

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
SERVICE ROAD OVER DUNDAS STREET EAST
QUEEN ELIZABETH WAY/HIGHWAY 403 IMPROVEMENTS
OAKVILLE, ONTARIO**

W.O. 07-20009

Geocres Number: 30M5-295

Report to

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Service Road @ Dundas Street\Final\Service Road over
Dundas St - FIDR.doc

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SERVICE ROAD OVER DUNDAS STREET EAST
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W.O. 09-20007

Geocres Number: 30M5-295

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 a new service road over Dundas Street East, just west of the Highway 403 in the Town of Oakville, Ontario. This investigation is part of the Queen Elizabeth Way (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 will be located at Dundas Street, approximately 250 m west of Highway 403. The lands surrounding the structure site are primarily undeveloped fields. There is a private residence north of the structure site and a commercial yard immediately to the southwest of the structure site.

The land adjacent to the site has a gently rolling topography, generally sloping down to the southeast toward Lake Ontario. To the west of the site, the ground slopes down for 600 m to the valley of Joshua Creek which meanders within a steep sided valley with a flood plain approximately 30 m wide.

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 6 and 7, 2013. A total of two boreholes were drilled and sampled at this site, identified as BH 13-01 and 13-02. Borehole 13-01 was drilled near the proposed south abutment while Borehole 13-02 was drilled near the proposed north abutment. The borehole depths ranged from 10.1 to 10.4 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 Dundas Street East.

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 in the open boreholes were recorded just prior to introducing water in the boreholes during the coring operations. A standpipe piezometer, consisting of 25 mm diameter PVC pipe with slotted screen, was installed in both boreholes to monitor groundwater levels after drilling. The installation details of the piezometers are summarised in Table 3.1.

Table 3.1 –Piezometer Installation Details

Borehole	Tip Position		Piezometer Installation Details
	Depth (m)	Elev. (m)	
13-01	10.4	161.1	Sand filter from 10.4 m to 8.5 m and bentonite holeplug from 8.5 m to surface.
13-02	10.1	161.9	Sand filter from 10.1 m to 8.2 m, bentonite holeplug from 8.2 m to 0.3 m, and flushmount protector installed with concrete at 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 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 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 fill (clayey silt or gravelly sand) overlying layers of silt, sandy silt to silty sand, and/or sand till. Shale bedrock was encountered at depth below these soil layers. A thin layer of topsoil was encountered at the surface in Borehole 13-01. More detailed descriptions of the individual strata encountered at the proposed structure site are presented below.

5.1 Topsoil

A thin layer of topsoil (75 mm) was encountered at the surface in Borehole 13-01. The topsoil thickness may vary between and beyond the borehole locations.

5.2 Clayey Silt Fill

Brown clayey silt fill containing some sand and occasional rootlets was encountered below the topsoil in Borehole 13-01.

The clayey silt fill was 0.7 m thick with the lower boundary of the clayey silt fill encountered at a depth of 0.8 m (elevation 170.8 m).

An SPT N-value of 7 blows for 0.3 m penetration was recorded in the cohesive fill, indicating a firm consistency. The moisture content of a single sample of the clayey silt fill was 19%.

5.3 Gravelly Sand Fill

Gravelly sand fill was encountered at surface in Borehole 13-02, which was drilled on the north shoulder of the existing Dundas Street East. The gravelly sand fill was brown in colour and contained trace silt and trace clay.

The gravelly sand fill was 0.9 m thick with the lower boundary encountered at elevation 171.0 m.

A single SPT N-value of 10 blows for 0.3 m penetration was recorded in the gravelly sand fill, indicating a compact relative density. The moisture content of the fill sample collected from Borehole 13-02 was 9%.

5.4 Silt

A native deposit of silt was encountered below the clayey silt fill in Borehole 13-01 and below the gravelly sand fill in Borehole 13-02. The silt was brown in colour and contained trace sand, trace gravel, and trace to some clay. Occasional iron oxide staining was observed on some samples of the silt.

The thickness of the silt deposit ranged from 2.2 m in Borehole 13-01 to 1.4 m in Borehole 13-02. The lower boundary of the silt deposit was encountered at depths of 3.0 m and 2.3 m, respectively (elevation 168.6 and 169.6).

SPT N-values of 15 to 75 blows for 0.3 m penetration were recorded in the silt, indicating a compact to very dense relative density. Moisture contents of samples of the silt ranged from 11 to 15%.

Two samples of the silt were selected for laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these samples are plotted on Figure B1, Appendix B.

Gravel %	0 to 5
Sand %	3 to 7
Silt %	78 to 82
Clay %	10 to 15

5.5 Sandy Silt to Sand

A deposit of sandy silt to sand was encountered below the silt layer in both boreholes. The sandy silt to sand was brown to grey in colour and reddish brown at depth in Borehole 13-02.

In Borehole 13-01 a 1.6 m thick layer of sandy silt was encountered while in Borehole 13-02 a 3.8 m thick layer of sand becoming silty sand was encountered. The lower boundary of this layer was encountered at a depth of 4.6 m in Borehole 13-01 (elevation 167.0 m) and at a depth of 6.1 m in Borehole 13-02 (elevation 165.9 m).

The SPT N-values in this deposit ranged from 18 to 79 indicating a compact to very dense relative density. Moisture contents of the samples of this deposit ranged from 12 to 19%.

Two samples of the sandy silt to sand were selected for grain 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 B2, Appendix B.

Gravel %	0
Sand %	34 to 92
Silt %	49
Clay %	17
Silt & Clay %	8

5.6 Sand Till

Reddish brown sand till containing some gravel and trace silt was encountered below the sandy silt in Borehole 13-01.

The sand till was 1.2 m thick with the lower boundary encountered at a depth of 5.8 m (elevation 165.8 m).

An SPT N-value of 51 blows for 0.3 m penetration was recorded in the sand till, indicating a very dense relative density. The moisture content of one sample of the sand till was 11%.

Glacial till inherently contains cobbles and boulders.

5.7 Shale Bedrock

Bedrock was encountered below the sand till in Borehole 13-01 and below the silty sand in Borehole 13-02. The depths and elevations at which bedrock was encountered in the boreholes are summarized in Table 5.1.

Table 5.1 – Depths and Elevations of Bedrock Surface

Foundation Element	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
North Abutment	13-02	6.1	165.9
South Abutment	13-01	5.8	165.8

The bedrock was described as thinly laminated reddish brown shale containing hard grey limestone interbeds up to 100 mm in thickness. The shale was generally described as highly weathered at the soil-bedrock interface and described as slightly weathered to fresh within 1 to 2 m of the soil-bedrock interface. Frequent horizontal fractures, 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 75 to 100%, indicating a good to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 4.

The average estimated unconfined compression strength (UCS) of the shale, interpreted from point load tests conducted on intact cores, ranged from 3 to 12 MPa, indicating a very weak to weak rock strength classification.

Some of the limestone interbeds were sufficiently thick for point load testing. The estimated UCS of the occasional limestone interbeds ranged from 34 to 42 MPa, indicating a medium strong rock strength classification.

5.8 Water Levels

Water was observed in both open boreholes at a depth of approximately 4.0 to 4.5 m prior to bedrock coring. Upon completion of the coring operations, water was observed at a depth of 2.7 m in Borehole 13-01 and at a depth of 1.2 m in Borehole 13-02. These water levels are not representative of the natural ground water level since water was added to the boreholes during the coring operations.

A standpipe piezometer was installed in each borehole, within the bedrock, to monitor the natural groundwater levels. The water levels measured in the piezometers are as follows.

Table 5.2 – Groundwater Depths and Elevations

Borehole	Date of Reading	Water Level		Comment
		Depth (m)	Elevation	
13-01	May 9, 2013	1.4	170.2	Piezometer
	May 30, 2013	1.9	169.7	Piezometer
	June 26, 2013	1.6	170.0	Piezometer
13-02	May 9, 2013	1.5	170.5	Piezometer
	May 30, 2013	2.0	170.0	Piezometer
	June 26, 2013	2.0	170.0	Piezometer

It should be noted that groundwater 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. from Ajax, Ontario supplied the 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.

Thurber Engineering Ltd.

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Review Principal



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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 at the Service Road over Dundas Street East.

The Service Road will cross over Dundas Street. The proposed finished grade of the Service Road structure will be near Elevations 180.0 and 179.0 at the north and south abutments, respectively. The existing ground surface within the proposed structure is near Elevations 172.5 to 171.2 at the north and south abutments, respectively. Hence, the Service Road approach fills will potentially be 7.5 m to 7.8 m high relative to the existing surrounding grade.

The Dundas Street grade will be approximately at Elevation 171.0 at this location.

The preliminary GA drawing dated April 2013 from McCormick Rankin (MRC) shows that the proposed Service Road Underpass is a single span structure supported by two abutments. The underpass spans a distance of approximately 42.0 m over the six-lane Dundas Street and W-S ramp entrance and the bridge deck is approximately 12.1 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 fill overlying native compact to very dense silt overlying compact to very dense layers of sandy silt to silty sand. The short term groundwater level measured in the piezometers was 2.0 m below the ground surface (Elevation 170.0).

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

- Spread footings bearing on native soil or bedrock
- Spread footings bearing on engineered fill
- Steel H-piles socketed into bedrock
- 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 Native Soil and Bedrock

Spread footings bearing on native soil or bedrock generally are the least expensive form of construction.

The existing fill encountered in Boreholes 13-01 and 13-02 is not considered to be suitable for the support of spread footings and the footings must be placed on the underlying competent native soils.

The design of spread footings bearing on native undisturbed compact to very dense silt or compact to very dense sand to sandy silt or on bedrock must be in accordance with the elevations and bearing resistances given in Table 8.1.

Table 8.1 – Bearing Resistances for Spread Footings

Element	Depth (m)	Elev.	ULS _f (kPa)	SLS (kPa)	Soil/Rock
North Abutment (BH 13-02)	1.5	170.5	600	400	Silt
	6.6	165.4	1000	-	Shale Bedrock
South Abutment (BH 13-01)	1.6	170.0	600	400	Silt
	6.3	165.3	1000	-	Shale Bedrock

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.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm.

Founding elevations presented in Table 8.1 are generally at or below the groundwater levels observed during the investigation. Temporary excavations are required to construct these footings below the water table and 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 native compact to very dense silt, compact to very dense sand to sandy silt or bedrock, ie. all existing overburden must be stripped prior to constructing the engineered fill.

If a footing on engineered fill bearing on native silts and sands or bedrock is used, it may be designed on the basis of the following concentric, vertical geotechnical resistances

- 900 kPa at factored ULS
- 350 kPa at SLS

The engineered fill must be founded on top of the native undisturbed silts and sands or 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% standard proctor maximum dry density (SPMDD) at $\pm 2\%$ optimum moisture content and generally conforming to the geometry 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 CHBDC Clause 6.7.3 and Clause 6.7.4.

Founding elevations presented in Table 8.1 are generally at or below the groundwater levels observed during the investigation. Temporary excavations are required to construct these footings below the water table and local groundwater control will be required to construct the footings in the dry and prevent disturbance of the footing base.

8.3 Steel H-Piles

The soil stratigraphy encountered at this site is considered to be suitable for the support of foundations on driven steel piles.

It is recommended that the H-piles be driven to refusal in the shale bedrock. The anticipated depths and elevations at which the H-piles are expected to meet refusal are given in Table 8.2.

Table 8.2 – Estimated Pile Tip Elevation

Element	Pile Tip Depth (m)	Pile Tip Elevation
North Abutment (BH 13-02)	6.4	165.6
South Abutment (BH 13-01)	6.1	165.5

For preliminary design, the vertical, axial, factored geotechnical resistance at ULS for two pile sections when driven to the bedrock are presented in Table 8.3. The SLS conditions do 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 site investigation and field testing to be completed at that time.

8.4 Augered Caissons (Drilled Shafts)

Caissons socketed into shale bedrock are also suitable for the support of structural loads at this site. To achieve the higher resistances required to make caisson foundations practical, the caissons should be socketed at least 3.0 m into the shale bedrock. The caissons must be installed to the minimum depths and elevations indicated in Table 8.4.

Table 8.4 – Founding Elevations for Caissons

Element	Founding Depth (m)	Founding Elevation
North Abutment (BH 13-02)	9.1	162.9
South Abutment (BH 13-01)	8.8	162.8

Typical preliminary geotechnical resistance has been calculated for the abutments for a range of caisson diameters for a rock socket depth of 3.0 m. The values are shown in Table 8.5.

Table 8.5 – Vertical Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Factored ULS_f (kN)
0.9	3500
1.2	5500
1.5	7500

The vertical geotechnical resistance for caisson foundations has been calculated assuming no contribution from sidewall resistance in the overburden soils. These values include the geotechnical resistance factor of 0.4 as specified in the CHBDC. The SLS conditions will not govern for caissons socketed into bedrock.

The caisson equipment must be able to dislodge and remove any obstructions and ensure that the rock socket is clean of debris. Temporary steel liners should be used to support the sides of the caisson shaft through the soils overlying the bedrock. The liners should be sealed into the bedrock to permit construction in the dry and prevent caving of cohesionless soils into the rock socket. After the liner is sealed into the bedrock a socket must be advanced a minimum of 3.0 m into the bedrock in order to found the caisson on sound bedrock.

8.5 Abutment Design Considerations

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional, semi-integral or integral abutments. Integral abutment design will require the use of steel H-piles.

8.6 Frost Cover

The design depth of frost penetration at this site is 1.2 m. It is recommended that all foundations 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. For the false integral abutment design shown in the preliminary GA, frost protection is not required for the underside of the abutment stem.

8.7 Recommended Foundation

From a geotechnical perspective, and based on current information, the recommended abutment foundation consists of steel H-piles driven to bedrock. A foundation constructed with piles will be able to utilize an integral abutment for the final bridge design.

9 DEWATERING

The highest recommended founding elevations for spread footings on earth are close to the recorded groundwater elevations. Excavations for these footings may penetrate below the groundwater level, depending on final design, and excavations for footings on bedrock will penetrate several metres below the groundwater level.

Accordingly, in preliminary design, it must be assumed that dewatering will be required during construction. The dewatering system must be designed by the Contractor, taking account of the ground conditions, the final design and his planned methods of construction. However, two methods that could be considered are:

- Vacuum wellpoints
- A steel pile cofferdam installed to bedrock, with pumping from sumps inside the cofferdam

10 APPROACH EMBANKMENTS

Based on the current boreholes drilled at the site, the approach embankments will be constructed over compact to very dense silt and compact to very dense silty sand to sandy silt over shale bedrock. The foundation soils are considered to provide adequate stability for approach embankments constructed using SSM or granular fill with a maximum slope of 2H:1V.

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 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.

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.
- The very dense till may contain cobbles and boulders, and the underlying shale bedrock contains hard limestone interbeds. Equipment selected to install caissons must be capable of penetrating these materials.
- Steel liner should be used to advance caissons into the bedrock to support the sidewalls, minimise groundwater inflow, prevent caving of cohesionless soils into the rock socket and enable machine cleaning of the socket base.
- Excavation below the groundwater level will lead to disturbance of the sides and base of the excavation due to sloughing and heaving of the soil caused by the unbalanced head of water. Accordingly, before any excavation is carried out below the groundwater level prevailing at the time of the excavation, the groundwater must be depressed to a level at least 500 mm below the base of the excavation and must be maintained at such level until the excavation has been backfilled to a level above the groundwater level.

13 INVESTIGATION FOR DETAIL DESIGN

During the detailed design phase of this 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.

Thurber Engineering Ltd.

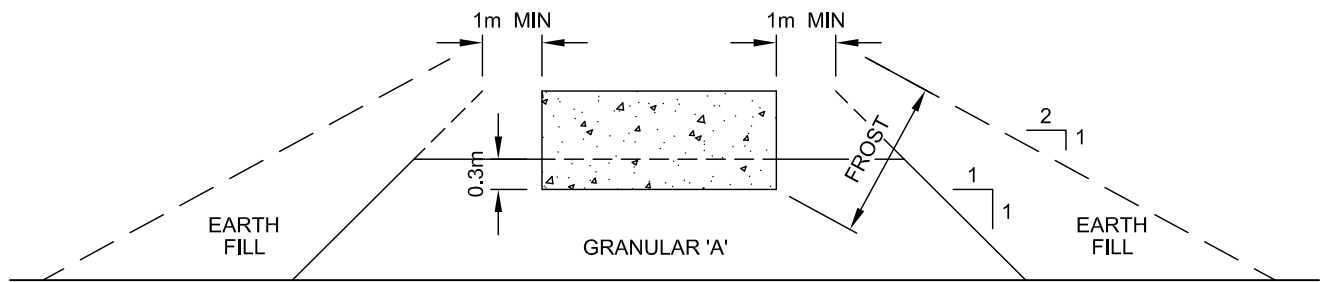
Lukasz Gilarski, P.Eng.
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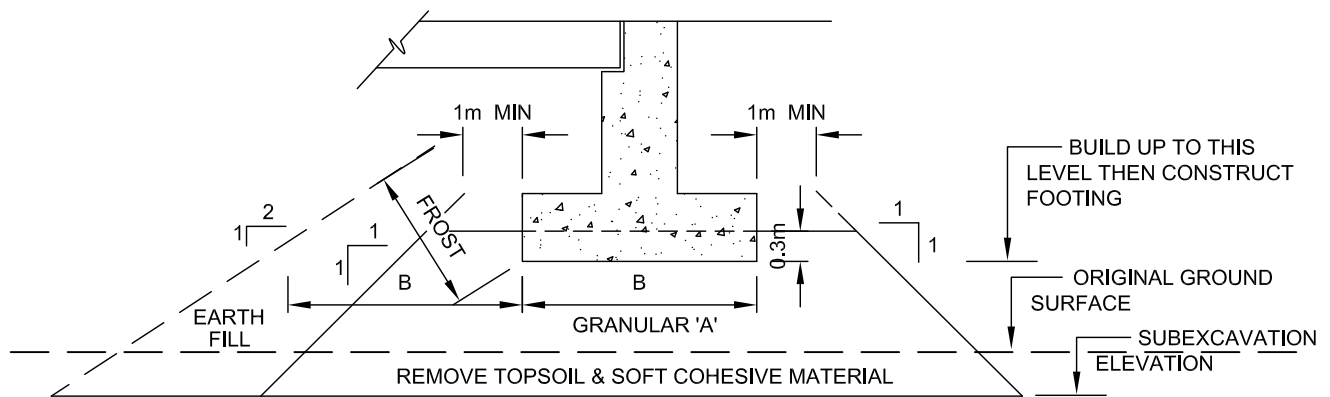


P.K. Chatterji, P.Eng., Ph.D.
Review Principal





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		Field Estimation of Hardness*
	(MPa)	(psi)	
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
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	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 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 13-01

1 OF 2

METRIC

W.P. _____ LOCATION N 4 818 950.4 E 289 049.1 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.07 - 2013.05.07 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE								
								● QUICK TRIAXIAL × LAB VANE								
				WATER CONTENT (%)												
171.6							20	40	60	80	100	20	40	60		
0.0	TOPSOIL: (75mm)															
0.1	Clayey SILT , some sand, occasional rootlets		1	SS	7											
170.8	Firm Brown (FILL)															
0.8	SILT , trace sand, trace clay, trace gravel		2	SS	35											
	Dense to Very Dense															
	Brown		3	SS	75											5 7 78 10
	Damp															
	Occasional iron oxide staining		4	SS	30											
168.6																
3.0	Sandy SILT , some clay		5	SS	18											0 34 49 17
	Compact															
	Grey															
	Moist															
167.0																
4.6	SAND , some gravel, trace silt		6	SS	51											
	Very Dense															
	Reddish Brown															
	Moist (TILL)															
165.8																
5.8	SHALE , highly weathered, reddish brown		7	SS	50/ 0.075											
	Start coring at 7.0m															RUN #1
	Slightly weathered to fresh, thinly bedded, occasional limestone interbeds															TCR=100%
																SCR=93%
																RQD=93%
																UCS=8MPa
																(Average)
	Limestone interbeds (25mm thick) at 7.2m, 7.6m, 8.2m, 8.4m		1	RUN												
	Limestone interbed (75mm) at 7.9m															
	Horizontal fracture at 7.6m and 7.7m															
	Vertical fracture (75mm long) at 7.9m															
	Horizontal fracture at 8.9m, 9.1m, 9.2m, 9.8m		2	RUN												RUN #2
	Highly broken zone (125mm thick) at 8.9m															TCR=100%
																SCR=90%
																RQD=80%
																UCS=12MPa
																(Average)
			</													

Continued Next Page

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

RECORD OF BOREHOLE No 13-01

2 OF 2

METRIC

W.P. _____ LOCATION N 4 818 950.4 E 289 049.1 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.07 - 2013.05.07 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
	Continued From Previous Page		3	RUN													
161.1																RUN #3 TCR=100% SCR=100%	
10.4	END OF BOREHOLE AT 10.4m. BOREHOLE OPEN TO 10.4m AND WATER LEVEL AT 2.7m 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 09/13 1.4 170.2 May 30/13 1.9 169.7 Jun 26/13 1.6 170.0																RQD=100% UCS=7MPa (Average)

RECORD OF BOREHOLE No 13-02

1 OF 2

METRIC

W.P. _____ LOCATION N 4 818 975.4 E 289 028.1 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.06 - 2013.05.06 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT							
							W P W W L							
							20 40 60 80 100							
							20 40 60 80 100							
							20 40 60							
172.0							172							
0.0	Gravelly SAND , trace silt, trace clay Compact Brown Wet (FILL)		1	SS	10									
171.0														
0.9	SILT , some clay, trace sand Compact to Very Dense Brown Damp		2	SS	15		171						0 3 82 15	
	Occasional iron oxide staining		3	SS	74									
							170							
169.6														
2.3	SAND , occasional silt seams Dense to Very Dense Brown Damp		4	SS	49		169							
	Wet		5	SS	56								0 92 8 (SI+CL)	
							168							
167.4														
4.6	Silty SAND Very Dense Reddish Brown Wet		6	SS	79		167							
165.9							166							
6.1	SHALE , highly weathered, reddish brown		7	SS	50/ 0.150									
							165							
	Start coring at 7.0m Slightly weathered to fresh, thinly bedded, occasional grey limestone interbeds													
	Limestone interbeds (25mm thick) at 7.0m, 7.6m Limestone interbed (100mm) at 7.2m Limestone interbed (50mm) at 8.1m		1	RUN			164						RUN #1 TCR=100% SCR=95% RQD=75% UCS=3MPa (Average)	
	Horizontal fractures at 7.1m, 7.2m, 7.3m, 7.5m, 7.7m, 8.8m, 9.1m, 9.2m, 9.5m, 9.6m, 9.7m, 9.9m						163						RUN #2 TCR=100% SCR=95% RQD=85% UCS=6MPa (Average)	
	Clay seam (50mm thick) at 8.7m		2	RUN										
									</					

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-02

2 OF 2

METRIC

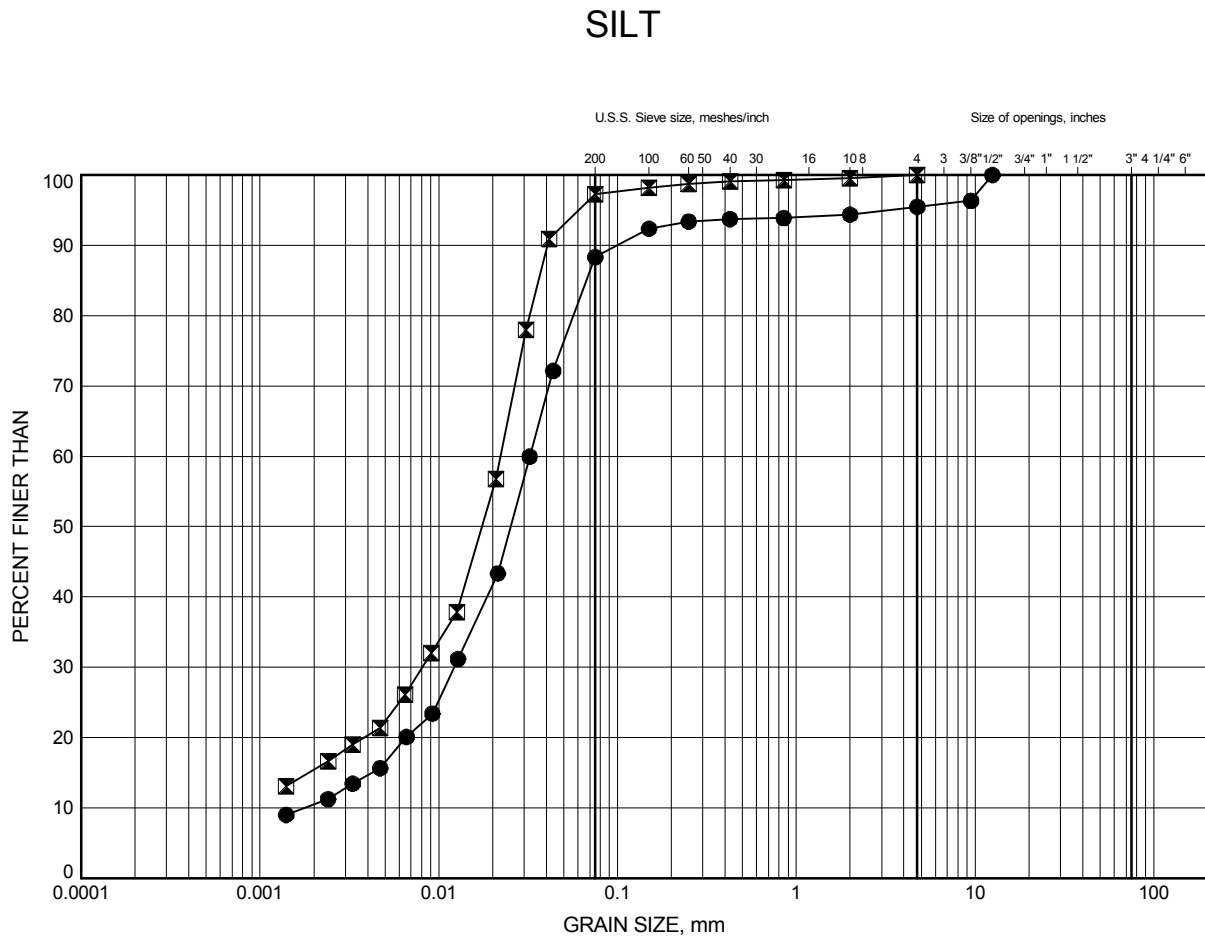
W.P. _____ LOCATION N 4 818 975.4 E 289 028.1 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.06 - 2013.05.06 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
161.9 10.1	END OF BOREHOLE AT 10.0m. BOREHOLE OPEN TO 10.0m AND WATER LEVEL AT 1.2m 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 09/13 1.5 170.5 May 30/13 2.0 170.0 Jun 26/13 2.0 170.0																

Appendix B
Laboratory Test Results

QEW and Hwy 403 GRAIN SIZE DISTRIBUTION

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-01	1.83	169.73
⊠	13-02	1.07	170.89

Date June 2013
W.P.

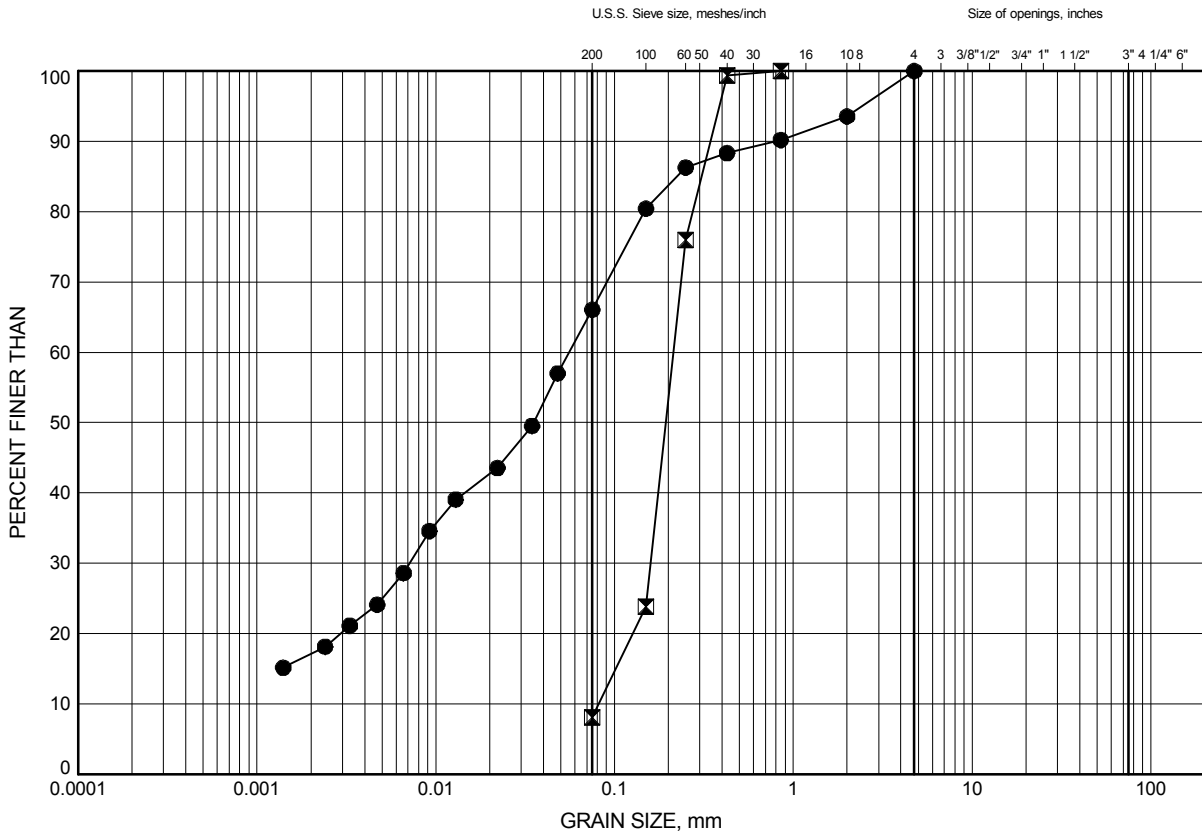


Prep'd AN
Chkd. LB

QEW and Hwy 403
GRAIN SIZE DISTRIBUTION

FIGURE B2

Sandy SILT to SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-01	3.35	168.20
⊠	13-02	3.35	168.60

Date June 2013
W.P.



Prep'd AN
Chkd. LB

Appendix C
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footing on Soil/Rock	Footings on Engineered Fill	Caissons Socketed into Bedrock	Socket Steel H-Piles in Shale Bedrock
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly than deep foundation elements. ii. A footing on rock will have higher geotechnical resistance than footing on native soil. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Economical to install. ii. Accommodates perched abutment. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by socketing caissons into bedrock. ii. Provide uplift and overturning resistance iii. Construction of caissons could continue in freezing weather. iv. Subexcavation of fill and variable material not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher geotechnical resistance than spread footings ii. Provide uplift and overturning resistance. iii. Comparatively short abutment possible. iv. Installation less influenced by weather and groundwater than spread footings. v. Permits integral abutment design.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Dewatering may be difficult depending on depth of excavation. ii. Ineffective for resistance to uplift or overturning. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Dewatering may be difficult depending on depth of excavation. ii. Lower geotechnical resistance than spread footings on bedrock iii. Ineffective for resistance to uplift or overturning. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Difficulty in dewatering, cleaning and inspecting bases. ii. Higher unit cost compared to other foundation options such as spread footings or driven piles. iii. Potential for difficulty in drilling through hard limestone interbeds. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings.
FEASIBLE	FEASIBLE	FEASIBLE	RECOMMENDED

Appendix D
Site Photographs



Photograph 1: Looking west along Dundas Street towards location of proposed overpass.



Photograph 2: Looking east along Dundas Street towards location of proposed overpass.

Appendix E
Borehole Locations and Soil Strata Drawing

