FOUNDATION INVESTIGATION AND DESIGN REPORT
BRONTE ROAD OVERPASS
QEW / BRONTE ROAD INTERCHANGE
OAKVILLE, ONTARIO
G.W.P. 169-00-00, SITE NO. 10-586
GEOCRES Number: 30M5-256

Report to

McCormick Rankin Corporation

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

January 31, 2007
File: 19-1351-63

C:\Thurber Projects 2006\19-1351-63\19135163 QEW Bronte Road Overpass Fdn Inv & Des FINAL Rep jan31 07.doc

TABLE OF CONTENTS

Tables

Appendices

FOUNDATION INVESTIGATION AND DESIGN REPORT

BRONTE ROAD OVERPASS

QEW / BRONTE ROAD INTERCHANGE

OAKVILLE, ONTARIO

G.W.P. 169-00-00, SITE NO. 10-586

GEOCRES Number: 30M5-256

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out at the location of the proposed Bronte Road Overpass structure that will carry the Queen Elizabeth Way (QEW) over the realigned Bronte Road.

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

Thurber was retained by McCormick Rankin Corporation (MRC) to carry out the foundation investigation at this site under the Ministry of Transportation (MTO) Agreement Number 2005-A-000566.

2 SITE DESCRIPTION

The overpass structure site is located on the tableland just east of the Bronte Creek valley. The existing QEW Bronte Creek bridge span and carry the QEW over the creek valley. The existing Bronte Road runs along the east valley slope and passes under the bridge.

The creek valley is incised up to approximately 28 m below the surrounding tableland. The valley slopes are steep and are formed into shale bedrock. On the tableland, shale is also present underneath overburden soils or fill (under the QEW) at shallow depth. It appears that drainage at the site flows towards Bronte Creek, which flows southward to Lake Ontario.

On the north side of the site (north of QEW), the terrain is largely flat with the ground surface varying between Elevations 121 m and 122 m. The ground surface then drops in the order of 4 m to 5 m towards the QEW at approximate Elevation 117 m. On the south side (south of QEW), the terrain is slightly undulating but generally slopes downward towards the southerly portion of Bronte Road situated at about Elevation 108 m. Vegetation is moderate consisting mainly of tall grass, shrubs and occasional small trees.

The project area appears to be located adjacent to the shoreline of the glacial Lake Iroquois. From published geological information, this area is situated at the border between a physiographic region known as the Peel Plain to the north and Iroquois Plain to the south. In this area, the relatively thin native soil deposits typically consist of cohesive soils (some tills) overlying shale bedrock of the Queenston Formation. The till is known to consist of shale and limestone fragments. Glacial lake deposits in the form of stratified silts and sands are present at locations along the shoreline of the glacial lake.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of June 9 to 20, 2006. The field work consisted of drilling and sampling a total of sixteen (16) boreholes to depths ranging from 1.5 m to 15.3 m. The boreholes were numbered 06-1 to 06-16. The approximate locations of all boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The planned borehole locations were staked and/or marked in the field by surveyors from J. D. Barnes Limited. But some boreholes needed to be relocated to avoid sloping grounds and underground utilities. Utility clearance was obtained at all borehole locations by Thurber prior to drilling. The northing and easting co-ordinates and ground surface elevations of the staked boreholes were provided by J.D. Barnes, except for the relocated boreholes where such co-ordinates and elevations were established by Thurber based on the original survey information.

DBW Drilling Limited of Toronto, Ontario supplied a track mounted CME 75 drill rig and a truck mounted D-90 drill rig, and conducted the drilling, sampling and in-situ testing operations for all the boreholes. Auger drilling techniques were used to advance the boreholes through soils and weathered rock. Soil and weathered rock samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Once the top of weathered shale was established and the shale was deemed to be suitable for coring (reasonable recovery anticipated), eight (8) of the boreholes were further advanced a nominal 3 m into bedrock by NQ size rotary coring techniques to recover core samples, except in Borehole 06-13 where rock cores up to the order of 10 m in length were recovered.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. One standpipe piezometer was installed in each of Boreholes 06-1 and 06-13 to permit monitoring of the groundwater level. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screens were installed in the boreholes. The sand screen surrounding the pipe was about 2m to 3 m in length. Bentonite holeplug seals were placed above the sand screen in each installation.

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 rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

On completion of drilling and sampling, the boreholes deeper than 3 m below existing ground surface were grouted with bentonite to the ground surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples including those of weathered shale were subjected to gradation analysis. Atterberg Limits Tests were performed on some of the samples. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point load and Unconfined Compression (UC) testing was carried out at selected locations on the rock cores and the results are shown in Table 1 attached immediately following the text, and on the Records of Boreholes in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these records and on the "Borehole Locations and Soil Strata" drawing in Appendix F. General description of the stratigraphy is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of site conditions.

In general, the site is underlain by relatively thin deposits of overburden soils overlying shale bedrock. At the locations beyond the QEW, the overburden soils range in thickness from about 1 to 4 m. The boreholes drilled on the QEW indicate that the pavement structure (asphalt and granular) is situated directly on bedrock. The overburden generally consists of topsoil/fill, silty clay, clayey silt till and occasional sand and silt layers.

5.1 Topsoil

Topsoil was encountered across the site in eleven boreholes to depths ranging between 50 and 150 mm as shown below.

Borehole	Topsoil Thickness (mm)
06-1	50
06-2	50
06-3	150
06-5	150
06-6	100
06-7	75
06-8	75
06-9	150

06-10	100
06-13	100
06-16	150

Topsoil thickness may vary between and beyond the boreholes.

5.2 Asphalt and Concrete

As part of the existing pavement structure, asphalt was encountered in the five boreholes drilled on the QEW and concrete was encountered below the asphalt in three of those five boreholes. The asphalt thickness ranged between 125 and 305 mm, and the concrete thickness ranged between 75 and 140 mm as shown below.

Borehole	Asphalt Thickness (mm)	Concrete Thickness (mm)
06-4	190	140
06-11	305	125
06-12	125 (upper) 100 (lower)	75 (upper) 125 (lower)
06-14	175	-
06-15	160	-

5.3 Fill

Fill was encountered below the asphalt and/or concrete, and immediately above shale in the five boreholes located on the QEW. The fill consists of sand and gravel, and is visually classified as a 19 mm crusher run limestone. The thickness of the fill ranges from 0.4 m to 0.7 m. The fill is largely in a compact state.

5.4 Sandy Silt to Sand and Silt

A deposit of brown coloured sandy silt to sand and silt was encountered in Boreholes 06-9, 06-13 and 06-16 below the topsoil. This granular deposit ranges between 1.3 m and 2.3 m in thickness, and generally extends to approximate Elevations 119.2 m to 113.6 m. This deposit is in a very loose to compact state with depth, as indicated by SPT 'N' values of 2 to 22 blows per 0.3 m penetration. Figure B1 in Appendix B presents the grain size distribution of samples of sandy silt to sand and silt. The measured moisture contents were in the order of 15% to 20%.

5.5 Silty Clay

In Boreholes 06-1, 06-2, 06-3, 06-5, 06-6, 06-8 and 06-13, the topsoil is underlain by a deposit of silty clay with some sand which extends to depths ranging from 0.7 to 1.3 m below existing ground surface, or approximate Elevations 110.9 m to 113.3 m except at the location of Borehole 06-13 where the base of the silty clay is at Elevation 120.5 m. The silty clay is typically brown to dark brown in colour. Trace topsoil, roots and rootlets were noted in the silty clay. Some limestone fragments and sand seams were also present.

Grain size analyses conducted on two samples retrieved from this unit are presented in Figures B2. These results indicate that this silty clay contains approximately 48% to 76% of silt, and 22% to 30% of clay. Atterberg Limits tests were also conducted on representative samples from this stratum and the results are presented in Figures B6. The silty clay samples had measured plasticity indices ranging between 9% and 19%, and corresponding liquid limits ranging from 23% to 41%, respectively. These values are indicative of a cohesive soil of low to intermediate plasticity (group symbol of CL to CI).

Standard Penetration Tests (SPT) conducted within this deposit gave typical 'N' values ranging from 4 to 30 blows per 0.3 m penetration. Based on these results, the silty clay is considered to have a firm to very stiff consistency. The measured moisture contents of samples recovered from this unit ranged from about 10% to 20%.

5.6 Clayey Silt Till

In Boreholes 06-7, 06-10, 06-13 and 06-16, a clayey silt till deposit underlies topsoil or the upper sands and silts. Where encountered, this till is 0.9 m to 1.8 m in thickness and extends to approximate Elevations 113.3 m to 117.8 m. This till is brown to reddish brown near the ground surface becoming grey in colour below the groundwater level.

Grain size analyses conducted on two samples retrieved from this unit are presented in Figure B3. These results indicate that this clayey silt till contains 30% to 45% of silt and 8% to 18% of clay. Glacial tills are known to contain cobbles and boulders.

Standard Penetration Tests (SPT) conducted within this till gave 'N' values ranging from 11 to 21 blows per 0.3 m penetration, indicating a stiff to very stiff consistency. The measured moisture contents of samples of this till varied from 12% to 18%.

5.7 Shale Bedrock

The overburden soils described above are underlain by weathered shale bedrock. In the boreholes outside of the QEW, the weathered shale was encountered at depths of between 1 m and 4 m below existing ground surface. On the QEW, the weathered shale was present immediately below the pavement structure at between 0.7 m and 0.9 m. Augering and SPT sampling within the weathered shale was carried out in all sixteen boreholes. Relatively higher resistance to augering was encountered at several locations within the weathered shale, inferring the presence of harder limestone interbeds. Bedrock was proven by coring beyond the augered depth in Boreholes 06-1, 06-2, 06-4, 06-7, 06-10, 06-11, 06-13 and 06-15. The following table summarizes the depth to weathered shale encountered at the borehole locations.

Borehole Number	Depth to Weathered Shale (m)	Top of Weathered Shale Elevation (m)
06-1*	1.2	110.9
06-2*	1.0	112.5
06-3	1.3	112.0
06-4*	0.7	116.6
06-5	0.9	113.3
06-6	0.8	112.7
06-7*	0.9	113.3
06-8	1.1	112.0
06-9	1.4	114.5
06-10*	1.8	114.3
06-11*	0.8	116.3
06-12	0.7	116.5
06-13*	3.9	117.3
06-14	0.9	115.9
06-15*	0.7	116.4
06-16	2.9	117.8

* Proved by coring below augered depth

The shale encountered at this site is fine grained, thinly bedded and reddish brown in colour that is typical of the Queenston Formation. The shale is interbedded with hard, grey limestone with occasional siltstone and some clay seams. The shale is typically in a highly to moderately weathered state within the upper 1 m to 2 m. Below this zone, the degree of weathering decreases with depth, and the rock becomes moderately to slightly weathered and occasionally fresh. The hard limestone interbeds typically range from 25 mm to 150 mm in thickness, with occasional layers up to 200 mm to 300 mm.

Total Core Recovery (TCR) of the bedrock was generally between 90% and 100%, with occasional values in the order of 85%. The Rock Quality Designation (RQD) values ranged between the order of 20% and 100%, but most values varied from 60% to 80% indicating a typically fair to good rock quality. The lower RQD values of the order of 20% were associated with the first core run within the upper weathered zone in several boreholes.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5 with occasional values between 6 and 10. Occasional zones of multiple fractures were present within the upper weathered portion of the rock. The discontinuities and bedding planes in the rock cores were largely horizontal with occasional sub-vertical joints.

Point load tests were carried out at selected intervals on the rock cores, and Unconfined Compression (UC) tests were conducted on selected rock cores. The Unconfined Compressive Strengths (UCS) of the rock cores, as inferred from the point load test results and directly measured from UC tests, are summarized in Table 1 attached immediately following the text. The strength profiles with depth are also presented on the Records of Boreholes in Appendix A. Laboratory details of the UC tests are presented in Appendix B.

The test results indicate that the UCS of the cores of shale range from less than 1 MPa to about 8 MPa. The strength of the shaley limestone range between about 5 MPa and 40 MPa. The strength of the very hard limestone interbeds range between 36 MPa and 72 MPa.

Borehole 06-13 penetrated some 12 m into the shale bedrock and terminated at approximate Elevation 106 m. The shale is in a highly to weathered state within the upper 1 m. Below this zone, the shale becomes moderately to slightly weathered. Higher rock strengths are associated with the hard limestone or siltstone interbeds. It is also apparent that the overall strength of the rock mass increases with depth.

5.8 Groundwater Conditions

Groundwater conditions were observed during and upon completion of drilling. Standpipe piezometers were installed and sealed within the bedrock in Boreholes 06-1 and 06-13 to permit longer term groundwater monitoring. To date, one set of piezometric readings has been obtained since completion of installation and is presented in the following table.

Borehole	Date	Ground Surface Elevation (m)	Groundwater	
			Depth (m)	Elevation (m)
06-1	June 28, 2006 July 6, 2006	112.2	4.1 4.2	108.1 108.0
06-13	June 28, 2006 July 6, 2006	121.1	11.3 11.2	109.8 109.9

It is noted that all groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

LICENSED PROFESSIONAL ENGINEER
S. PANG
Jan.31/07
PROVINCE OF ONTARIO

Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.,
Associate, Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
JAN.31/07
PROVINCE OF ONTARIO

Report Reviewed by:
Alastair Gorman, P.Eng.,
Associate, Project Manager

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
31/07
PROVINCE OF ONTARIO

Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

FOUNDATION INVESTIGATION AND DESIGN REPORT
BRONTE ROAD OVERPASS
QEW / BRONTE ROAD INTERCHANGE
OAKVILLE, ONTARIO
G.W.P. 169-00-00, SITE NO. 10-586
GEOCRES Number: 30M5-256

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents foundation design recommendations to assist the design team to select and design a suitable foundation system and approach embankments and cuts for the proposed structure.

It is understood that a new bridge will be constructed to carry the QEW over a realigned Bronte Road. The proposed Bronte Road alignment will be located to the east of the access road (abandoned, old alignment). The proposed Bronte Road will be in a cut that will vary in depth from 12 m below existing ground at the north to 4 m at the south.

Based on the plan and profile drawings provided by MRC, the proposed structure consists of a single span precast concrete girder bridge. A centre to centre span of approximately 43 m in length is to be supported on two abutments. The new Bronte Road alignment is at a skew angle of approximately 18° with the QEW. Under the proposed bridge, the proposed grade of the new Bronte Road varies from approximately Elevations 109.2 to 108 m in a north to south direction. At this location, a cut of up to the order of 8 m depth below existing ground will be required. At the immediate approaches, 3 to 4 m of fill will need to be placed to reach a design QEW grade of approximate Elevation 118 m, resulting in a difference between the road grades in the order of 10m.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained during the course of this investigation.

7 STRUCTURE FOUNDATIONS

The proposed bridge at this site is a single span overpass structure supported on the east and west abutments.

The stratigraphy encountered within the southern portion of the proposed foundation elements consists of 1 m to 2 m of overburden silty clay overlying weathered shale bedrock. In the vicinity of the northern portion of the foundation elements, the overburden increases to the order of 1 m to

4 m in thickness and consists of silty clay, sands and silts, and clayey silt till. The groundwater level is present at approximate Elevation 110 m on the north side of QEW, and at Elevation 108 m on the south side of QEW.

Bedrock elevations, the proposed grades along Bronte Road centreline and other relevant information are presented in the following:

Location	Borehole Number	Elevations (m)		
		Existing Ground Surface	Proposed Bronte Road Centreline Grade	Top of Weathered Shale
West Abutment				
NW Area	06-11*	117.1	≈109.2	116.3
NE Area	06-12	117.2	≈109.2	116.5
Centre Area	06-10*	116.1	≈108.5	114.3
SW Area	06-7*	114.3	≈108.0	113.3
SE Area	06-6	113.5	≈108.0	112.7
East Abutment				
NW Area	06-14	116.8	≈109.2	115.9
NE Area	06-15*	117.1	≈109.2	116.4
Centre Area	06-4*	117.3	≈108.5	116.6
SW Area	06-2*	113.4	≈108.0	112.5
SE Area	06-3	113.2	≈108.0	112.0

* Bedrock proved by coring below augered depth.

The elevations at which bedrock was encountered at the abutment wingwall locations are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Weathered Shale
Northwest Wingwall	06-13	121.1	117.3
Southwest Wingwall	06-8	113.0	112.0
Northeast Wingwall	06-16	120.7	117.8
Southeast Wingwall	06-1	112.2	110.9

The variation of the bedrock surface elevations along the foundation elements at the abutment and wingwall locations are illustrated on the Borehole Locations and Soil Strata Drawing in Appendix F.

The above data indicates that the proposed cut for Bronte Road will be largely through shale bedrock. After the proposed new Bronte Road cut is constructed to the design grade, the entire subgrade will consist of shale bedrock with hard limestone and siltstone interbeds.

7.1 Foundation Alternatives

This section presents discussions on feasible foundation alternatives, recommendations and foundation design parameters for feasible and/or preferred foundation options for this site.

During initial assessment, consideration was given to the following foundation types:

- Spread footings
- Steel H-piles
- Augered caissons (drilled shafts)

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

Given the anticipated presence of bedrock at grade after the Bronte Road cut is formed, it is considered that spread footing foundations on bedrock is a practical option for foundation support for the abutments at this site.

It is however understood that an integral abutment design is currently being considered. Due to the presence of bedrock at the base of the cut, steel H-piles for integral abutment design must be socketted into the bedrock.

Augered caissons are not considered to be practical since footings on bedrock will provide a more cost effective solution.

7.2 Spread Footings on Bedrock

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of the proposed Bronte Road, the new structure can be supported on spread footings founded on shale bedrock with hard limestone interbeds. The required depth of excavation for footing construction on bedrock will vary depending on the final road grade. Based on the proposed Bronte Road grade, the footings may be founded at or below Elevation 108 m near the northerly limit, lowering to or below Elevation 106.5 m near the southerly limit of the structure.

The concrete footings may be constructed directly on shale bedrock. In cases where the underside of a footing is higher than the bedrock subgrade due to over-excavation or otherwise, mass concrete fill with a structural strength of 30 MPa should be used to raise the subgrade to the design footing level.

In most cases, the excavations are anticipated to be formed in near vertical open cuts. Temporary groundwater control will likely involve perimeter ditches and pumping from filtered sumps. Surface drainage should be diverted away from the footing excavations at all times.

All footing excavations should be inspected prior to placing concrete to confirm that the base has been adequately cleaned. All shattered, loosened rock fragments and any clay seams should be removed from the footprint of the footing and replaced with mass concrete fill as necessary. Hand cleaning or air blasting may be required to remove loose rock and softened materials. Shale is prone to rapid deterioration upon exposure to water and air. It is recommended that a working mat of 30 MPa concrete at least 100 mm thick be placed immediately after inspection and approval.

The footings will be founded on the less weathered portion of the shale bedrock. Based on existing information, the footings should be founded at a depth of at least 0.5 m below the final grade of the base of cut.

7.2.1 Bearing Resistance

Footings bearing on shale bedrock below the Bronte Road grade, at or below the above recommended elevations, may be designed using a factored geotechnical resistance at ULS of 1,500 kPa for vertical, concentric loads. Effects of load inclination and eccentricity should be taken into account in accordance with the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4. The design of the footing may be governed by other considerations such as sliding resistance or overturning moment.

The SLS condition will not govern for footings founded on bedrock.

The same value of resistance may be used where mass concrete, including working mats, with a structural strength of 30 MPa is placed in neat contact with a clean, sound bedrock surface.

7.2.2 Horizontal Resistance of Footings

Resistance to lateral forces / sliding resistance between the footing concrete and the bedrock surface should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.7.

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance.

7.2.3 Frost Cover

Although the Queenston Shale is geologically defined as bedrock, it is susceptible to frost action. Therefore, all footings founded on shale must be provided with a minimum 1.2 m of earth cover as frost protection.

7.3 Steel H-Piles

In the interest of achieving an integral abutment design, it is considered feasible to excavate the bedrock to a pre-determined line behind the centreline of the abutment, and to support the abutment on steel H-piles set into sockets that are drilled into bedrock. The sockets should be pre-drilled and the socket base should be cleaned of loose and shattered rock. The pile should then be lowered into the socket and the remaining space grouted with 30 MPa concrete. Based on the proposed Bronte Road grade and a minimum socket depth of 3 m below the final grade, the pile tip elevations are estimated to vary from Elevation 106.2 m or lower near the northerly limit, lowering to Elevation 105 m or lower near the southerly limit of the structure.

7.3.1 Axial Resistance

For a HP 310 x 110 pile grouted within a 600 mm nominal diameter socket extended at least 3 m into shale bedrock below the proposed grade of the Bronte Road cut, a factored geotechnical resistance at ULS of 2,000 kN per pile may be used for design.

The SLS condition does not apply to piles found on bedrock.

The structural resistance of the pile should be reviewed by the structural designer to confirm that the value given above is not exceeded.

7.3.2 Lateral Resistance

For rock sockets formed within the relatively sound shale with limestone interbeds, the ultimate passive force that can be mobilized by the embedded portion of a socket is given by :

$$P_p = 6 \cdot c \cdot D \cdot L$$

where c = 300 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)
D = diameter of socket, m
L = depth of socket in rock, m

The structural designer should check whether a 3 m deep socket is sufficient to provide base fixity.

For integral abutments, the flexibility of the pile can be increased by providing a double or single corrugated steel pipe (CSP) system. Considering that the flexible zone is situated in a narrow zone of compacted fill between a rock face and a RSS wall facing, the double concentric pipe configuration is more suitable. The double concentric pipe liner involves the use of 600 mm and 800 mm diameter CSP pipes with a minimum of 3 m below the underside of the abutment. The space between the two pipes is left unfilled. The sand for filling the annular void between the H-pile and the inner CSP should be backfilled with uniformly graded sand meeting the gradation requirements presented in Table D1 in Appendix D.

7.3.3 Pile Installation

The pre-drilled holes for forming the pile socket should have a nominal diameter of 600mm.

The contract documents should contain an NSSP alerting the contract bidders to the fact that the bedrock contains hard layers and that coring and/or rock breaking equipment may be required to form the sockets. Suggested wording for this NSSP is provided in Appendix E. General reference can also be made to SP 903S01.

Subsequent to the seating of the piles in the socket, the remaining space in the pre-drilled holes should be grouted with 30 MPa concrete.

7.3.4 Frost Protection

Frost protection is not required for pile caps formed on free draining fill.

8 EXCAVATION AND BACKFILL

8.1 General

All temporary excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the overburden silty clay, clayey silt till and the sand and silt deposits may be classified as Type 3 soils. The upper 2 m (weathered portion) of the shale may be classified as a Type 2 material.

Vertical sides of temporary excavations formed in the Queenston Shale bedrock below the upper weathered zone should be stable on the short term, i.e. no more than one week. If the excavation is not backfilled within that time, the shale may deteriorate and slough into the excavation. Accordingly, excavated shale faces that are expected to be exposed for more than one week should be sloped or provided with support in accordance with OHSA and O.Reg. 213/91.

8.2 Foundations and Road Cut

All excavations for footing construction will be carried out through shale with hard limestone and siltstone interbeds. The excavation and backfilling for foundations should be carried out in accordance with SP 902S01. This Special Provision is listed in Appendix E.

The proposed Bronte Road will involve both horizontal and vertical realignment. A permanent cut into relatively thin layers of soil and into the underlying predominantly shale with hard limestone and siltstone interbeds will be required.

8.3 Earth Excavation

The extent of earth excavation will be relatively small at this site. For formation of the cut, the excavation will extend through 1 m to 4 m of silty clay, clayey silt till, sand and silt materials. Cobbles and boulders may be encountered in the overburden soils. It is anticipated that temporary excavations through these soils may be carried out with side slopes not steeper than 1H: 1V. Flatter slopes may be required at the locations where the soils are less competent than what is assumed during design or where water seepage affects surficial stability.

8.4 Roadway Protection

It is anticipated that roadway protection will be required during construction. An item titled "Protection System" as per SP 105S19 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 and the alignment of the shoring be specified for this site.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall. Due to the presence of shallow bedrock, the soldier piles will need to be installed through pre-drilled holes and socketted into bedrock in order to develop the required fixity. It is anticipated that the shoring system may be stiffened by cross bracings, where applicable. Anchors may also be required at some locations.

For a temporary braced soldier pile and lagging wall, the lateral pressure diagram as shown in Figure D2 in Appendix D may be used for design using the parameter values shown below.

γ	=	20 kN/m^3
γ_w	=	10 kN/m^3
K_a	=	0.31 (road embankment fill)
	=	0.33 (sandy silt to silty sand)
	=	0.37 (silty clay and clayey silt till, and weathered shale)
h_w	=	0 (assuming no hydrostatic pressure build-up behind a presumably permeable wall)
H	=	depth to base of excavation (rock surface) (m)

The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is given by the expression in Section 7.3.2.

It should be pointed out that the actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall. These factors should also be considered when designing the shoring system. All shoring systems should be designed by a Professional Engineer experienced in such designs.

8.5 Rock Excavation

The bulk of the excavation to form the permanent cut will be extended into the relatively sound shale with hard limestone and siltstone interbeds. Heavy excavating equipment, ripping machinery and rock breakers/splitters may be required to break up hard limestone and other intact rock slabs. The contract documents should contain an NSSP alerting the contract bidders that rock excavation may require the use of such equipment. Suggested wordings for this NSSP is provided in Appendix E.

Any rock excavation should be carried out in accordance with the Special Provision, Amendment to OPSS 120, 1994 listed in Appendix E.

Where the excavation is extended into harder and less weathered rock and if blasting is permitted, the Contractor may elect to use carefully controlled drill (line drilling) and blast techniques to enhance a neat excavation line and minimize face instabilities. The design of the blast and removal procedures should be the responsibility of the Contractor. However, it is important that the Contractor's procedures incorporate methods of avoiding damage to the founding surfaces and nearby roads and structures. Any damage to the founding surfaces on bedrock must be adequately repaired prior to constructing the foundation.

The Contractor's blasting and monitoring plan must not result in damage to any nearby roadways, buildings or other structures such as the QEW bridge over Bronte Creek. The contract documents should alert the contractor of any such structures. During construction, it is recommended that the Contract Administrator retain the services of blasting and rock mechanics specialists to examine and assess the Contractor's procedures prior to approving them.

8.6 Permanent Cut

Immediately to the north of the QEW, the cut will be up to 11 m below the existing grade. In the vicinity of the QEW, the cut will be in the order of 9 m below existing road grade. The proposed Bronte Road grade at the structure location varies from approximate Elevations 109.2 to 108 m in a north to south direction. Existing information indicates that the groundwater table is at depth of 3 m or more below the top of weathered shale, or within 1 m above the proposed Bronte Road grade. A combination of vertical cuts and inclined slopes may be used. Approximately 5 m of rock will be exposed below the underside of the abutment stem.

Vertical Cuts

Consideration may be given to creating vertically sided cuts at the abutment locations in order to limit the quantity of excavation. The face of the rock excavation should lie approximately 200 mm behind the back face of the abutment. For erosion protection, aesthetic and other considerations, it is recommended that the rock face be protected by a concrete facing. Typically, this facing should be formed flush with the front face of the abutment and poured neat against the rock face, except for the provision of drainage. The concrete facing will necessarily encase the CSPs surrounding the piles, but must be separated from the abutment ballast wall so that movement of the structure will not result in loading of the concrete facing.

From a geotechnical standpoint, "passive" rock anchors (i.e. rock dowels that are grouted but not prestressed), resin grouted rock bolts (such as those supplied by DSI or Williams) or equivalent, may be used to maintain contact between the concrete and the rock mass.

It is anticipated that the rock dowels/bolts will not be required to sustain appreciable loading. Due to the presence of horizontal bedding planes in the shale, however, it is considered prudent that all of the dowels/bolts be installed sub-horizontally to minimize the unlikely event of "pull-out".

For planning and design purposes, a single row of dowels/bolts with a nominal grouted length within the rock mass of 2 m may be assumed for cut heights of up to 5 m. For higher cuts, the grouted length may be increased to 3 m or a second row of dowels/bolts should be installed. It is recommended that the proprietary suppliers be consulted for further details of such application of their products.

Corrosion protection schemes, such as those recommended by the proprietary rock anchor/bolt suppliers, must be implemented.

It is understood that consideration has been given to minimizing the extent of RSS wingwalls at the abutments. This can be achieved by reducing the length of the inclined open cut slopes along the realigned Bronte Road. Instead of the inclined slopes, the vertically sided cuts with concrete facings, as outlined above, may be extended beyond the abutment locations. Along cut sections beyond the abutments, the thickness of the concrete facing may be reduced to the order of 400 mm to 500 mm. These areas must be provided with rock dowels/bolts as described above.

Prior to placing the concrete facing, the exposed rock slopes should be scaled and any loose rock fragments should be removed. Some form of drainage should be provided behind the concrete facing. Consideration may be given to placing proprietary strip drains on the rock face prior to forming the concrete facing. For positive drainage from the back side of the concrete facing, these strip drains should be connected to the drainage system along the realigned Bronte Road.

Inclined Slopes

Permanent cut slopes through the overburden soils and the shale bedrock should be formed at inclination not steeper than 2H : 1V.

Slopes formed at inclinations not steeper than 2H : 1V will be stable from a global stability perspective. However, exposed shale will be subject to weathering and surficial sloughing may occur unless the slopes are protected, or until adequate vegetation growth has been established.

Vegetation cover should be established on all exposed soil and shale bedrock slopes to protect against surficial erosion. General reference may be made to the latest revision of OPSS 572 and related special provision(s) for more detailed requirements, where applicable.

9 GROUNDWATER CONTROL

9.1 General

Permanent Cut

In the immediate vicinity of the proposed overpass bridge, the groundwater level ranges between Elevations 108 m and 110 m, and the proposed grade of the new Bronte Road will be at or within 1 m below the groundwater level. Any groundwater seepage from perched water in the silty clay and clayey silt till is anticipated to be small. As excavation for the cut proceeds, however, seepage from the more permeable sands and silts, fractures in the rock, and surface water accumulating within the cut must be controlled and drained. Gravity drainage or unwatering systems must be installed to allow the excavation work to be carried out in the dry. It is expected that the rate of seepage will decrease over time as the local groundwater table is drawn down. Continued seepage is expected in the long term and a permanent drainage system must be designed and implemented.

Temporary Excavations for Footing Construction

If spread footing foundations are used, temporary excavations for footing construction are anticipated to extend to the order of 1.5 to 2 m below the groundwater level. The footing bases must be kept dry as shale is prone to rapid deterioration upon exposure to water and air. Accordingly, unwatering systems must be installed to allow footing construction to be carried out in the dry.

9.2 Temporary Drainage

The design of temporary groundwater control systems is the responsibility of the Contractor. Some unwatering methods that are considered feasible for this site include the following:

- Within the cut, drainage ditches supplemented by pumping from filtered sumps can be used to control groundwater seepage, surface runoff and precipitation. Surface runoff should be diverted away from the cut at all times.
- For footing construction, groundwater control measures such as perimeter ditches and pumping from filtered sumps should be implemented to remove water

accumulated at the footing base prior to placing concrete. Concrete must be placed in the dry.

It is understood that an assessment to determine whether an MOE Permit to Take Water (PTTW) is required for this work is being carried out by others.

9.3 Permanent Drainage

A permanent drainage system will be required to remove groundwater seeping from slope faces and the subgrade, and from precipitation. Water seepage behind vertical and near vertical rock cuts with concrete facings has been addressed in the previous Section 8.6. Some drainage measures that are considered feasible for inclined slopes with no concrete facings and for the new roadway include the following:

- Assuming an urban cross-section, longitudinal subdrains connected to a gravity outlet or to a pump station can be installed on both sides of the roadway.
- Gravel sheeting should be used in areas where seepage occurs from inclined slopes without concrete facings.

10 APPROACH EMBANKMENTS

The approach embankments for this structure will be constructed on overburden soils, and directly on bedrock near the abutments. The embankments are expected to be generally 2 to 3 m high except up to 9 m (Elevation 117 m) above final Bronte Road grade (Elevation 108 m) immediately behind the abutments. The shale bedrock and overburden will satisfactorily support the approach fills at this site assuming that all topsoil, organics, disturbed or softened/loosened materials will be removed from the footprints of the embankments prior to placing fill.

It is recommended that the same granular materials recommended for use as backfill to the abutments (see Section 12) be used to construct the approach fills. Excavated shale should not be used for approach fill construction.

Embankment side slopes constructed using engineered granular materials resting on a competent shale bedrock or overburden subgrade will be stable at inclinations not steeper than 2H : 1V. Factors of Safety of greater than 1.3 against global instability will be maintained.

Anticipated foundation settlement due to compression of the foundation shale should be negligible. For compacted granular fill, settlement due to embankment compression can be up to 0.5% of the embankment height, or approximately 45 mm for 9 m high fills. This settlement should be complete by the end of construction.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and listed in Appendix E.

The approach embankments will be resting directly on shale and overburden, which has no potential for liquefaction. Consequently, the foundation of the approach embankments will be stable with respect to seismic activities at this site.

Earth fill slopes must be provided with erosion protection in accordance with OPSS 572 and related special provision(s).

11 RETAINED SOIL SYSTEMS

It is understood that Retained Soil System (RSS) walls will be used at this site. Given the presence of bedrock at grade after the cut is formed, the use of these walls is considered feasible.

RSS walls should be specified to be "High Performance" and "High Appearance". The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

11.1 Foundation

In general, the levelling pad for the RSS wall may be founded directly on bedrock at the elevations recommended below. Mass concrete may be used to the raise the grade in case of over-excavation.

The founding elevations for the RSS walls are as follows:

West Abutment, Northwest and Northeast Wingwalls

- Elevation 109 m or below near the northerly limit to Elevation 108 m or below near the southerly limit.

East Abutment, Southwest and Southeast Wingwalls

- Elevation 109 m or below near the northerly limit to Elevation 108 m or below near the southerly limit.

The reinforced earth block of the RSS walls may also be founded directly on the shale bedrock.

The following values may be used for the foundation design of an RSS wall:

- Factored geotechnical resistance at ULS of 1,500 kPa for walls founded directly on the shale bedrock (SLS is not applicable for foundations on rock) at the above elevations.
- Ultimate coefficient of friction between cast in-situ concrete and shale bedrock is 0.7.
- Ultimate coefficient of friction between cast in-situ concrete levelling pad on Granular A is 0.7.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

11.2 Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the geometry of the embankment slopes.

It is envisaged that the main body of the RSS will be founded directly on shale bedrock.

RSS walls founded directly on shale will have a F.S. of at least 1.3 against global failure.

The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

For walls founded on bedrock, settlements will be negligible.

12 BACKFILL TO ABUTMENTS

Granular materials such as those meeting OPSS Granular A or B requirements should be used as backfill to the abutment walls and wingwalls.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993".

Excavated shale is prone to deterioration and is difficult to compact adequately. Therefore, excavated shale must not be used as backfill to the abutments.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.06. The design of the abutment must incorporate a subdrain as shown on OPSD 3501.000.

13 STATIC EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000, as recommended, the static lateral earth pressure will be governed by the properties of the material within the backfill limits shown, i.e. a line projected up at 1.5H:1V for granular backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

For fully drained backfill, earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC, but are generally given by the following expression:

$$P_h = K(\gamma h + q)$$

where P_h = horizontal pressure on the wall (kPa)
K = earth pressure coefficient (see below)
γ = unit weight of retained soil (see below)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical unfactored values are shown in the following table.

Wall Condition	**Earth Pressure Coefficient (K)**			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°; \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°; \gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil)	3.70	-	3.30	-

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters are provided in Table A3.1.7 of the CHBDC for the Oakville area:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05

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

14.2 Liquefaction Potential

There is no potential for liquefaction for bridge foundations on bedrock.

The immediate approach embankments will be founded directly on bedrock and, therefore, there is negligible potential for liquefaction below the embankments. The embankments themselves will be constructed above groundwater level that is expected to be maintained below the final Bronte Road grade and are, therefore, not considered to be in danger of liquefaction.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

However, results of calculations for these dynamic earth pressure coefficients, using an angle of friction between the wall and backfill material of 0.5ϕ, indicate that these values are comparable to the static earth pressure coefficients. As such, it is recommended that the static values recommended in Section 13 be used for seismic considerations as well.

15 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- if blasting is to be employed, that experts in this field should be retained to review the contractor's proposed work procedures, to monitor the contractor's work, and to ensure that there will be negligible adverse effects on nearby highways, roads, structures and buried utilities
- rock mechanics specialists who should be retained to inspect all exposed rock slopes and faces, regardless of the methods of excavation, to confirm stability of the rock slopes
- potential disturbance of the bedrock under the foundations due to blasting or other excavation procedures
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation
- the presence of shattered rock during excavation for footing construction.

LICENSED PROFESSIONAL ENGINEER
Sydney Pang
S. PANG
Jan. 31/07
PROVINCE OF ONTARIO

Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.,
Associate, Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
JAN 31/07
PROVINCE OF ONTARIO

Report Reviewed by:
Alastair Gorman, P.Eng.,
Associate, Project Manager

LICENSED PROFESSIONAL ENGINEER
PKChatterji
P. K. CHATTERJI
Jan 31/07
PROVINCE OF ONTARIO

Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

TABLE 1 - Point Load and Unconfined Compressive Strength Test Results
QEW / Bronte Road Interchange

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type	
	06-1						
2			**3.55**	**-**	**1.3**	**shale**	**UC Test**
	13	4	4.06	2.58	61.84	limestone	
	14	3	4.34	0.03	0.80	shale	
3			**5.05**	**-**	**5.60**	**shale**	**UC Test**
			5.40	**-**	**4.60**	**shale**	**UC Test**
	18	0	5.49	0.04	0.99	shale	
	19	4	5.89	0.11	2.64	shale	

Total Rock Core

Average	Minimum	Maximum	
11	1	62	MPa

Run #	Average
2	21.31
3	3.46

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-2					
2	17	3	5.26	0.03	0.83	shale

Total Rock Core

Run #	Average	
2	0.83	MPa

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-4					
2	10	2	3.10	0.04	0.86	shale
	12	2	3.71	0.11	2.59	shale
	12	10	3.91	0.04	0.86	shale
	13	9	4.19	0.96	22.94	shale/limestone
	14	2	4.32	1.54	36.89	limestone

Total Rock Core

Average	Minimum	Maximum	
13	1	37	MPa

Run #	Average
2	12.83

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-7					
1	10	7	3.23	3.46	82.94	limestone

Total Rock Core

Run #	Average	
1	82.94	MPa

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-10					
1	9	4	2.84	2.76	66.14	limestone
2	11	7	3.53	0.04	0.84	shale
	12	5	3.78	0.03	0.81	shale
3	13	10	4.22	0.17	4.10	shale
	15	6	4.72	0.04	1.04	shale
	15	6	4.72	0.04	0.85	shale
	16	11	5.16	0.10	2.46	shale
	18	3	5.56	0.34	8.23	shale/limestone

Total Rock Core

Average	Minimum	Maximum	
11	1	66	MPa

Run #	Average
1	66.14
2	0.83
3	3.34

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-11					
1	6	8	2.03	0.49	11.83	shale/limestone
	7	4	2.24	0.44	10.66	shale/limestone
2	10	9	3.28	0.04	0.86	shale
	11	2	3.40	0.03	0.74	shale
	12	5	3.78	0.56	13.54	shale/limestone

Total Rock Core

Average	Minimum	Maximum	
8	1	14	MPa

Run #	Average
1	11.24
2	5.05

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type	
	06-13						
1	16	0	4.88	0.04	1.04	shale/limestone	
	16	0	4.88	0.68	16.39	shale/limestone	
	17	6	5.33	3.02	72.57	limestone	
	17	6	5.33	2.73	65.43	limestone	
			5.50	**-**	**5.60**	**shale/limestone**	**UC Test**
	19	0	5.79	1.73	41.47	limestone	
	19	0	5.79	0.69	16.53	shale/limestone	
	20	6	6.25	0.07	1.58	shale	
2	21	11	6.68	0.04	1.04	shale	
	23	2	7.06	0.10	2.45	shale	
	25	0	7.62	0.03	0.82	shale	
3	26	4	8.03	0.03	0.81	shale/limestone	
	28	5	8.66	0.04	1.04	shale/limestone	
	28	5	8.66	1.56	37.34	limestone	
	30	5	9.27	0.09	2.12	shale/limestone	

Total Rock Core

Average	Minimum	Maximum	
11	1	73	MPa

Run #	Average
1	27.58
2	1.44
3	10.33

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Type	
	06-13						
4	31	11	9.73	0.04	1.04	shale	
			9.80	**-**	**12.90**	**shale/limestone**	**UC Test**
	33	6	10.21	0.08	2.00	shale	
	33	6	10.21	0.04	1.04	shale	
	34	10	10.62	0.17	4.02	shale	
5	37	0	11.28	0.68	16.23	shale/limestone	
			11.60	**-**	**7.60**	**shale/limestone**	**UC Test**
	38	8	11.79	0.17	4.12	shale	
			12.30	**-**	**5.40**	**shale**	**UC Test**
6	41	4	12.60	0.17	4.14	shale	
	42	9	13.03	0.34	8.25	shale	
	44	10	13.67	0.09	2.14	shale	
7	46	6	14.17	0.04	1.04	shale	
	46	6	14.17	0.08	2.00	shale	
	47	7	14.50	0.04	1.04	shale	
	47	7	14.50	0.17	4.02	shale/limestone	
	49	6	15.09	0.68	16.23	shale/limestone	

Run #	Average
4	4.20
5	8.34
6	4.84
7	4.86

Run	Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)	Rock Type
	06-15					
	5	7	1.70	0.19	4.44	shale
	6	0	1.83	0.22	5.18	shale
	12	3	3.73	0.22	5.30	shale

Total Rock Core

Average	Minimum	Maximum	
5	4	5	MPa

Run #	Average
1	4.81
2	5.30

Appendix A

Record of Borehole Sheets

THURBER

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. 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 the naked eye

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 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	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample
WS Wash Sample
AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample
TP Thin Wall Piston Sample
PH Sampler Advanced by Hydraulic Pressure
PM Sampler Advanced by Manual Pressure
WH Sampler Advanced by Self Static Weight
RC Rock Core
SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▽ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value — Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT — Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

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			

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.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

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

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 percentage 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.

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-1

METRIC

W.P. 169-00-00 | LOCATION N 4 807 835.7 E 285 411.3 (Bronte) | ORIGINATED BY SLL

HWY QEW | BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring | COMPILED BY JHL

DATUM Geodetic | DATE 13.06.06 - 13.06.06 | CHECKED BY SKP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
112.2									
0.0	TOPSOIL: (50 mm)				112				
	Silty **CLAY**, some sand, trace root and rootlets Firm to Very Stiff Reddish Brown Moist	1	SS	6		2	21	47	30
		2	SS	30					
110.9 1.2	**SHALE**, weathered, with grey limestone interbeds Reddish Brown				111				
		3	SS	79		0	7	68	25
		4	SS	66/ 275	110				
109.5 2.7	END OF SPT AND AUGERING AT 2.72 m. RECORD OF BOREHOLE CONTINUES ON 06-1R								

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/m^3

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

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

RECORD OF BOREHOLE 06-1R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 13 June 2006
COMPLETED : 13 June 2006
INCLINATION: Vertical AZIMUTH:
SHEET 1 OF 1
DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN; F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED; SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR; FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	FIELD/LABORATORY TESTING RESULTS
	RUN	SHALE BEDROCK, moderately becoming slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey, strong LIMESTONE interbeds, and occasional clay seams Clay seam (25 mm) at 2.79 m Limestone (<25 mm) at 2.82 m		1	0.091	100		Bentonite
4	RUN	Limestone (50 mm) at 3.51, 4.09 m Limestone (25 mm) at 4.25 m Clay seam (<25 mm)		2	0.090	100	Subvertical joint 4.22 to 4.27 m	108.06 Sand Filter 107.60
6	RUN	Limestone (50 mm) at 4.82 m Clay seam (<25 mm) Limestone (75 mm) at 5.20, 5.31 m Limestone (25 mm) at 5.84, 6.04 m	105.88 6.30	3	0.095	100		Slotted Screen 106.08
8 10 12		END OF BOREHOLE AT 6.30 m. BOREHOLE GROUTED TO SURFACE UPON COMPLETION OF PIEZOMETER INSTALLATION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Jun 28/06	4.1	108.1
Jul 6/06	4.2	108.0

Column headers also include: SYMBOLIC LOG; RECOVERY (TOTAL CORE % / SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER .3 m 5 10 15 20; DIP w.r.t. Core Axis 0 30 60 90; HYDRAULIC CONDUCTIVITY k, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; 25 Unconfined 50 Compressive Strength 75 (MPa); ● Point Load Test Diametral; ▲ Point Load Test Axial; ⌧ Laborotory UCS Test

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION WATER LEVEL (date)

DEEP/DUAL INSTALLATION WATER LEVEL (date)

LOGGED : SLL
CHECKED : SKP

THURBER

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-2

METRIC

W.P. 169-00-00 LOCATION N 4 807 834.0 E 285 390.6 (Bronte) ORIGINATED BY SLL

HWY QEW BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY JHL

DATUM Geodetic DATE 12.06.06 - 12.06.06 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
113.4 / 0.0	TOPSOIL: (50 mm)				
	Silty **CLAY**, trace sand seams, topsoil stained, trace roots and rootlets. Firm. Dark Brown. Moist	1	SS	5	113
112.5 / 1.0	**SHALE**, weathered, with grey siltstone layer. Reddish Brown	2	SS	27	112
		3	SS	97/ .275	
111.1 / 2.4	END OF SPT AND AUGERING AT 2.37 m. RECORD OF BOREHOLE CONTINUES ON 06-2R	4	SS	50/ .075	

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

+ ³, × ³: Numbers refer to Sensitivity

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

ONTMT4S 5163.GPJ 12/07/06

RECORD OF BOREHOLE 06-2R

PROJECT : QEW / Bronte Road Interchange
LOCATION : Oakville, Ontario
STARTED : 12 June 2006
COMPLETED : 12 June 2006

PROJECT NO. 169-00-00

SHEET 1 OF 1

INCLINATION: Vertical AZIMUTH:

DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⊠ Laborotory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATRATION RATE (m/min)	FLUSH COLOUR % RETURN
		SHALE BEDROCK, moderately to slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds, and occasional clay seams				
	RUN	Silt seam (50 mm) at 3.23 m		1	0.085	100
		Clay seam (50 mm) at 3.51 m				
		Limestone layer (25 mm) at 3.71 m				
4						
		Limestone layer (25 mm) at 4.27 m Siltstone layer (125 mm) at 4.32 m				
	RUN	Limestone layer (25 mm) at 4.88 m		2	0.092	100
		Limestone layer (125 mm) at 5.47 m	107.85			
		END OF BOREHOLE AT 5.59 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.	5.59			
6						
8						
10						
12						

RECOVERY: TOTAL CORE %, SOLID CORE %; R.Q.D. %; FRACT. INDEX PER .3 m; DISCONTINUITY DATA: DIP w.r.t. Core Axis, TYPE AND SURFACE DESCRIPTION; HYDRAULIC CONDUCTIVITY k, cm/sec; Unconfined Compressive Strength (MPa)

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION — WATER LEVEL (date)

DEEP/DUAL INSTALLATION — WATER LEVEL (date)

LOGGED : SLL

CHECKED : SKP

THURBER

ROCKM 5163.GPJ 12/07/06

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-3

METRIC

W.P. 169-00-00	LOCATION N 4 807 847.0 E 285 398.0 (Bronte)		ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers		COMPILED BY JHL
DATUM Geodetic	DATE 12.06.06 - 12.06.06		CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
113.2									
0.0	TOPSOIL: (150 mm)								
0.2	Silty **CLAY**, trace sand, topsoil stained Firm to Very Stiff Dark Brown to Brown Moist (TILL)	1	SS	6	○				
		2	SS	28	○	0	2	76	22
112.0 / 1.3	**SHALE**, weathered, with grey siltstone layer Reddish Brown	3	SS	90/ 250	○	0	1	84	15
		4	SS	50/ 125	○				
	Siltstone layer at 3.13 to 3.30 m	5	SS	50/ 100	○				
		6	SS	50/ 150	○				
108.6 / 4.7	END OF BOREHOLE AT 4.67 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.	7	SS	50/ 100	○				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (113, 112, 111, 110, 109); 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).

+ 3 , × 3 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-4

METRIC

W.P. 169-00-00 LOCATION N 4 807 873.7 E 285 359.9 (Bronte) ORIGINATED BY SLL

HWY QEW BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY JHL

DATUM Geodetic DATE 18.06.06 - 18.06.06 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	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
117.3											
0.0	**ASPHALT**: (190 mm)										
117.0 / 0.2	CONCRETE: (140 mm)										
0.3	SAND and GRAVEL: (19 mm CRUSHER RUN LIMESTONE) Brown (FILL)						117				
116.6 / 0.7	SHALE, weathered, with limestone fragments Reddish Brown		1	SS	50/ .050						
115.9 / 1.4	END OF SPT AND AUGERING AT 1.42 m. RECORD OF BOREHOLE CONTINUES ON 06-4R		2	SS	50/ .050		116				

+ ³, × ³: Numbers refer to Sensitivity

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

ONTMT4S 5163.GPJ 12/07/06

RECORD OF BOREHOLE 06-4R

PROJECT : QEW / Bronte Road Interchange
LOCATION : Oakville, Ontario
STARTED : 06 June 2006
COMPLETED : 16 June 2006

PROJECT NO. 169-00-00

SHEET 1 OF 1

INCLINATION: Vertical AZIMUTH:

DATUM Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⌧ Laboratory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN
		SHALE BEDROCK, moderately to slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds and occasional clay seams				
2	RUN	Limestone layer (25 mm) at 2.08 m Limestone layer (50 mm) at 2.32, 2.44 m		1	0.138	100
4	RUN	Limestone layer (50 mm)at 4.04, 4.21 m Clay seam (<25mm) at 4.0m Limestone layer (140 mm) at 4.32 m	112.80 4.50	2	0.152	100
		END OF BOREHOLE AT 4.50 m. BOREHOLE GROUTED WITH BENTONITE AND PATCHED WITH ASPHALT TO SURFACE.				
6						
8						
10						

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
WATER LEVEL (date)

LOGGED : SLL
CHECKED : SKP

THURBER

RECORD OF BOREHOLE No 06-5

METRIC

W.P. 169-00-00 | LOCATION N 4 807 865.3 E 285 401.2 (Bronte) | ORIGINATED BY SLL
HWY QEW | BOREHOLE TYPE Solid Stem Augers | COMPILED BY JHL
DATUM Geodetic | DATE 13.06.06 - 13.06.06 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
114.2					
0.0	TOPSOIL: (150 mm)				
0.2	Silty CLAY, some sand, trace rootlets Firm Brown Moist	1	SS	6	o
113.3 0.9	SHALE, weathered, with grey siltstone layer Reddish Brown	2	SS	35	o
		3	SS	50/ .100	o
	Inferred limestone layer at 2.52 to 2.59m	4	SS	50/ .125	o
		5	SS	50/ .050	o
110.2		6	SS	50/ .125	o
3.9	END OF BOREHOLE AT 3.93 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (114, 113, 112, 111); 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

THURBER

RECORD OF BOREHOLE No 06-6

METRIC

W.P.	169-00-00	LOCATION	N 4 807 807.0 E 285 368.0 (Bronte)	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	09.06.06 - 09.06.06	CHECKED BY	SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
113.5 / 0.0	TOPSOIL: (100 mm)						
0.1	Silty **CLAY**, trace limestone fragments Stiff Brown Moist	1	SS	8	113	○	
112.7 / 0.8	**SHALE**, weathered Reddish Brown	2	SS	86/ .275		○	
		3	SS	50/ .125	112	○	
	Inferred limestone layer at 2.37 to 2.44m	4	SS	50/ .125	111	○	
		5	SS	50/ .125	110	○	0 2 87 11
109.6		6	SS	50/ .125		○	
3.9	END OF BOREHOLE AT 3.93 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.						

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

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-7

METRIC

W.P. 169-00-00 LOCATION N 4 807 800.2 E 285 362.3 (Bronte) ORIGINATED BY SLL
HWY QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY JHL
DATUM Geodetic DATE 09.06.06 - 09.06.06 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.3						
0.0	TOPSOIL: (75 mm)					
0.1	Clayey SILT, trace roots, topsoil stained, Firm, Dark Brown, Moist (TILL)	1	SS	4	114	3 34 46 17
113.3 / 0.9	SHALE, weathered, with grey limestone and siltstone fragments, Reddish Brown	2	SS	12	113	
		3	SS	84/.275		0 9 76 15
		4	SS	50/.150	112	
111.1		5	SS	50/.075		
3.1	END OF SPT AND AUGERING AT 3.13 m. RECORD OF BOREHOLE CONTINUES ON 06-7R					

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; 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/m^3.

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE 06-7R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 09 June 2006
COMPLETED : 09 June 2006
SHEET 1 OF 1
INCLINATION: Vertical AZIMUTH:
DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⊠ Laborotory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER .3 m	DIP w.r.t. Core Axis	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	25 Unconfined 50 Compressive Strength 75 (MPa)
	RUN	**SHALE BEDROCK**, moderately becoming slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds, and occasional clay seams Limestone layer (50 mm) at 3.28, 3.56 m			1	0.122	100								
4	RUN	Limestone layer (75 mm) at 4.21 m Clay seam (25 mm) at 4.29, 4.45 m			2	0.104	100								
	RUN	Limestone layer (75 mm) at 5.35, 5.49 and 6.02 m			3	0.080	100								
6		END OF BOREHOLE AT 6.12 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.		108.14 6.12											
8															
10															
12															

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION — WATER LEVEL (date)

DEEP/DUAL INSTALLATION — WATER LEVEL (date)

LOGGED : SLL

CHECKED : SKP

THURBER

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-8

METRIC

W.P.	169-00-00	LOCATION	N 4 807 787.7 E 285 370.8 (Bronte)	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	09.06.06 - 09.06.06	CHECKED BY	SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
113.0 / 0.0	TOPSOIL: (75 mm)				113					
0.1	Silty **CLAY**, some sand, topsoil stained. Stiff to Very Stiff. Brown. Moist	1	SS	8		○				
112.0 / 1.1	**SHALE**, weathered with grey limestone layers. Reddish Brown	2	SS	39	112	○				
		3	SS	84/.200		○ ├─┤	0	3	80	17
		4	SS	50/.050	111	○				
		5	SS	50/.100	110	○				
	Limestone fragments	6	SS	50/.075	109	○				
		7	SS	50/.075	108	○				
		8	SS	50/.050		○	0	5	82	13
106.9 / 6.1	END OF BOREHOLE AT 6.15 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.	9	SS	50/.050	107	○				

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-9

METRIC

W.P.	169-00-00	LOCATION	N 4 807 802.7 E 285 345.3 (Bronte)	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	09.06.06 - 09.06.06	CHECKED BY	SKP

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.0					116	
0.0	TOPSOIL: (150 mm)					
0.2	Sandy SILT, some sand, topsoil stained, Very Loose, Brown, Moist	1	SS	3		
		2	SS	2	115	
114.6 / 1.4	SHALE, weathered, with grey limestone layers, Reddish Brown. Inferred limestone layer at 2.42 to 2.54m and 2.74 to 2.84m	3	SS	15	114	0 10 67 23
		4	SS	50/ .125		
		5	SS	50/ .100	113	
		6	SS	50/ .100	112	
		7	SS	50/ .150	111	
110.3		8	SS	76/ .200		0 5 81 14
5.7	END OF BOREHOLE AT 5.70 m. BOREHOLE OPEN AND WATER LEVEL AT 5.23 m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.					

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; 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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-10

METRIC

W.P. 169-00-00	LOCATION N 4 807 820.4 E 285 351.1 (Bronte)		ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring		COMPILED BY JHL
DATUM Geodetic	DATE 12.06.06 - 12.06.06		CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.1							
0.0	TOPSOIL: (100 mm)				116		
0.1	Stiff Clayey SILT, trace to some gravel Stiff Reddish Brown Moist (TILL)	1	SS	10		○ ⊢⊣	10 53 29 8
	sand seam (50 mm)	2	SS	8	115	○	
114.3 / 1.8	SHALE, weathered, with grey limestone layer Reddish Brown	3	SS	65	114	○ ○	
113.5 / 2.6	END OF SPT AND AUGERING AT 2.57 m. RECORD OF BOREHOLE CONTINUES ON 06-10R	4	SS	50/ .125		○	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE 06-10R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 12 June 2006
COMPLETED : 12 June 2006
INCLINATION: Vertical AZIMUTH:
SHEET 1 OF 1
DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⊠ Laboratory UCS Test

DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN
RUN	SHALE, moderately to slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds, and occasional clay seams. Limestone layers from 2.64 to 2.92 m		1	0.068	100
RUN	Limestone layer (50 mm) at 3.25 m; Clay seam (50 mm) at 3.49 m		2	0.065	100
RUN	Limestone layer (50 mm) at 4.65, 4.77 m; Limestone layer (50 mm) at 5.31, 5.47 m	110.48 / 5.61	3	0.095	100
	END OF BOREHOLE AT 5.61 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.				

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION — WATER LEVEL (date)

DEEP/DUAL INSTALLATION — WATER LEVEL (date)

LOGGED : SLL
CHECKED : SKP

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 06-11

METRIC

W.P. 169-00-00 LOCATION N 4 807 853.3 E 285 326.9 (Bronte) ORIGINATED BY SLL

HWY QEW BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY JHL

DATUM Geodetic DATE 20.06.06 - 20.06.06 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.1									
0.0	ASPHALT: (305 mm)					117			
116.8 / 0.3	CONCRETE: (125 mm)								
116.7 / 0.4	SAND and GRAVEL: (19 mm CRUSHER RUN LIMESTONE) Compact Brown (FILL)								
116.3 / 0.8	SHALE, weathered Reddish Brown	1	SS	50/ .075		116	○		
115.6 / 1.4	END OF SPT AND AUGERING AT 1.47 m. RECORD OF BOREHOLE CONTINUES ON 06-11R	2	SS	50/ .075			○		

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

RECORD OF BOREHOLE 06-11R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 20 June 2006
COMPLETED : 20 June 2006
INCLINATION: Vertical AZIMUTH:

DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⌧ Laboratory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
	RUN	**SHALE**, moderately to slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds Limestone layer (175 mm) at 1.45 m Limestone layer (50 mm)at 1.98, 2.13 m Limestone layer (175 mm) at 2.21 m		1	0.190	100	
2							
	RUN	Limestone layer (25 mm) at 3.20 m Limestone layer (50 mm) at 4.01 m Limestone layer (100 mm) at 4.15 m		2	0.139	100	Vertical joint 4.32 to 4.50 m
4							
		END OF BOREHOLE AT 4.50 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.	112.60 4.50				
6							
8							
10							

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION WATER LEVEL (date)

DEEP/DUAL INSTALLATION WATER LEVEL (date)

LOGGED : SLL

CHECKED : SKP

THURBER

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-12

METRIC

W.P. 169-00-00 LOCATION N 4 807 864.5 E 285 336.4 (Bronte) ORIGINATED BY SLL

HWY QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY JHL

DATUM Geodetic DATE 20.06.06 - 20.06.06 CHECKED BY SKP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	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
117.2 / 0.0	ASPHALT: (125 mm)									
116.9 / 0.1	CONCRETE: (75 mm)					117				
116.8 / 0.2	ASPHALT: (100 mm)									
116.5 / 0.3	CONCRETE: (125 mm)									
0.4	SAND and GRAVEL: (19 mm CRUSHER RUN LIMESTONE) (FILL)									
0.7	SHALE, weathered, with limestone layers Reddish Brown	1	SS	50/.100		116				
	Inferred limestone layer (75 mm) at 1.68 m	2	SS	50/.075						
	Inferred limestone layer (50 mm) at 2.29 m	3	SS	50/.075		115				
		4	SS	50/.050		114				
113.3 / 3.9	Inferred limestone layer (25 mm) at 3.76 m	5	SS	50/.050						
	END OF BOREHOLE AT 3.86 m. BOREHOLE GROUTED WITH BENTONITE AND PATCHED WITH ASPHALT TO SURFACE.									

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-13

METRIC

W.P. 169-00-00 | LOCATION N 4 807 876.2 E 285 321.1 (Bronte) | ORIGINATED BY SLL

HWY QEW | BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring | COMPILED BY JHL

DATUM Geodetic | DATE 13.06.06 - 13.06.06 | CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
121.1						
0.0	TOPSOIL: (100 mm)					
0.1	Silty **CLAY**, some sand, Stiff, Dark Brown, Moist	1	SS	8	121	
120.5 / 0.7	**SAND**, Loose, Brown, Moist	2	SS	9	120	
119.7 / 1.5	Sandy **SILT**, trace clay, trace sand, Compact, Brown, Moist to Wet	3	SS	20		0 25 67 8
		4	SS	22	119	
118.2 / 3.0	Clayey **SILT**, trace gravel, Stiff, Grey, Moist (TILL)	5	SS	14	118	
117.3 / 3.9	**SHALE**, weathered, Reddish Brown	6	SS	36	117	0 12 88 (SI+CL)
116.4		7	SS	50/.150		
4.7	END OF SPT AND AUGERING AT 5.79 m. RECORD OF BOREHOLE CONTINUES ON 06-13R					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+^{3}, ×^{3}: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE 06-13R

PROJECT : QEW / Bronte Road Interchange

LOCATION : Oakville, Ontario

STARTED : 13 June 2006

COMPLETED : 13 June 2006

PROJECT NO. 169-00-00

THURBER

SHEET 1 OF 2

INCLINATION: Vertical AZIMUTH:

DATUM Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⌶ Laborotory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	RUN No.	PENETRATRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FIELD/LABORATORY TESTING RESULTS
	RUN	SHALE, moderately becoming slightly weathered to fresh, fine grained, thinly bedded, reddish brown, with frequent grey strong LIMESTONE interbeds, and occasional clay seams Limestone layer (25 mm) at 4.86 m Limestone layer (75 mm) at 4.90 m Limestone layer (50 mm) at 5.00 m Limestone layer (150 mm) at 5.64 m Limestone layer (125 mm) at 5.81 m	1	0.090	100		
6	RUN	Limestone layer (50 mm) at 6.20 m Limestone layer (100 mm) at 6.38 m Limestone layer (25 mm) at 6.55 m	2	0.090	100		
8	RUN	Limestone layer (100 mm) at 7.97 m Limestone layer (25 mm) at 8.11 m Limestone layer (35 mm) at 8.46 m Limestone layer (25 mm) at 8.67, 8.74, 9.07 m	3	0.073	100	Vertical joint 9.32 to 9.46 m	
10	RUN	Limestone layer (50 mm) at 9.93 m Limestone layer (75 mm) at 10.46 m Limestone layer (50 mm) at 10.67 m	4	0.090	100		
12	RUN	Limestone layer (50 mm) at 10.97 m Limestone layer (150 mm) at 11.53 m	5	0.102	100		Bentonite 108.74
	RUN	Limestone layer (100 mm) at 12.63 m Limestone layer (50 mm) at 12.78, 13.27 m Limestone layer (150 mm) at 13.63 m	6	0.085	100	Subvertical joint 12.77 to 12.83 m	Sand Filter 107.40
14	RUN	Limestone layer (50 mm) at 13.92 m Limestone layer (25 mm) at 14.33 m	7	0.090	100		Slotted

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION — WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION — WATER LEVEL (date)

LOGGED : SLL

CHECKED : SKP

THURBER

RECORD OF BOREHOLE 06-13R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 13 June 2006
COMPLETED : 13 June 2006
INCLINATION: Vertical AZIMUTH:

DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⊠ Laboratory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER .3 m	DISCONTINUITY DATA DIP w.r.t. Core Axis	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS
		Limestone layer (50 mm) at 14.86, 14.99 m		105.88												Screen 105.88
		END OF BOREHOLE AT 15.27 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE UPON COMPLETION OF PIEZOMETER INSTALLATION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.		15.27												105.70
16																
		WATER LEVEL READINGS:														
18																
20																
22																
24																

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Jun 28/06	11.3	109.8
Jul 6/06	11.2	109.9

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION WATER LEVEL (date)

LOGGED : SLL
CHECKED : SKP

THURBER

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-14

METRIC

W.P.	169-00-00	LOCATION	N 4 807 888.4 E 285 354.0 (Bronte)	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	19.06.06 - 19.06.06	CHECKED BY	SKP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	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
116.8											
0.0	ASPHALT: (175 mm)										
0.2	SAND and GRAVEL: (19 mm CRUSHER RUN LIMESTONE) Compact Brown (FILL)		1	AS					○		
115.9 0.9	SHALE, weathered Reddish Brown		1	SS	50/ .075		116		○		
115.3 1.5	AUGER REFUSAL AT 1.47 m. END OF BOREHOLE AT 1.47 m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS AND PATCHED WITH ASPHALT TO SURFACE.		2	SS	50/ .025				○		

ONTMT4S 5163.GPJ 12/07/06

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-15

METRIC

W.P. 169-00-00
LOCATION N 4 807 897.5 E 285 361.0 (Bronte)
ORIGINATED BY SLL

HWY QEW
BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring
COMPILED BY JHL

DATUM Geodetic
DATE 19.06.06 - 19.06.06
CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
117.1					
0.0	ASPHALT: (160 mm)				
0.2	SAND and GRAVEL: (19 mm CRUSHER RUN LIMESTONE) Compact Brown (FILL)	1	AS		o
116.4 / 0.7	SHALE, weathered Reddish Brown	1	SS	50/.075	o
115.7		2	SS	50/.050	o
1.5	END OF SPT AND AUGERING AT 1.47 m. RECORD OF BOREHOLE CONTINUES ON 06-15R				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (117, 116); 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

ONTMT4S 5163.GPJ 12/07/06

RECORD OF BOREHOLE 06-15R

PROJECT : QEW / Bronte Road Interchange
PROJECT NO. 169-00-00
LOCATION : Oakville, Ontario
STARTED : 19 June 2006
COMPLETED : 19 June 2006
SHEET 1 OF 1
INCLINATION: Vertical AZIMUTH:
DATUM Geodetic

THURBER

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

FIELD/LABORATORY TESTING RESULTS: ● Point Load Test Diametral; ▲ Point Load Test Axial; ⊠ Laborotory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN
2	RUN	**SHALE BEDROCK**, moderately becoming slightly weathered, fine grained, thinly bedded, reddish brown, with frequent grey, strong LIMESTONE interbeds, and occasional clay seams Limestone layer (50 mm) at 1.76 m Limestone layer (50 mm) at 1.96 m			1	0.190	100
4	RUN	Limestone layer (75 mm) at 3.00, 3.35 m Clay seam (25 mm) at 3.05, 3.18 m Limestone layer (50 mm) at 3.51 m Limestone layer (150 mm) at 3.66 m		112.61 4.52	2	0.190	100
		END OF BOREHOLE AT 4.52 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.					
6							
8							
10							

Other columns: RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20), R.Q.D. % (80 60 40 20), FRACT. INDEX PER 0.3 m (5 10 15 20), DISCONTINUITY DATA (DIP w.r.t. Core Axis 0 30 60 90; TYPE AND SURFACE DESCRIPTION), HYDRAULIC CONDUCTIVITY k, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}), Unconfined Compressive Strength (MPa) 25 50 75.

GROUNDWATER ELEVATIONS

SHALLOW/SINGLE INSTALLATION WATER LEVEL (date)

DEEP/DUAL INSTALLATION WATER LEVEL (date)

LOGGED : SLL
CHECKED : SKP

THURBER

ROCKM 5163.GPJ 12/07/06

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-16

METRIC

W.P. 169-00-00 LOCATION N 4 807 916.8 E 285 354.5 (Bronte) ORIGINATED BY SLL

HWY QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY JHL

DATUM Geodetic DATE 13.06.06 - 13.06.06 CHECKED BY SKP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
120.7 0.0	TOPSOIL: (150 mm)					
0.2	SILT and SAND, trace clay, trace roots and rootlets Loose to Compact Dark Brown Moist	1	SS	5		
					120	
		2	SS	16		0 57 38 5
119.2 1.5	Clayey SILT, trace sand Stiff to Very Stiff Grey Wet (TILL)	3	SS	11	119	
		4	SS	21	118	
117.8	Shale fragment at 2.90 m					
2.9	SHALE, weathered Reddish Brown	5	SS	50/ .100		
117.4 3.3	END OF BOREHOLE AT 3.32 m. AUGER REFUSAL AT 3.32 m. BOREHOLE OPEN AND DRY TO BOTTOM ON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.					

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; 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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 06-16A

METRIC

W.P. 169-00-00 | LOCATION N 4 807 916.8 E 285 353.0 (Bronte) | ORIGINATED BY SLL

HWY QEW | BOREHOLE TYPE Solid Stem Augers | COMPILED BY JHL

DATUM Geodetic | DATE 14.06.06 - 14.06.06 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
120.7 / 0.0	AUGERED WITHOUT SAMPLING TO 3.8 m.										
							120				
							119				
							118				
	Inferred weathered SHALE started at 2.8 m Reddish Brown Inferred Limestone layer at 3.20 to 3.28 m										
							117				
			1	SS	50/ .125				o		
116.0 / 4.7			2	SS	50/ .100				o		
	END OF BOREHOLE AT 4.67 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.										

Legend: DCPT scale 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

Appendix B

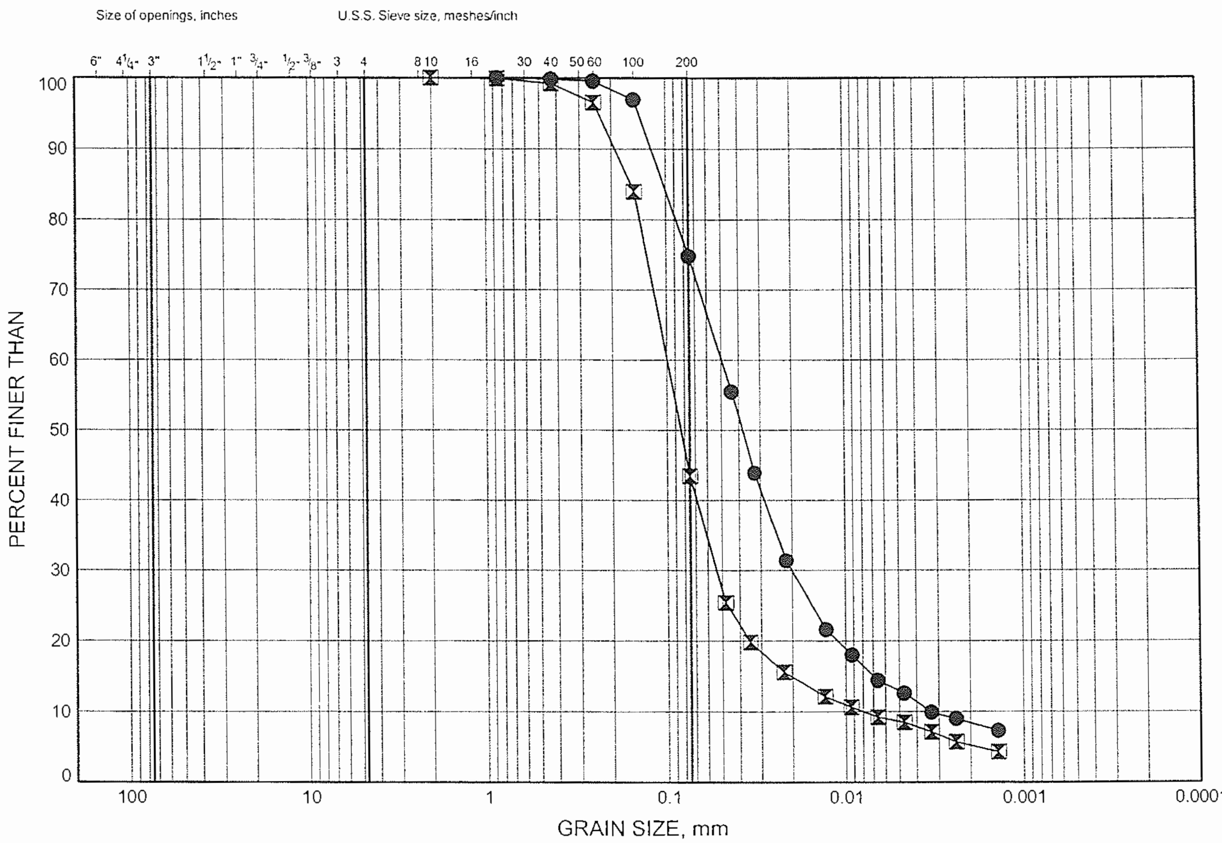
Laboratory Test Results

QEW / Bronte Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE B1

SANDY SILT TO SAND AND SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-13	1.83	119.31
⊠	06-16	1.07	119.61

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

Chkd. SKP

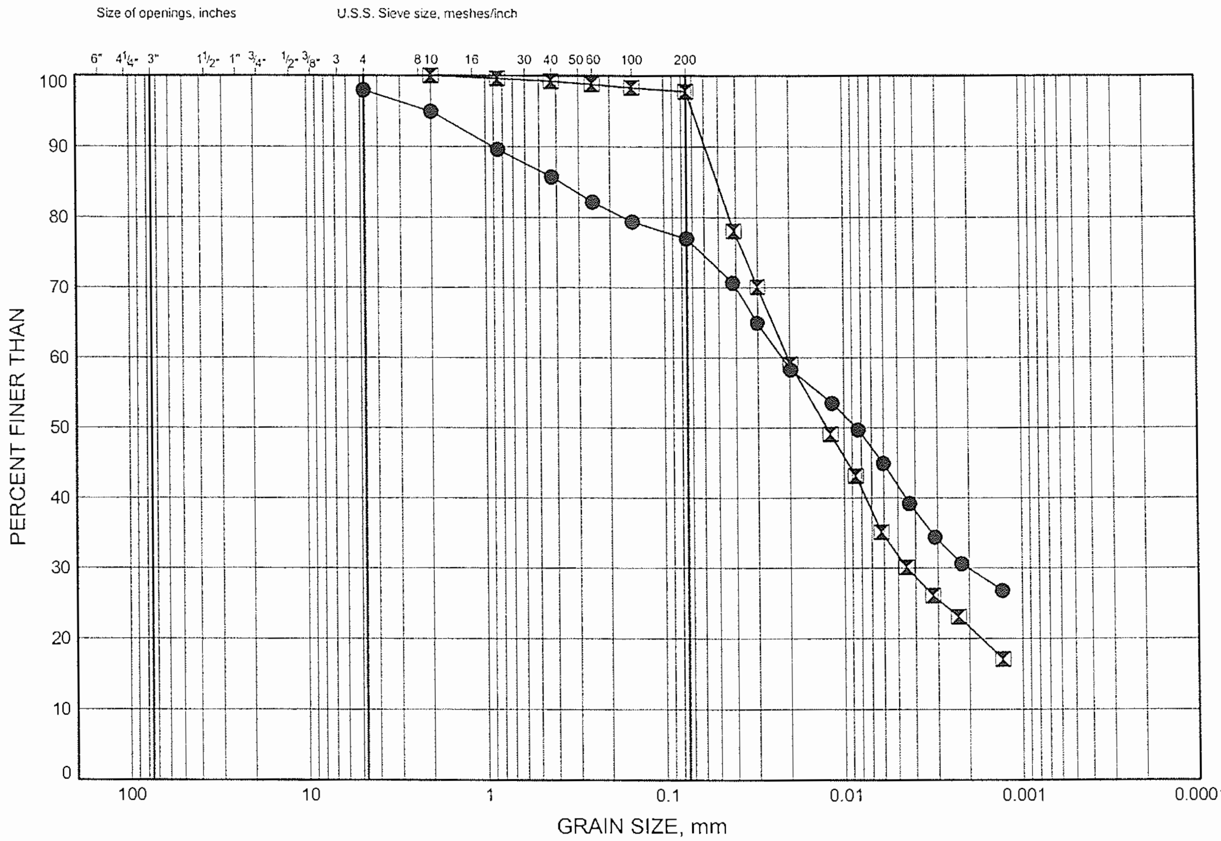
THURBGSD 5163.GPJ 12/07/06

QEW / Bronte Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-1	0.30	111.87
⧗	06-3	1.07	112.17

THURBGSD 5163.GPJ 12/07/06

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

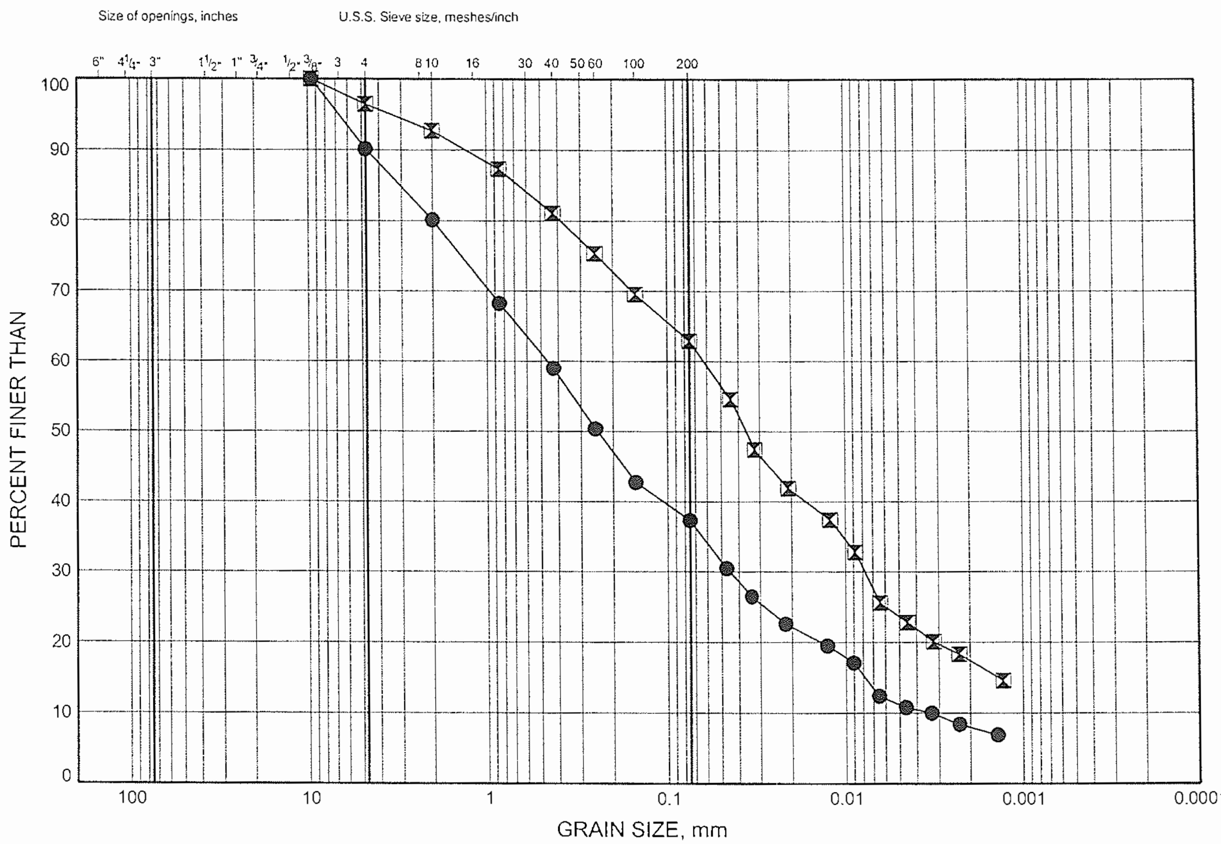
Chkd. SKP

QEW / Bronte Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE B3

CLAYEY SILT TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-10	0.30	115.79
⧗	06-7	0.30	113.96

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

Chkd. SKP

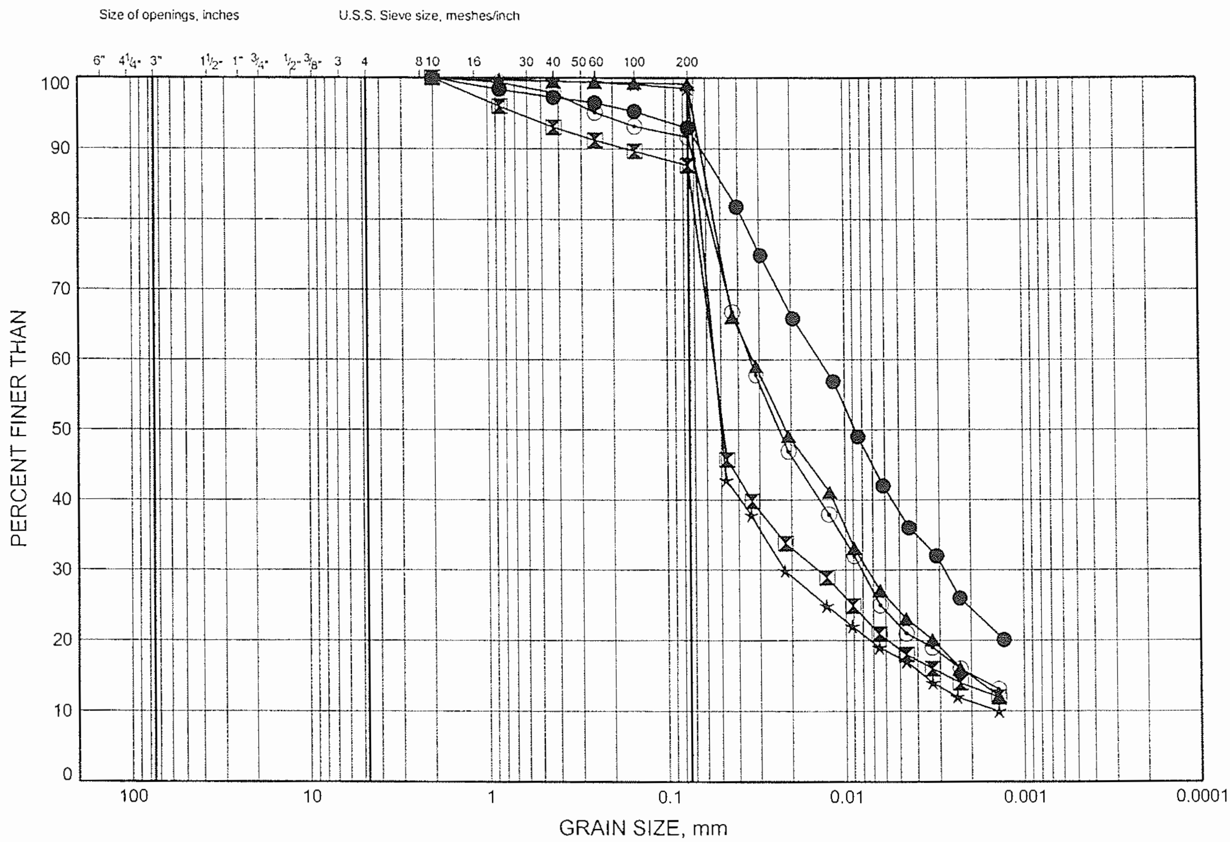
THURBGSD 5163.GPJ 12/07/06

QEW / Bronte Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE B4

WEATHERED SHALE



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-1	1.75	110.42
⧗	06-13	4.11	117.03
▲	06-3	1.83	111.41
★	06-6	3.11	110.41
⊙	06-7	1.75	112.51

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

Chkd. SKP

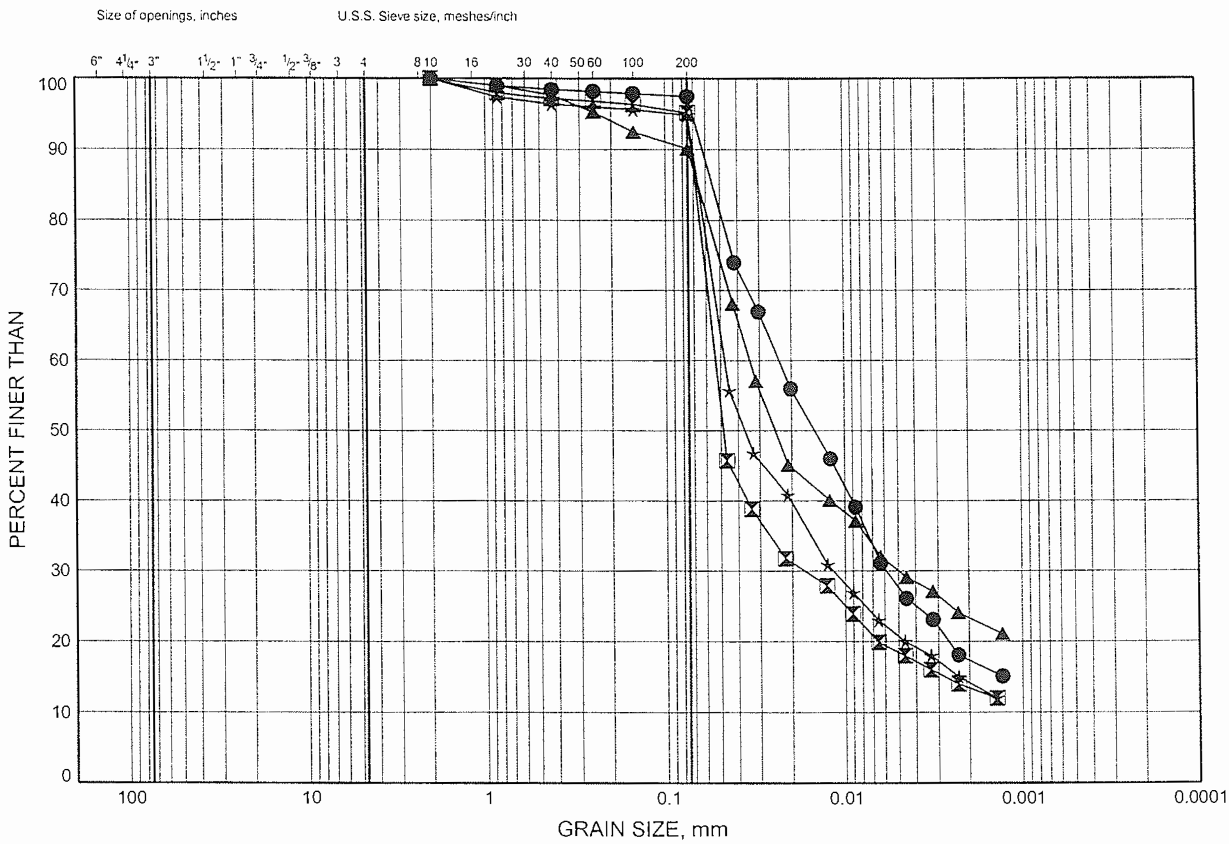
THURBGSD 5163.GPJ 12/07/06

QEW / Bronte Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE B5

WEATHERED SHALE



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-8	1.70	111.34
⊠	06-8	5.66	107.38
▲	06-9	1.93	114.05
★	06-9	5.43	110.55

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

Chkd. SKP

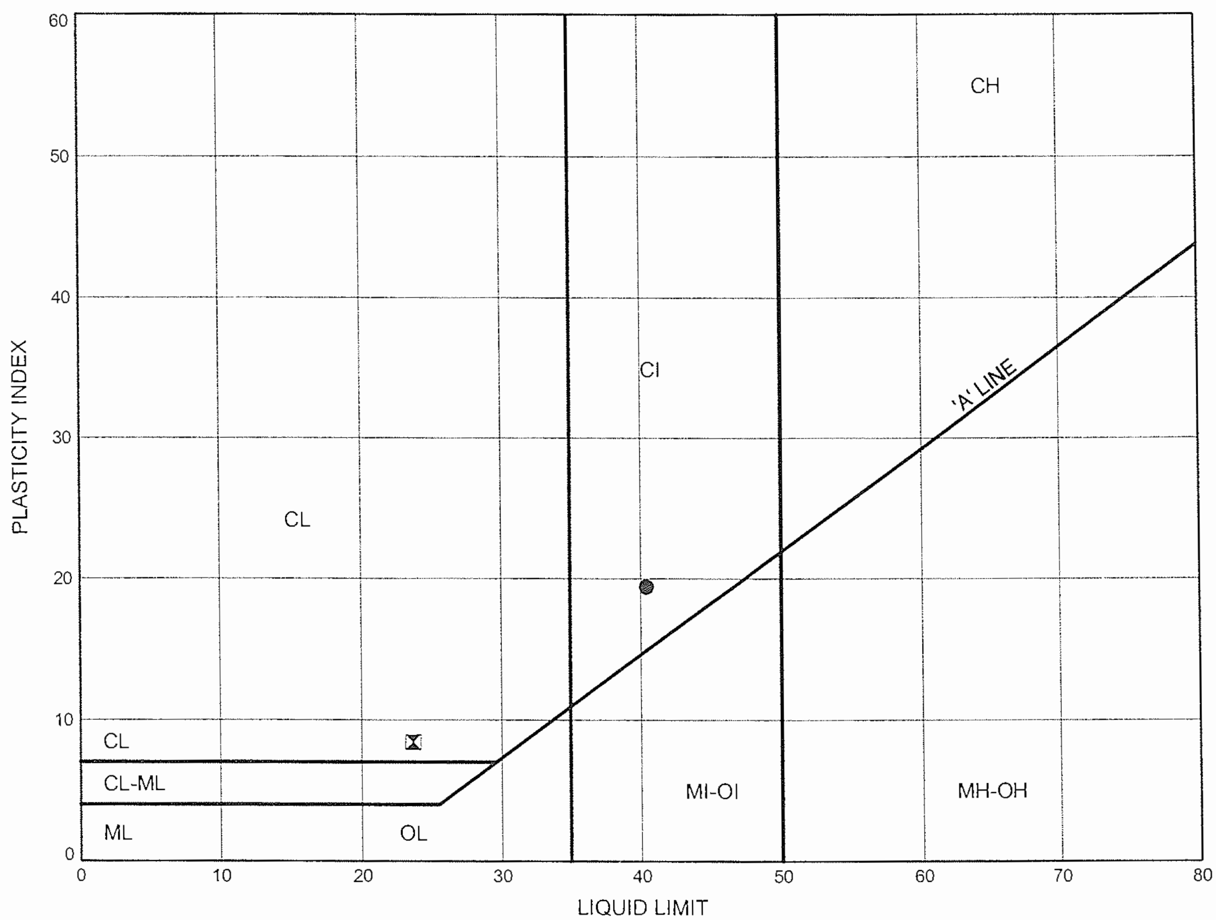
THURBGSD 5163.GPJ 12/07/06

QEW / Bronte Road Interchange

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-1	0.30	111.87
⊠	06-10	0.30	115.79

THURBALT 5163.GPJ 12/07/06

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

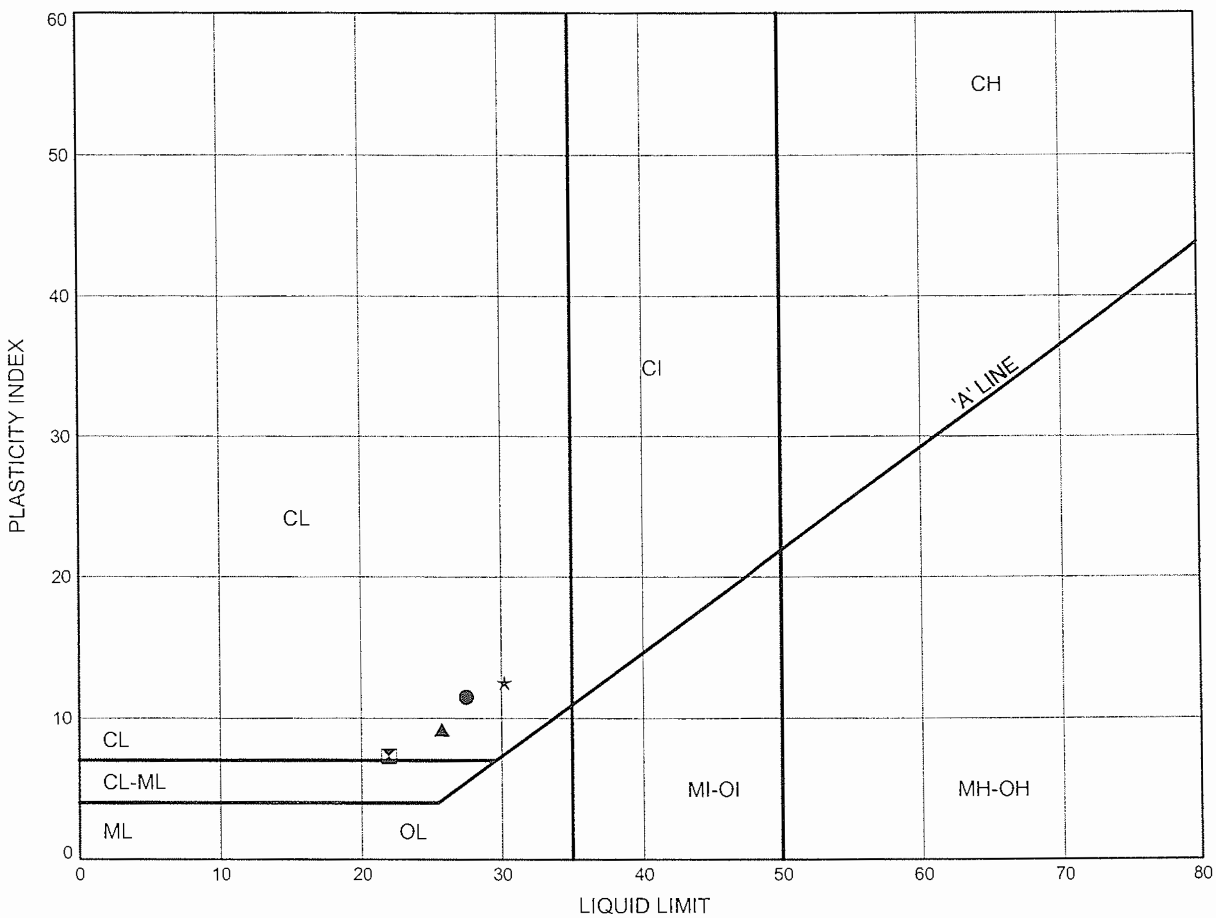
Chkd. SKP

QEW / Bronte Road Interchange

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

WEATHERED SHALE



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-1	1.75	110.42
⧖	06-13	4.11	117.03
▲	06-7	1.75	112.51
★	06-8	1.70	111.34

Date July 2006

Project 169-00-00

THURBER

Prep'd JHL

Chkd. SKP

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	1
BOREHOLE NUMBER	06-1	SAMPLE DEPTH, m	0.25-0.38

TEST CONDITIONS

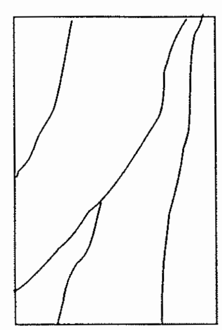
MACHINE SPEED, mm/min	0.91	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	1.95

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.13	WATER CONTENT, (specimen) %	7.40
SAMPLE DIAMETER, cm	4.69	UNIT WEIGHT, kN/m^3	24.21
SAMPLE AREA, cm^2	17.28	DRY UNIT WT., kN/m^3	2.30
SAMPLE VOLUME, cm^3	157.80	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	389.71	VOID RATIO	-
DRY WEIGHT, g	362.86		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

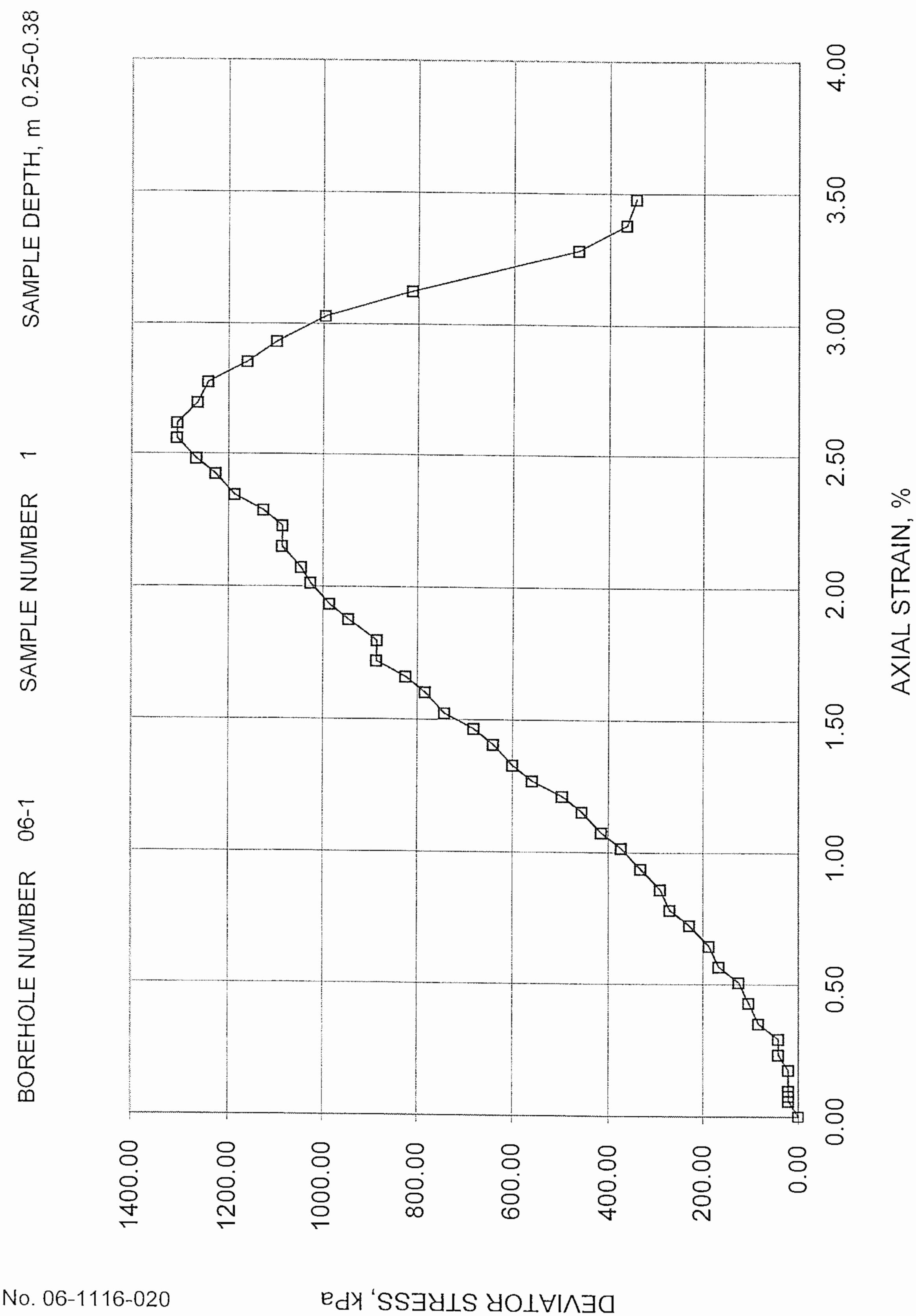
STRAIN AT FAILURE, %	2.6	COMPRESSIVE STRESS, MPa	1.3

REMARKS:

DATE: June 14, 2006

Golder Associates

UNCONFINED COMPRESSION TEST (UC) | **FIGURE**



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	2
BOREHOLE NUMBER	06-1	SAMPLE DEPTH, m	0.23-0.33

TEST CONDITIONS

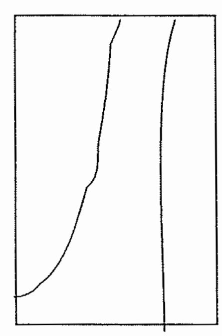
MACHINE SPEED, mm/min	0.80	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	1.70

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.04	WATER CONTENT, (specimen) %	3.60
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	24.76
SAMPLE AREA, cm^2	17.57	DRY UNIT WT., kN/m^3	2.44
SAMPLE VOLUME, cm^3	141.31	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	356.85	VOID RATIO	-
DRY WEIGHT, g	344.45		

VISUAL INSPECTION

FAILURE SKETCH



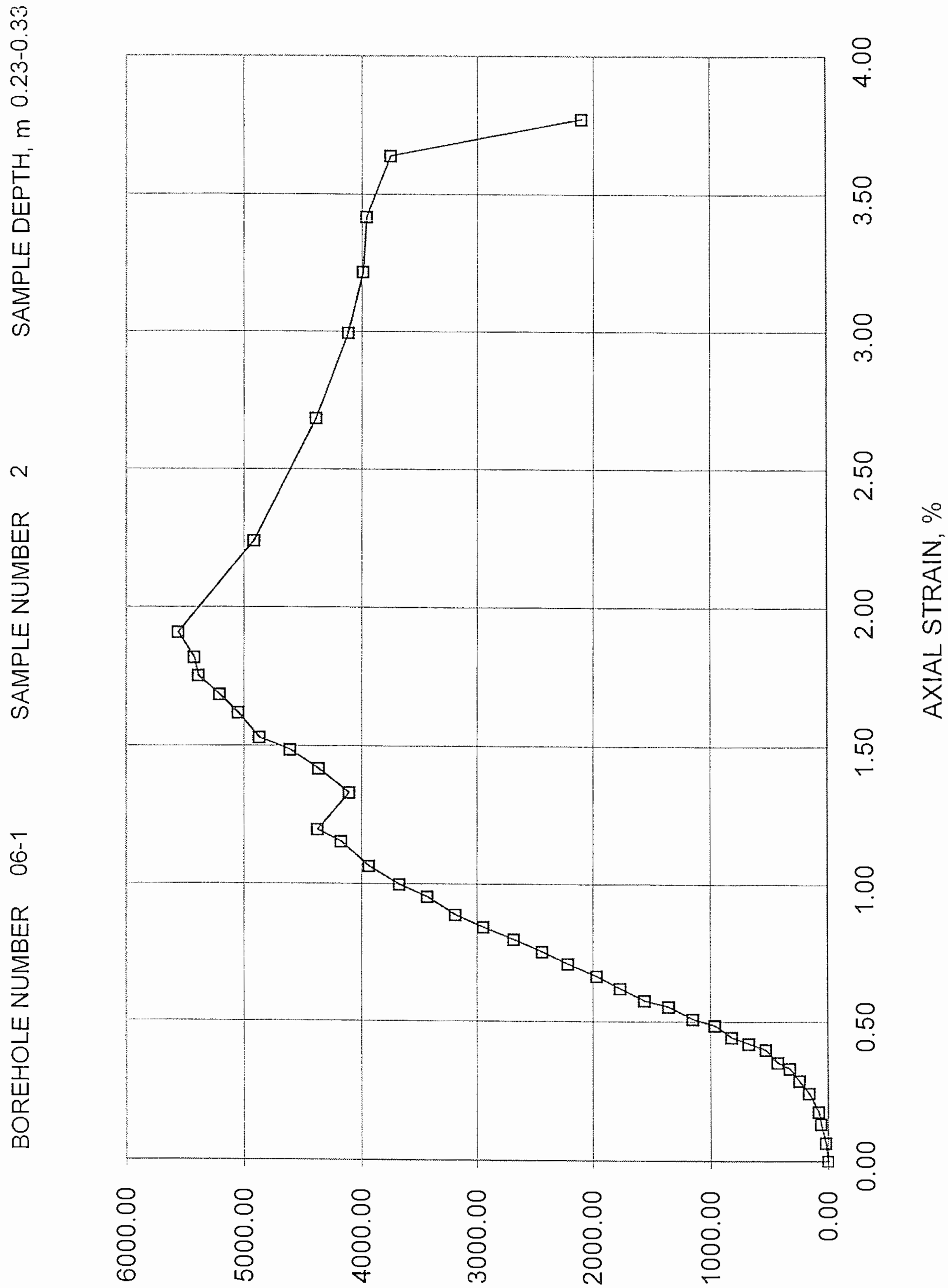
TEST RESULTS

STRAIN AT FAILURE, %	1.9	COMPRESSIVE STRESS, MPa	5.6

REMARKS:

DATE: June 16, 2006

UNCONFINED COMPRESSION TEST (UC)	FIGURE



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	3
BOREHOLE NUMBER	06-1	SAMPLE DEPTH, m	0.53-0.74

TEST CONDITIONS

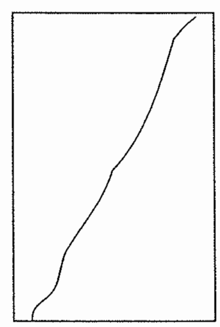
MACHINE SPEED, mm/min	0.12	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.76	WATER CONTENT, (specimen) %	4.80
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	25.09
SAMPLE AREA, cm^2	17.65	DRY UNIT WT., kN/m^3	2.44
SAMPLE VOLUME, cm^3	189.87	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	485.90	VOID RATIO	-
DRY WEIGHT, g	463.65		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

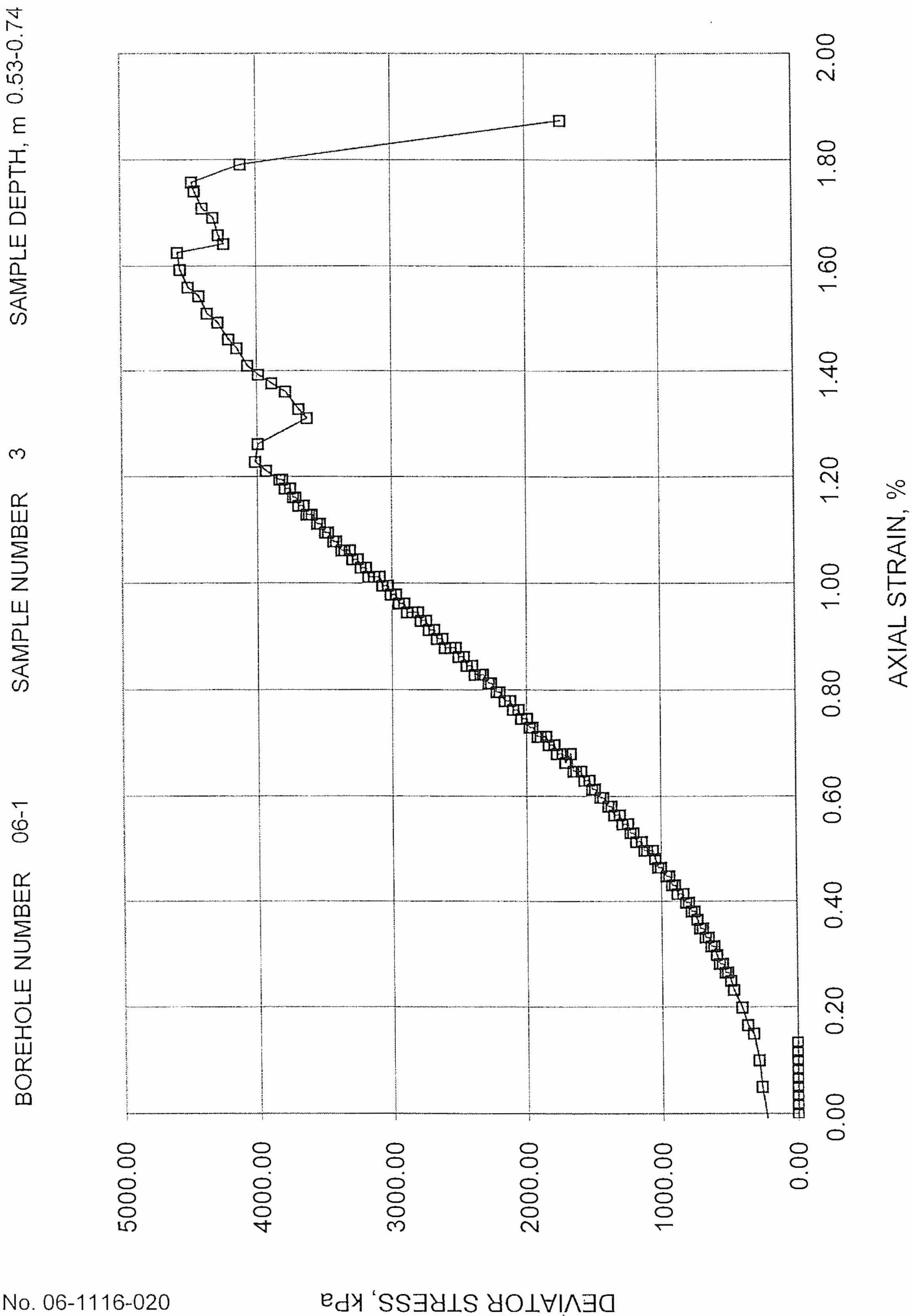
STRAIN AT FAILURE, %	1.6	COMPRESSIVE STRESS, MPa	4.6

REMARKS:

DATE: June 14, 2006

Golder Associates

UNCONFINED COMPRESSION TEST (UC)	FIGURE



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	4
BOREHOLE NUMBER	06-13	SAMPLE DEPTH, m	0.63-0.79

TEST CONDITIONS

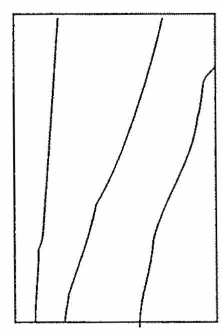
MACHINE SPEED, mm/min	0.81	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	1.71

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.08	WATER CONTENT, (specimen) %	3.40
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	25.27
SAMPLE AREA, cm^2	17.57	DRY UNIT WT., kN/m^3	2.49
SAMPLE VOLUME, cm^3	141.98	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	365.92	VOID RATIO	-
DRY WEIGHT, g	353.89		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

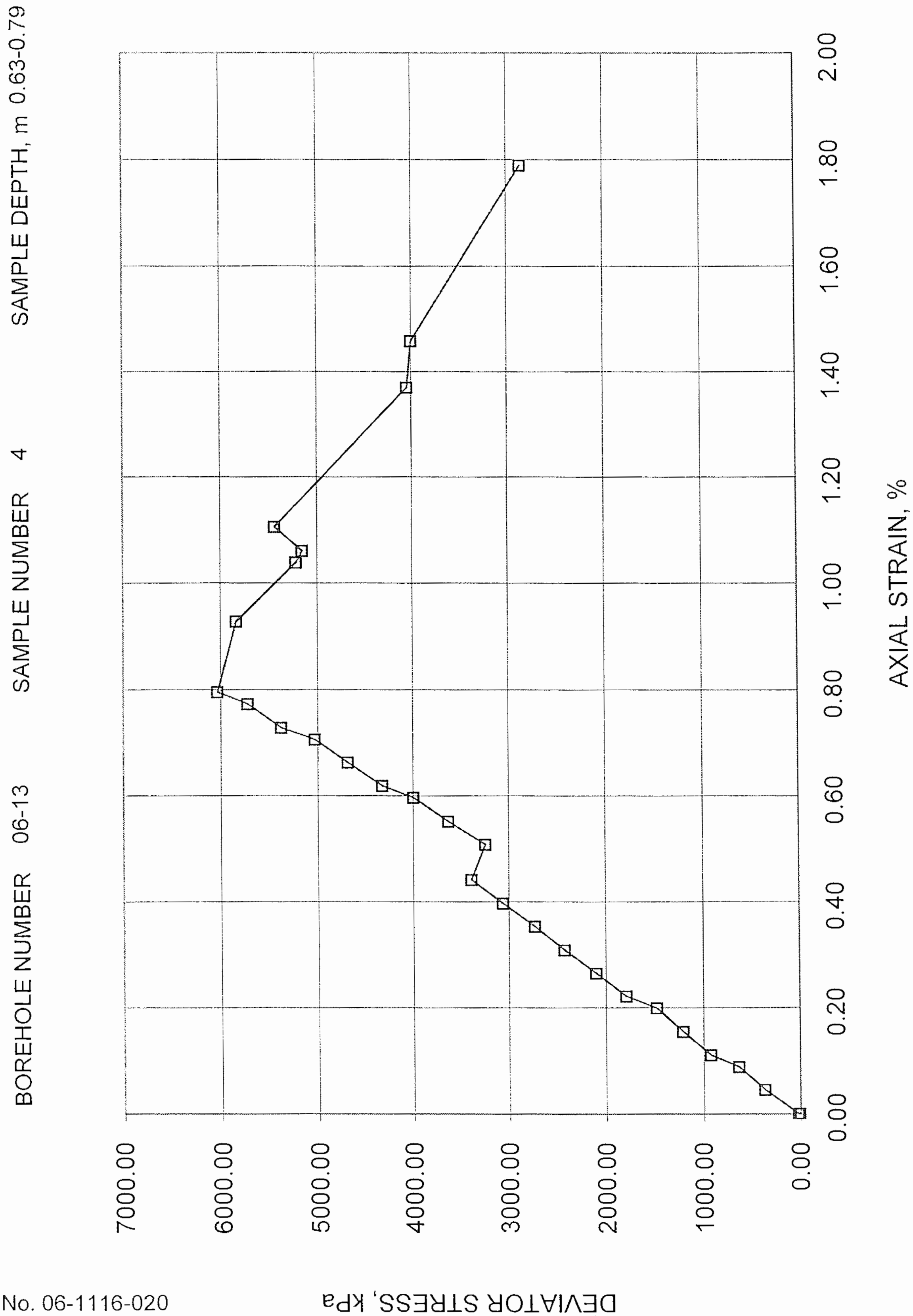
STRAIN AT FAILURE, %	0.8	COMPRESSIVE STRESS, MPa	6.0

REMARKS:

DATE: June 16, 2006

Golder Associates

UNCONFINED COMPRESSION TEST (UC)	FIGURE



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	5
BOREHOLE NUMBER	06-13	SAMPLE DEPTH, m	0.43-0.58

TEST CONDITIONS

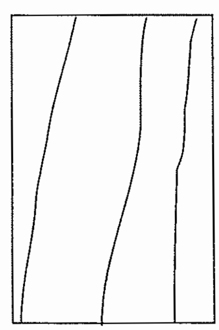
MACHINE SPEED, mm/min	1.03	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	2.16

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.25	WATER CONTENT, (specimen) %	3.30
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	25.54
SAMPLE AREA, cm^2	17.71	DRY UNIT WT., kN/m^3	2.52
SAMPLE VOLUME, cm^3	181.48	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	472.88	VOID RATIO	-
DRY WEIGHT, g	457.77		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	1.5	COMPRESSIVE STRESS, MPa	12.9

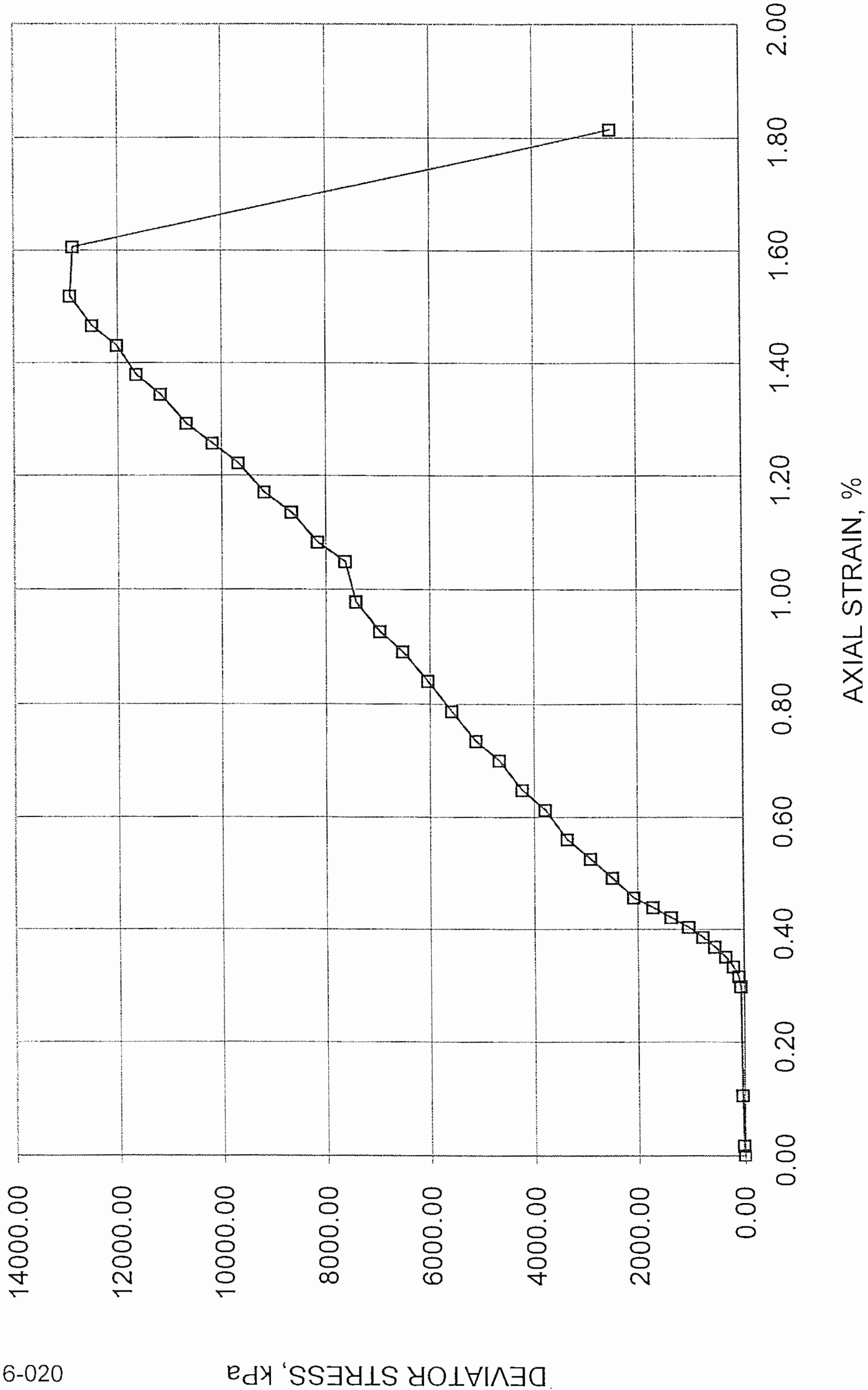
REMARKS:

DATE: June 16, 2006

Golder Associates

UNCONFINED COMPRESSION TEST (UC)	FIGURE

BOREHOLE NUMBER 06-13 SAMPLE NUMBER 5 SAMPLE DEPTH, m 0.43-0.58



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	6
BOREHOLE NUMBER	06-13	SAMPLE DEPTH, m	0.69-0.81

TEST CONDITIONS

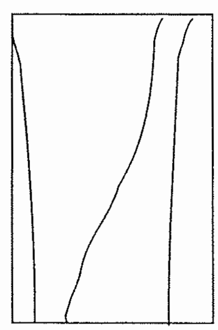
MACHINE SPEED, mm/min	0.94	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	1.97

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.37	WATER CONTENT, (specimen) %	2.30
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	25.28
SAMPLE AREA, cm^2	17.72	DRY UNIT WT., kN/m^3	2.52
SAMPLE VOLUME, cm^3	166.04	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	428.18	VOID RATIO	-
DRY WEIGHT, g	418.55		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

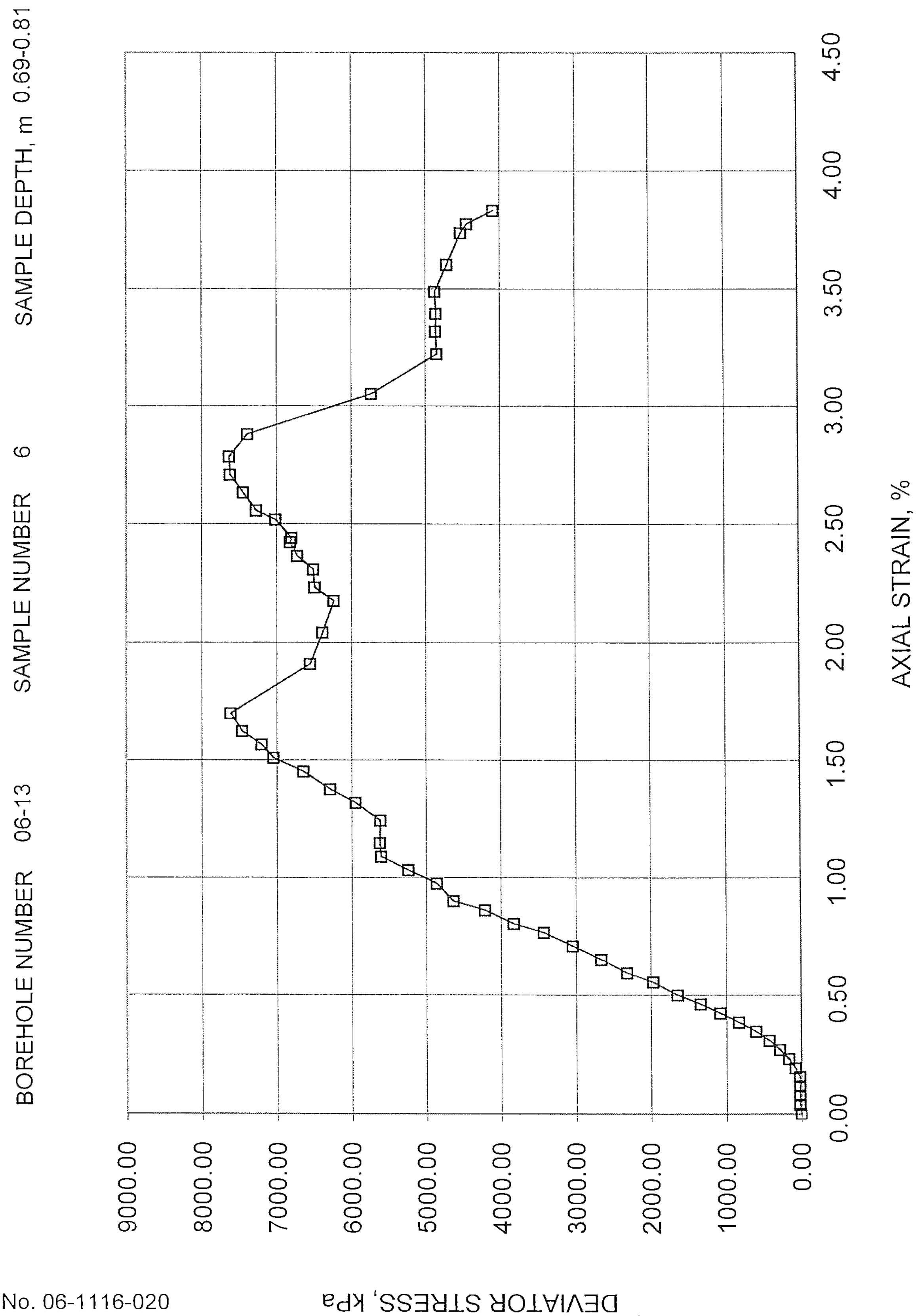
STRAIN AT FAILURE, %	3.5	COMPRESSIVE STRESS, MPa	7.6

REMARKS:

DATE: June 16, 2006

Golder Associates

UNCONFINED COMPRESSION TEST (UC)	FIGURE



Project No. 06-1116-020

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-020	SAMPLE NUMBER	7
BOREHOLE NUMBER	06-13	SAMPLE DEPTH, m	1.32-1.50

TEST CONDITIONS

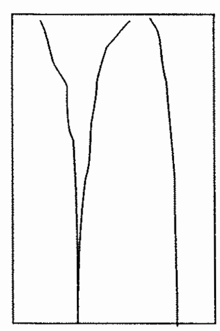
MACHINE SPEED, mm/min	0.77	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST	>2 <15	L/D	1.63

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	7.74	WATER CONTENT, (specimen) %	3.70
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	25.15
SAMPLE AREA, cm^2	17.65	DRY UNIT WT., kN/m^3	2.47
SAMPLE VOLUME, cm^3	136.58	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	350.38	VOID RATIO	-
DRY WEIGHT, g	337.88		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

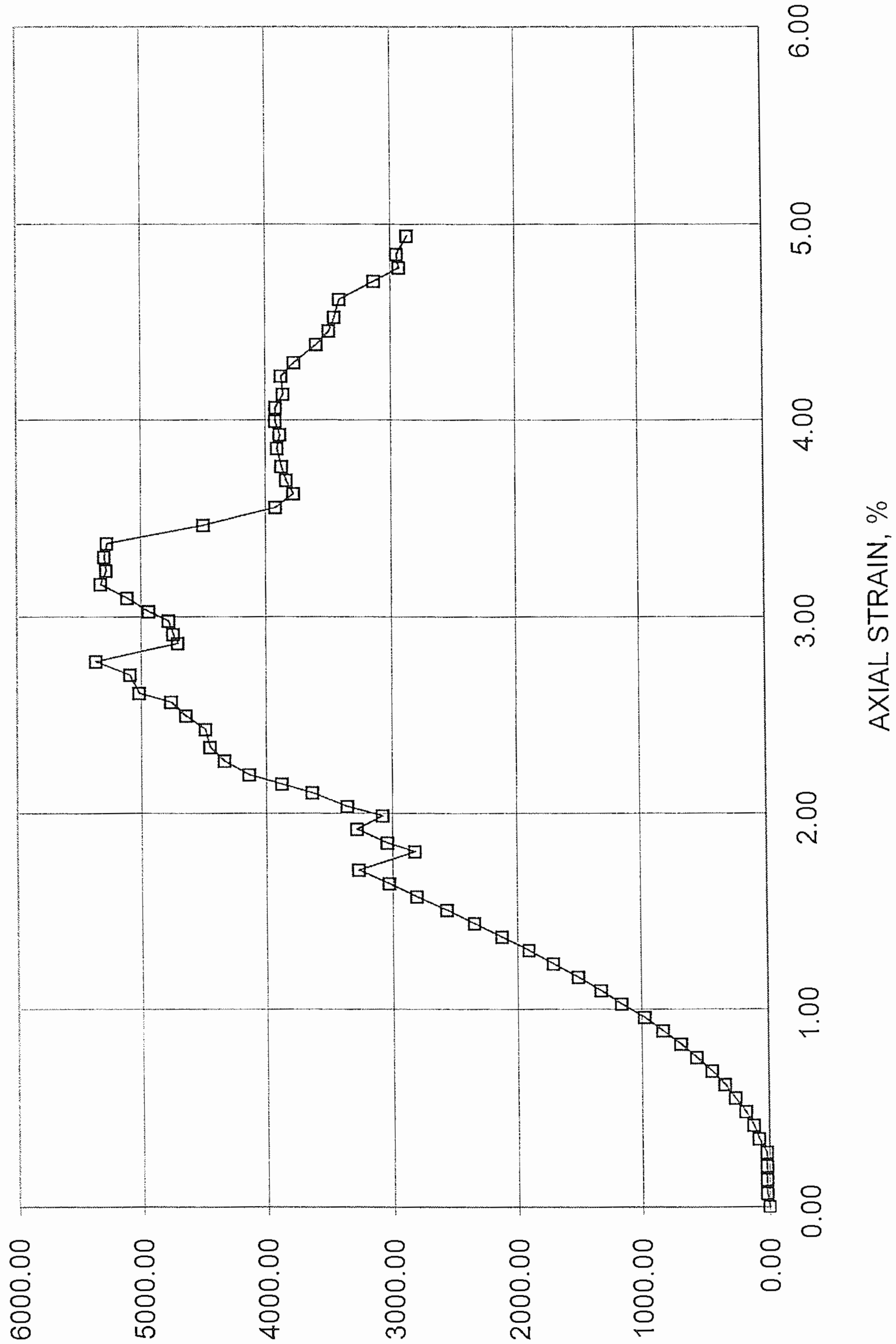
STRAIN AT FAILURE, %	2.8	COMPRESSIVE STRESS, MPa	5.4

REMARKS:

DATE: June 16, 2006

UNCONFINED COMPRESSION TEST (UC)	FIGURE

BOREHOLE NUMBER 06-13 SAMPLE NUMBER 7 SAMPLE DEPTH, m 1.32-1.50



Project No. 06-1116-020

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Steel H-Piles	Footings on Bedrock	Augered Caisson
West and East Abutments	*Advantages:* i. Required foundation type for integral abutment design. ii. Foundation construction requires less volume of excavation than footings. *Disadvantages:* i. Presence of bedrock at design grade of the cut and fixity requirement of integral abutment piles require that the piles be socketted into bedrock.	*Advantages:* i. Presence of bedrock at design grade of the cut. ii. High values of geotechnical resistance are available on the bedrock. *Disadvantages:* i. Stepped footing may be required due to undulation of bedrock. ii. Higher cost of excavation into bedrock. iii. Mass concrete fill may be required to raise the founding subgrade level. iv. Cannot be incorporated into an integral abutment design.	*Advantages:* i. High values of geotechnical resistance are available on the bedrock. *Disadvantages:* i. Potential difficulties in forming the caisson holes into bedrock; pneumatic rock splitting/breaking and/or coring equipment may be required to penetrate the shale with hard limestone interbeds. ii. Higher cost of excavation into bedrock. iii. Dewatering may be required. iv. Inspection by geotechnical personnel may be required.

THURBER

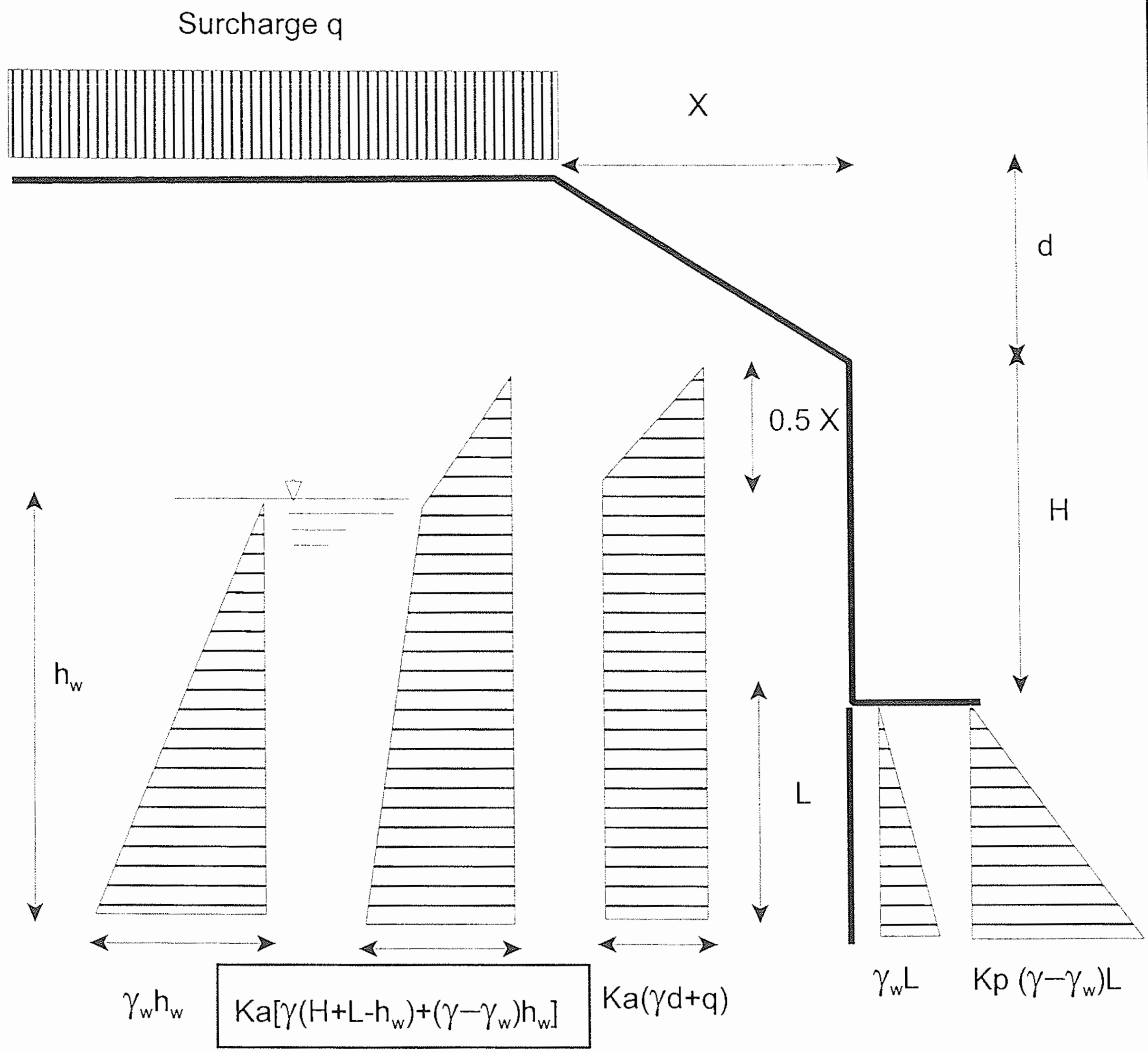
Appendix D

Table and Figures

TABLE D1

The annular space surrounding the piles in an integral abutment design should be filled with sand meeting the following gradation requirements :

MTO Sieve Designation	Percentage Passing by Mass
2 mm (# 10)	100
600 μm (# 30)	80 – 100
425 μm (# 40)	40 – 80
250 μm (# 60)	5 – 25
150 μm (# 100)	0 – 6



γ = unit weight of soil

γ_w = unit weight of water

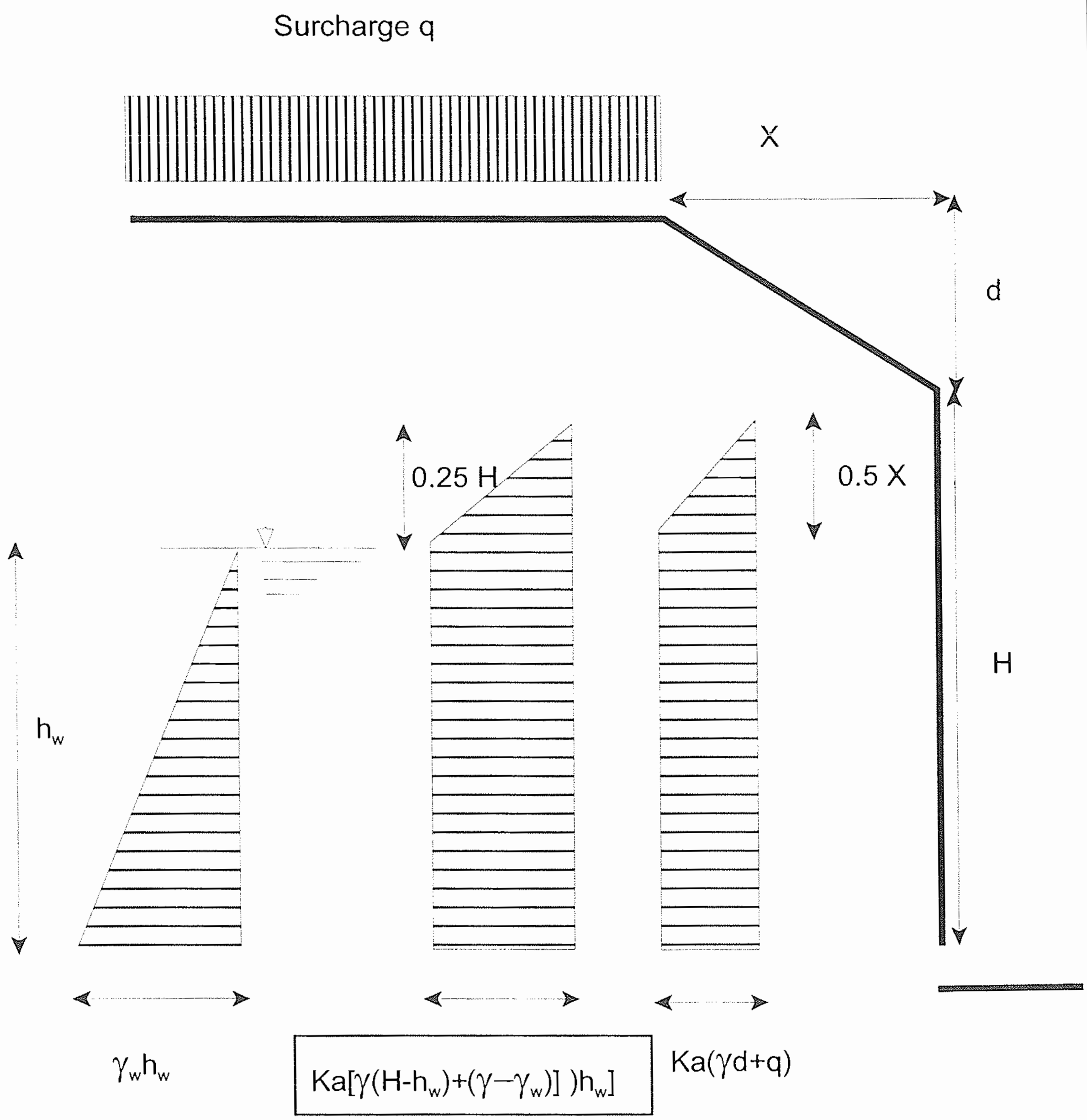
Ka = active earth pressure coefficient

Kp = passive earth pressure coefficient

THURBER

LATERAL PRESSURE DISTRIBUTION FOR ANCHORED WALL

FIGURE D1



γ = unit weight of clay

γ_w = unit weight of water

Ka = earth pressure coefficient

THURBER	**LATERAL PRESSURE DISTRIBUTION FOR BRACED SHORING**	**FIGURE D2**

Appendix E

List of SPs and OPSS

Suggested Text for Selected NSSP

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

- SP 902 S01
- SP 903 S01
- SP 539 S01
- OPSS 120 (1994)
- OPSS 206 (2002)
- OPSS 501
- OPSS 572
- OPSS 942 (2004)
- OPSS 1010

2. Suggested Text for NSSP on "Rock Excavation"

The strength of the shale bedrock increases with depth and there is presence of very hard limestone and/or siltstone interbeds within the shale bedrock. Bulk excavation and pre-drilling through the sound shale and the hard interbeds may be difficult. As such, intensive use of pneumatic rock splitting/breaking equipment, ripping machinery or other methods of loosening the bedrock may be required and should be available on site to assist in rock excavation.

3. Suggested Text for NSSP on "Installation of Pile Sockets"

For pile socket installation in the shale bedrock, in addition to augering equipment, rock coring equipment or pneumatic rock splitting/breaking equipment may be required to penetrate the hard interbeds.

Appendix F

Drawing

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

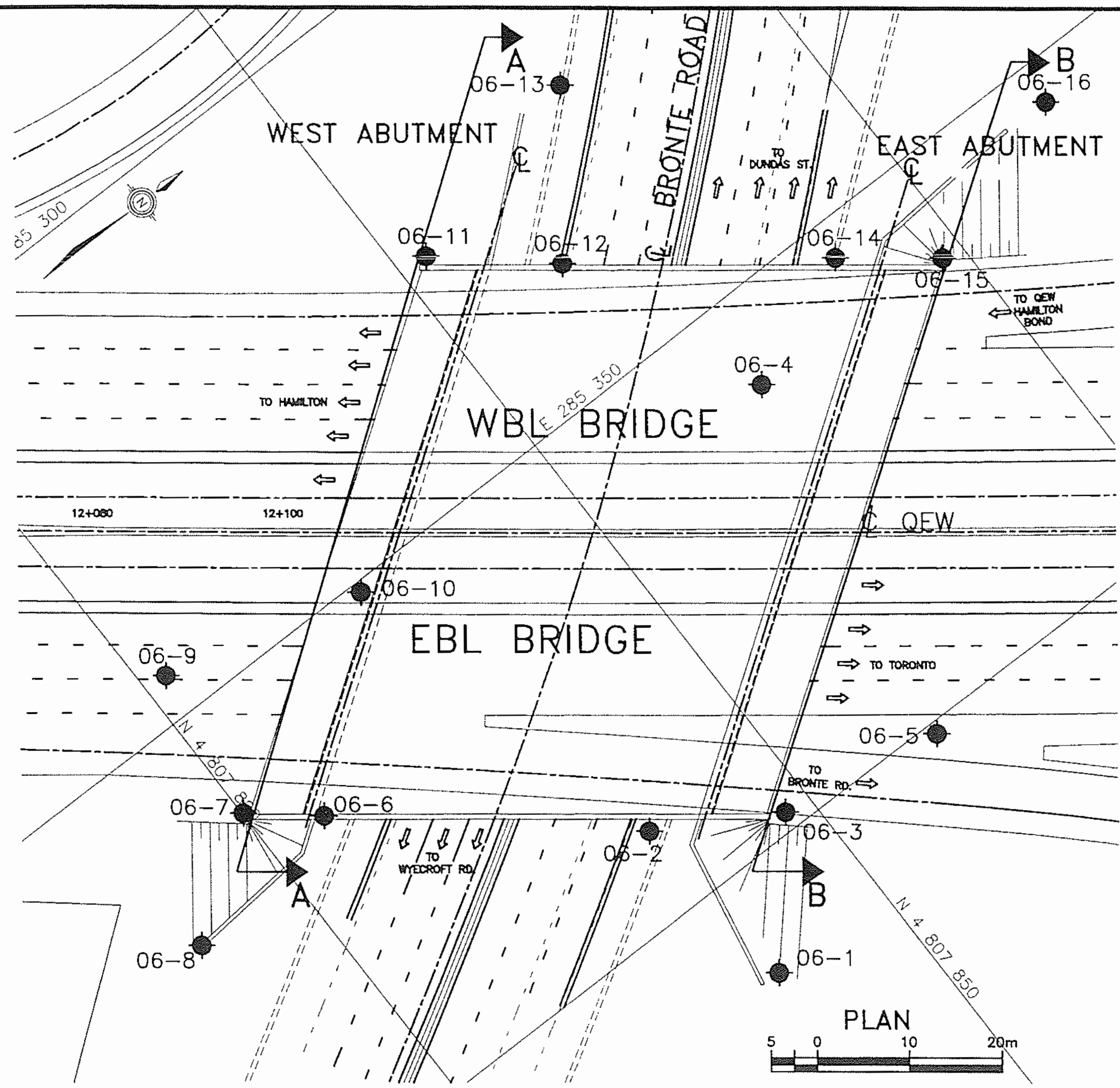
CONT No 2007–2026
GWP No.169–00–00

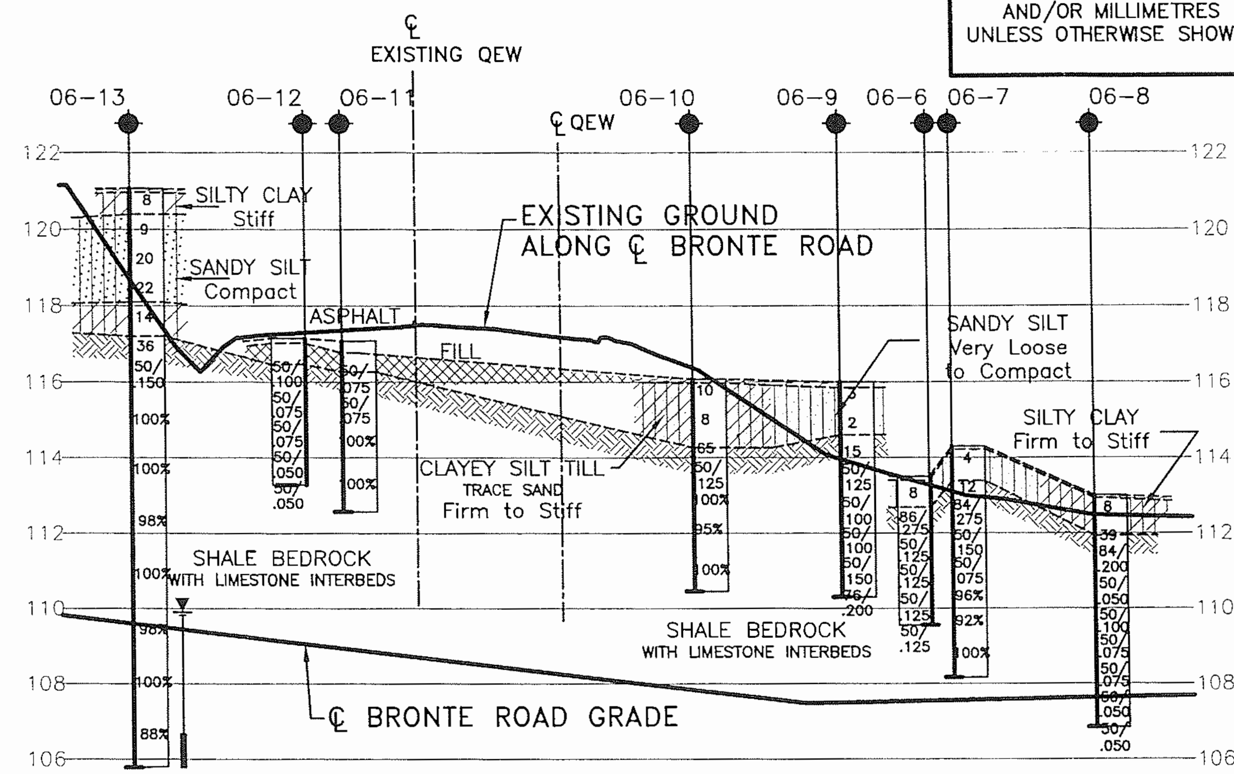
SHEET

QEW
BRONTE ROAD OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

McCORMICK RANKIN CORPORATION

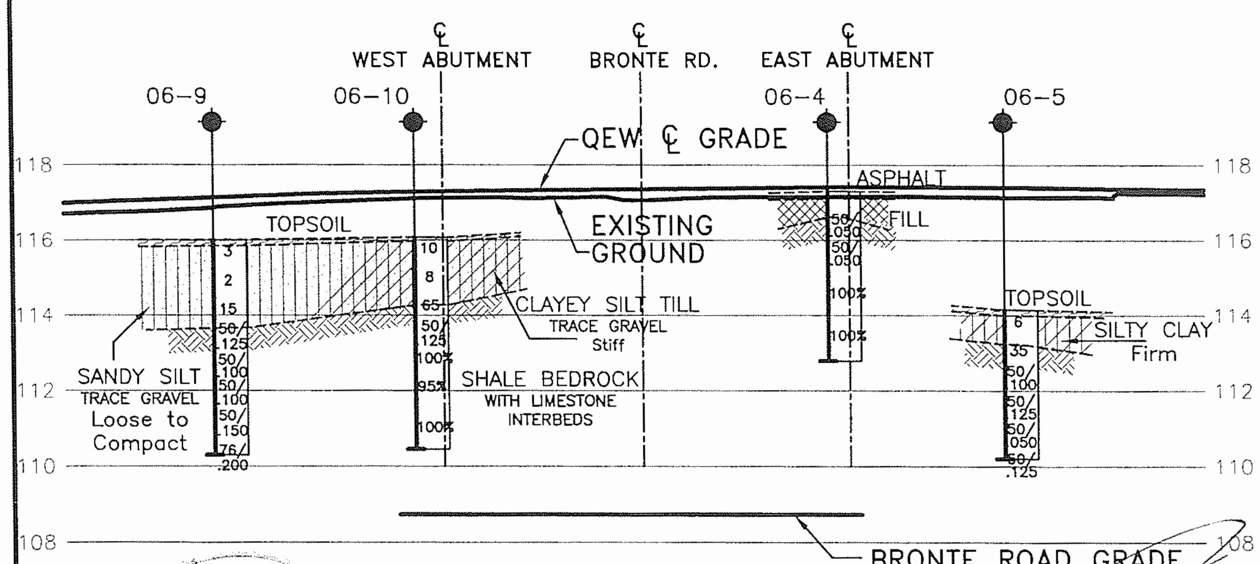
THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS





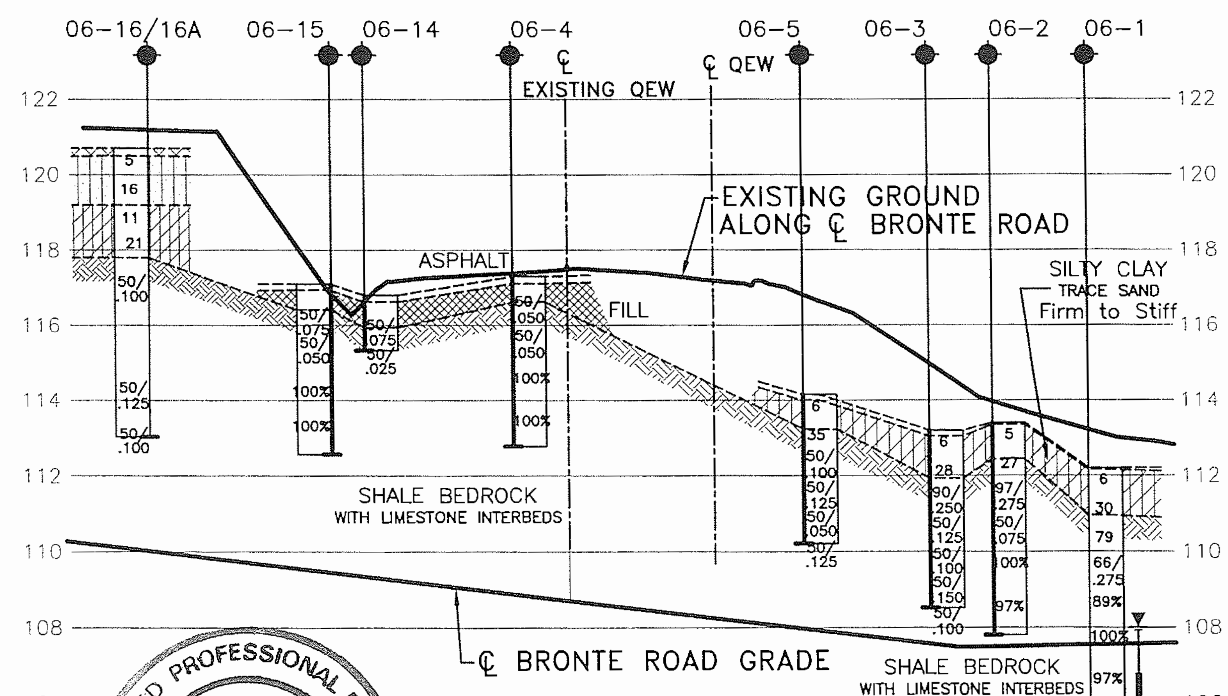
SECTION A–A

HOR 1:800
VER 1:200



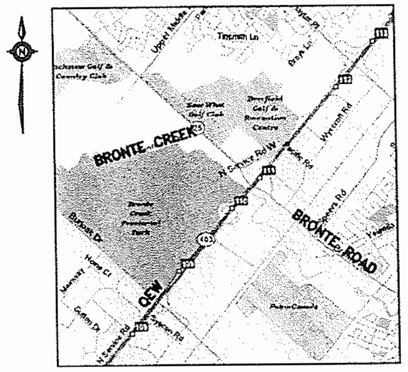
PROFILE ℄ QEW

HOR 1:800
VER 1:200



SECTION B–B

HOR 1:800
VER 1:200



KEYPLAN

LEGEND

Symbol	Description
●	BoreHole
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▼	Water Level
	Head Artesion Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
06–1	112.2	4 807 835.7	285 411.3
06–2	113.4	4 807 834.0	285 390.6
06–3	113.2	4 807 847.0	285 398.0
06–4	117.3	4 807 873.7	285 359.9
06–5	114.2	4 807 865.3	285 401.2
06–6	113.5	4 807 807.0	285 368.0
06–7	114.3	4 807 800.2	285 362.3
06–8	113.0	4 807 787.7	285 370.8
06–9	116.0	4 807 802.7	285 345.3
06–10	116.1	4 807 820.4	285 351.1
06–11	117.1	4 807 853.3	285 326.9
06–12	117.2	4 807 864.5	285 336.4
06–13	121.1	4 807 876.2	285 321.1
06–14	116.8	4 807 888.4	285 354.0
06–15	117.1	4 807 897.5	285 361.0
06–16/16A	120.7	4 807 916.8	285 354.5

-NOTE-
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

LICENSED PROFESSIONAL ENGINEER
S. PANG
Jan.31/07
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 31/07
PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS			
JAN 07	SKP		FINAL
JUL 06	SKP		ISSUED AS DRAFT FOR REVIEW
DATE	BY		DESCRIPTION

DESIGN SKP	CHK	CODE	LOAD	DATE JULY 06
DRAWN JHL	CHK	SITE	STRUCT	DWG 19–1351–63–1

DRAWING NAME: TED5163 – PLAN&PROFILE
CREATED: JULY 06
MODIFIED:

FILENAME: D:\Job Files\19\1351\63 QEW Bronte Rd\ted5163-Plan&Profile.dwg
PLOTDATE: Feb 01, 2007 – 11:58am