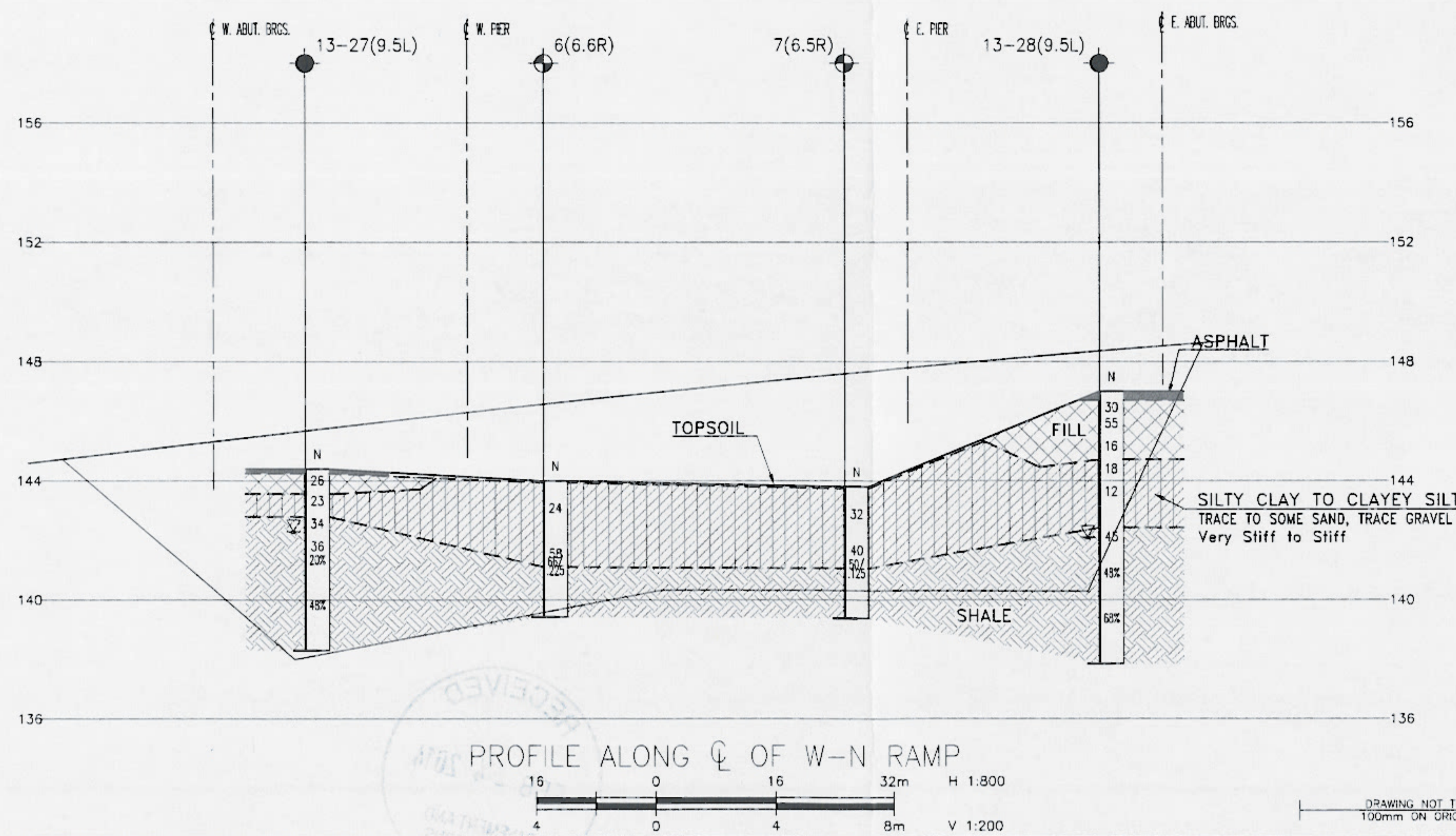
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



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

A. E. GORMAN
Oct 15/13
PROVINCE OF ONTARIO





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
13-27	144.4	4 817 672.3	290 767.7
13-28	147.0	4 817 778.0	290 786.7

-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. 30M5-299

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\lesl184-PlanProfile-315-1(Hwy403-QEW W-N Ramp).dwg
PLOTDATE: 10/11/2013 5:16 PM