DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
GRADE SEPARATION STRUCTURE
STREET 'A' OVER S-E RAMP
QEW – BURLOAK DRIVE INTERCHANGE

Geocres Number:

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed grade separation structure to carry a new roadway (Street 'A') over the S-E ramp of the QEW-Burloak Drive interchange in Oakville, Ontario. The report includes investigation at the locations of retaining walls potentially required in connection with the structure and new roadway.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections, and a written description of the subsurface conditions. A model of the subsurface conditions was developed to describe the geotechnical conditions influencing design and construction of the foundations and approach embankments for the structure, as well as for associated retaining walls.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation.

2 SITE DESCRIPTION

The site is located in the southeast quadrant of the QEW-Burloak Drive interchange. The existing S-E ramp and South Service Road run roughly parallel and in an approximate north-south direction through the proposed grade separation structure location. The roadway embankments are built up some 1 to 3 m above the adjacent ground surface and are separated by a 1 to 2 m deep ditch.

The site is situated approximately 50 m east of Burloak Drive and the approach embankment to the Burloak Drive structure over QEW. About 50 m south of the site, the east branch of Sheldon Creek flows west to east through a culvert under the ramp and service road and then into a treed ravine. The land to the east of the site and north of the Sheldon Creek ravine consists of a relatively flat field.

Lands to the west of Burloak Drive are developed for commercial/industrial purposes. Lands to the immediate east are presently undeveloped. Bronte Creek Provincial Park is located north of the QEW.

The general site area is located within the physiographic region known as the Iroquois Plain, characterized by a gently sloping till plain locally overlain by deposits of former Lake Iroquois. The site lies in an area of silty clay to clayey silt till (Halton Till) overlying relatively shallow shale bedrock of the Queenston Formation.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing at the proposed location of the grade separation structure and retaining walls during the period November 11 to 25, 2005. The site investigation consisted of drilling and sampling 14 boreholes at the structure abutments and approaches, as well as five boreholes along the alignment of a potential retaining wall along proposed Street 'A' to the east of the structure.

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawings in Appendix D. The coordinates and elevations of the boreholes are given on these drawings and on the individual Record of Borehole Sheets in Appendix A.

The majority of the boreholes were terminated upon auger refusal in shale bedrock at depths of 2.6 to 7.8 m. Two boreholes at each abutment were advanced into shale bedrock by coring to total depths of 7.2 to 10.2 m, with a minimum 3.0 m of rock core recovered in each borehole.

Prior to the start of drilling, utility clearances were obtained for all borehole locations. Encroachment permits and road occupancy permits were also obtained.

Solid stem augers were used to advance the boreholes in overburden and into shale. Samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ rock coring equipment was used to recover core samples of the underlying bedrock.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The inspector logged the boreholes, visually examined the recovered samples, and transported them to Thurber's laboratory for further examination and testing. The recovered rock core was described in the field, packaged in core boxes with moist paper towels and parafilm wrap, and returned to our laboratory for examination and testing.

Standpipe piezometers, consisting of 19 mm PVC pipes with slotted tip, were installed in selected boreholes to monitor groundwater levels. The completion details are shown in Table 3.1.

Table 3.1 – Piezometer Details

Piezometer Location	Tip Depth/ Elevation (m)	Completion Details
BH05-1	6.1/111.1	Piezometer with 0.8 m tip installed at bottom of borehole. Sand filter to 4.9, holeplug and bentonite seal to 0.3.
BH05-7	6.1/111.8	Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 4.3, holeplug and bentonite grout to surface.
BH05-12	3.7/111.0	Piezometer with 0.8 m tip installed at bottom of borehole. Sand filter to 2.1, holeplug to surface.
BH05-16	4.0/111.3	Piezometer with 0.8 m tip installed at bottom of borehole. Sand filter to 3.0, holeplug and bentonite grout to 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.

Approximately 25% of the recovered samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing. The results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

Selected rock core samples were subjected to Point Load Testing to assist evaluation of the compressive strength of the bedrock.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and the Borehole Locations and Soil Strata Drawings in Appendix D for details of the encountered soil stratigraphy. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

5.1 Grade Separation Structure (BH05-1 to BH05-14)

The soil stratigraphy encountered at this site comprises topsoil or fill associated with the existing roadworks, overlying silty clay, underlain by shale bedrock. More detailed descriptions of the individual strata are presented below.

5.1.1 Topsoil

Topsoil was identified surficially in six boreholes drilled off of the roadway shoulders. The topsoil thickness was established only at the borehole locations and ranged from 100 to 200 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.1.2 Fill

Approximately 600 mm of granular roadbase material was encountered in eight boreholes drilled through the shoulders of the existing S-E ramp and South Service Road. At one location (BH05-7), a 75 mm thick asphalt surface was encountered.

Silty clay fill was encountered below the topsoil or granular roadbed fill in all boreholes except the approach holes and the northeast retaining wall hole (BH05-12 to 05-14). The fill extends to depths of 1.2 to 4.5 m (elevation 113.3 to 115.1 m). Standard penetration test (SPT) N-values obtained in the fill typically ranged from 9 to 20 blows/0.3 m penetration, indicating a stiff to very stiff consistency. N-values of up to 56 blows/0.3 m obtained at isolated locations probably reflect the presence of shale fragments in the fill. Moisture contents varied from 5 to 22%, typically 10 to 20%.

The results of grain size distribution analyses conducted on the clay fill are presented on Figure B1 of Appendix B. The results of Atterberg Limits testing, Figure B4, indicate that the fill is of low plasticity.

5.1.3 Silty Clay

Cohesive silty clay was encountered below the topsoil and fill in all boreholes at the site. The depth of the upper and lower boundaries of the clay varied with location; the upper boundary was contacted at depths ranging from 0.1 to 4.5 m (elevation 113.3 to 115.9 m), and the lower boundary was encountered at depths of 1.4 to 6.1 m (elevation 111.9 to 113.7 m).

The clay contains sand, gravel and shale fragments. Grain size distribution results for the clay, presented on the Record of Borehole sheets and Figures B2 and B3 of Appendix B, indicate that the percentage of sand and gravel size particles (larger than 75 μm) generally ranges from 18 to 24%, with three samples showing 2 to 6%. Atterberg Limits testing, Figures B5 and B6, indicates that the clay is of low to medium plasticity.

The consistency of the clay is typically hard (SPT N-values of 29 to 66 blows/0.3 m) with upper zones of firm to very stiff material (N-values of 7 to 23 blows/0.3 m). Moisture contents ranged from about 9 to 22%, with a higher value of 38% measured in a sample near the ground surface potentially containing organic material.

5.1.4 Shale Bedrock

Shale bedrock was contacted in all boreholes at depths of 1.4 to 6.1 m. The depths and elevations at which shale was contacted are summarized in Table 5.1.

Table 5.1 – Depth to Shale

Location	Borehole	Shale	
		Depth (m)	Elevation (m)
West Abutment			
Northwest Corner	05-1	5.1	112.1
Northeast Corner	05-2	4.6	113.6
Southeast Corner	05-5	6.1	113.4
Southwest Corner	05-6	3.3	113.7
East Abutment			
Northwest Corner	05-3	5.8	111.9
Northeast Corner	05-4	1.8	112.7
Southwest Corner	05-7	4.4	113.5
Southeast Corner	05-8	4.4	113.5
Retaining Walls			
Southwest	05-9	5.8	113.3
Southeast	05-10	5.5	112.1
Northwest	05-11	4.8	113.4
Northeast	05-12	2.4	112.3
West Approach	05-13	4.0	112.1
East Approach	05-14	1.4	112.0

In general, the boreholes were terminated in the shale bedrock upon auger refusal at depths of 2.6 to 7.8 m. A minimum length 3.0 m of shale core was recovered from two boreholes at each abutment.

The bedrock consists of reddish brown shale of the Queenston Formation. The shale is thinly bedded with occasional clay seams and limestone interbeds. In general, the bedrock is highly weathered in the upper 1 to 2 m and was easily penetrated by the drill augers. Below this depth, the rock becomes moderately weathered and refused further auger penetration.

Point Load tests conducted on the cores were generally not representative because the cores would easily split along the bedding planes at very low loads. Where valid tests were completed, the estimated unconfined compressive strengths of the cores varied widely from 1.6 to 109 MPa. Based on previous experience in the area, Queenston shale is typically a weak to very weak rock with medium to high strength interbeds.

Total core recovery (TCR) of the bedrock cores ranged from 90 to 100%. The Rock Quality Designation (RQD) of the shale core generally ranged from 53 to 77%, indicating a fair quality rock. However, RQD values of 0 and 22% were obtained in the initial core run in two boreholes (BH05-4 and 05-6), indicating the upper portion of the bedrock is very poor quality.

5.1.5 Water Levels

Water was observed at depths of 5.1 and 4.9 m (elevation 114.4 and 114.2 m) in two boreholes (BH05-5 and 05-9) drilled on the shoulder of the existing S-E ramp. This water is likely perched in the embankment fill above the less permeable silty clay. The groundwater levels measured in the piezometers installed in selected boreholes are shown in Table 5.2.

Table 5.2 – Measured Groundwater Levels

Location	Borehole	Date	Water Level (m)	
			Depth	Elevation
West Abutment, North End	05-1	06-Dec-2005	3.6	113.6
East Abutment, South End	05-7	06-Dec-2005	5.1	112.8
Northeast Retaining Wall	05-12	06-Dec-2005	1.5	113.2

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.2 Retaining Wall along Proposed Street 'A' (BH05-15 to BH05-19)

Boreholes 05-15 to 05-19 were drilled near the crest of the Sheldon Creek ravine slope, along the south side of the proposed roadway to the east of the grade separation structure. The soil stratigraphy encountered in these boreholes comprises topsoil or fill overlying silty clay till, underlain by shale bedrock at shallow depth. More detailed descriptions of the individual strata are presented below.

5.2.1 Topsoil

A 100 to 300 mm thick topsoil layer was identified surficially in four boreholes. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.2.2 Fill

A 200 mm layer of granular roadbase material was encountered in borehole 05-19 drilled through a driveway at the east end of the site.

5.2.3 Silty Clay

Cohesive silty clay was encountered below the topsoil and fill in all boreholes. The clay layer was 0.5 to 1.5 m thick and underlain by bedrock. The consistency of the clay is typically hard (SPT N-values exceeding 40 blows/0.3 m), locally stiff (N-value of 9) at one location (BH05-15).

Grain size distribution results for the clay are presented on Figure B3 of Appendix B. Atterberg Limits testing, Figure B6, indicates that the clay is of low to medium plasticity. Moisture contents were typically about 10%, with higher values of 43 and 21% measured in samples from borehole 05-15.

5.2.4 Shale Bedrock

Reddish brown shale of the Queenston Formation was contacted in all boreholes at depths of 0.7 to 1.7 m (elevation 111.6 to 115.5 m). The boreholes were terminated in the shale bedrock upon auger refusal at depths of 2.6 to 5.0 m. In general, the bedrock is highly weathered in the upper portions and becomes less weathered with depth

5.2.5 Water Levels

Water was not observed in the boreholes during or upon completion of drilling. On December 6, 2005, water was measured at 2.4 m depth (elevation 112.9 m) in the pizometer installed in borehole 05-16.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

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

A single span, 15.8 m long by 17.5 m wide reinforced concrete structure is proposed at this site. The preliminary General Arrangement drawing indicates that the road grade will be near Elevation 122.0 m on new Street 'A' over the structure and at approximate elevation 115.5 m on the new S-E Ramp.

At the west approach, the original ground lies between Elevation 117.0 to 119.5 m, resulting in an approach fill approximately 2.5 to 5.0 m high. At the east approach, the original ground lies near Elevation 114.0 m, resulting in an approach fill approximately 8.0 m high.

Construction of the new S-E Ramp will include abandoning the existing South Service Road. At the proposed structure location, existing road grade on the South Service Road is near Elevation 118.0 m. A cut of about 2.5 m below the existing pavement surface is therefore required to establish the new ramp grades under the structure.

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

8 STRUCTURE FOUNDATIONS

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme is recommended.

Based on the results of the exploratory boreholes drilled at the proposed abutment locations, the stratigraphy consists of topsoil or fill associated with the existing roadworks, overlying silty clay, underlain by shale bedrock at depths of 1.4 to 6.1 m.

Initial consideration was given to the following foundation types:

- Spread footings on native soil
- Spread footings on shale bedrock
- Spread footings on engineered fill
- Caissons (drilled shaft piles)
- Driven steel H-piles

Appendix C contains a table presenting a comparison of the technical advantages and disadvantages of the different foundation schemes at this site.

8.1 Spread Footings

8.1.1 Footings on Native Soil

The existing fill and upper zones of firm to stiff silty clay at the site are considered unsuitable materials to support spread footings due to the variable consistency, the low bearing resistance available, and the potential for comparatively large settlements. Extending footings down to bear on the underlying very stiff to hard silty clay could be considered.

Provided a minimum footing width of 2 m is maintained, footings bearing on the hard silty clay may be designed for a concentric, vertical geotechnical resistance of 600 kPa at factored ULS and a resistance of 400 kPa at SLS. Footings designed using these resistances should be founded at or below the following elevations:

Table 8.1 – Maximum Elevation of Footings on Hard Native Clay

Location	Borehole Number	Maximum Founding Elevation (m)
West Abutment:		
North End	05-1, 05-2	113.6
South End	05-5, 05-6	114.3
East Abutment:		
North End	05-3, 05-4	112.8
South End	05-7, 05-8	113.5 (shale)
Retaining Walls:		
Southwest	05-9	113.9
Southeast	05-10	112.7
Northwest	05-11	113.9
Northeast	05-12	113.2

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

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm.

The lateral resistance of the footings founded on hard silty clay may be computed using an unfactored friction coefficient of 0.55. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Following inspection and approval of the excavation base, the founding surfaces must be covered by placement of a minimum 50 mm thick mud slab of concrete as protection against softening of the clay and degradation of the shale.

Where subexcavation is required to remove unsuitable material from below the design founding level, the design founding level should be re-established using mass concrete. In order to avoid excessive cover over the east abutment footing, mass concrete may also be used under the footing at the north end of this abutment to achieve the required founding level.

In view of the relatively shallow depth to hard silty clay and the relatively high bearing resistance available, construction of spread footings on the native soil is a favoured option from a geotechnical viewpoint.

8.1.2 Footings on Shale Bedrock

If required, an increased bearing resistance could be achieved by extending footings down through the fill and silty clay to bear on the underlying shale bedrock. Provided a minimum footing width of 1.5 m is maintained, footings constructed at least 0.5 m below the shale surface may be designed for a concentric, vertical geotechnical resistance of 1,000 kPa at factored ULS. Footings designed using this resistance should be founded at or below the following elevations:

Table 8.2 – Maximum Elevation of Footings on Shale

Location	Borehole Number	Maximum Founding Elevation (m)
West Abutment:		
North End	05-1, 05-2	111.6
South End	05-5, 05-6	112.9
East Abutment:		
North End	05-3, 05-4	111.4
South End	05-7, 05-8	113.0
Retaining Walls:		
Southwest	05-9	112.8
Southeast	05-10	111.6
Northwest	05-11	112.9
Northeast	05-12	111.8

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

The bearing resistance at SLS (25 mm of settlement) is not expected to govern design of spread footings on shale bedrock.

The lateral resistance of the footings founded on shale may be computed using an unfactored friction coefficient of 0.70. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

In view of the increased depth if excavation required to extend footings to shale and the relatively high bearing resistance available in the overlying silty clay, construction of spread footings on bedrock is unlikely to be the favoured option.

8.1.3 Footings on Engineered Fill

Construction of spread footings on engineered fill constructed over the native, very stiff to hard silty clay may be considered. The fill should be at least 1.5 m thick below the footing and have an underside no higher than the elevations shown in Table 8.1. All fill and softened (firm) zones of the native silty clay should be removed prior to placement of the engineered fill. The fill should be constructed in accordance with the geometry illustrated in Figure 1, Appendix C.

The engineered fill must consist of OPSS Granular A placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content.

Provided a minimum footing width of 2 m is maintained, a footing bearing on the engineered fill may be designed for a concentric, vertical geotechnical resistance of 900 kPa at factored ULS and a resistance of 350 kPa at SLS.

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

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 12 mm across the width of the structure.

The lateral resistance of the footings founded on granular engineered fill may be computed using an unfactored friction coefficient of 0.7. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

Construction of an engineered fill pad may be less economical than extending the footings down to hard native clay, and therefore this option may be less favourable than spread footings founded on native silty clay.

8.2 Augered Caissons (Drilled Shafts)

The subsurface conditions at this site are considered to be suitable for installation of caisson foundations socketed into bedrock.

The caissons will develop resistance through a combination of sidewall shear and end bearing in the rock socket. Values for factored base resistance and sidewall shear recommended for computation of the axial geotechnical resistance of the caisson are presented in Table 8.3.

Table 8.3 – Caisson End Bearing and Sidewall Resistance

Depth Below Shale Surface (m)	Factored End Bearing Resistance at ULS (kPa)	Factored Sidewall Resistance at ULS (kPa)
0.5 to 2.5 m	2,000	200
Greater than 2.5 m	3,000	400

The factored axial resistance computed for selected caisson diameters and socket lengths, based on the factored resistance values indicated in Table 8.3, are shown in Table 8.4.

Table 8.4 – Axial Geotechnical Resistance for Selected Caisson Geometries

Socket Diameter (m)	Socket Length (m) Below Shale Surface	Factored Geotechnical Resistance at ULS (kN)
1.2	2.0	3,400
	3.5	6,400
	5.0	8,700
1.8	2.0	6,800
	4.0	13,300
	6.0	17,800
2.4	2.5	16,600
	5.0	24,100
	7.5	31,600

The elevations at which shale was contacted in the boreholes are provided in Table 5.1. In calculating the above figures, the upper 0.5 m of shale was ignored to allow for variations in the degree of weathering in the upper part of the shale formation.

The SLS condition is not expected to govern design of caissons bearing on bedrock. This should be confirmed prior to finalizing design, when details of the socket geometry and loading per caisson are established.

The sidewall shear values presented in Table 8.3 apply to caissons with a minimum centre-to-centre spacing of three caisson diameters. If the caisson spacing is less than three diameters, the sidewall resistance values should be reduced. A reduction of 15% should be applied for each caisson located at a centre-to-centre spacing of two diameters; shear reduction values between two and three diameter spacings may be interpolated. Caissons designed using sidewall shear should not be spaced closer than two caisson diameters.

In general, the soil deposits at the abutments are thin and highly over-consolidated. Settlements induced in the native soils around the caissons by construction of the approach embankments will be substantially complete as construction of the embankment is completed. Post-construction downdrag on the caissons is therefore not considered to be an issue at this site.

The lateral resistance of the caissons may be calculated using the coefficient of horizontal subgrade reaction k_s and ultimate lateral resistance p_{ult} estimated as follows:

k_s = $67\ s_u / D$ (kN/m^3)

p_{ult} = $2\ s_u$ (kPa) at surface, increasing linearly to
$9\ s_u$ (kPa) at a depth of 3 caisson diameters and below

where s_u = undrained shear strength
= 250 kPa for hard clay below the elevations indicated in Table 8.1
= 450 kPa for the upper 2 m of weathered shale below the shale surface
= 1,000 kPa for shale at least 2 m below the shale surface

The recommended parameters may be used for numerical analysis of the interaction between the caisson and surrounding soil. The lateral pressures obtained by the numerical analysis should not exceed the ultimate lateral resistance.

The spring constant K_s and ultimate spring load P_{ult} values for numerical analysis can be obtained by multiplying the k_s and p_{ult} values by the caisson diameter and the vertical distance between nodal points of the numerical model mesh along the caisson.

The modulus of subgrade reaction may have to be reduced, based on the caisson spacing. The reduction factors to be used for caissons oriented perpendicular or parallel to the direction of loading are provided in Table 8.5. Intermediate values may be obtained by linear interpolation.

Table 8.5 – Subgrade Reaction Reduction Factors for Caisson Spacing

Condition	Caisson Spacing, Centre to Centre*	Reduction Factor
Caissons oriented ***perpendicular*** to direction of loading	4D	1.0
	1D	0.5
Caissons oriented ***parallel*** to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

* where D is the diameter of caisson

The Contractor should be alerted to the following potential concerns during construction of the caissons:

- The fill and till soils are likely to contain shale and limestone fragments, and possibly cobbles and boulders, that must be penetrated or removed during caisson installation.
- The shale contains hard interbeds of limestone, siltstone or calcareous shale that may slow production and/or require the use of coring or breaking equipment to penetrate.
- Use of a caisson liner may be required to maintain the caisson sidewalls in areas of deeper fill or if zones of perched water are encountered. A caisson liner will be required to enable cleaning and down-hole inspection of the caisson socket.

8.3 Driven Steel Piles

The use of driven steel H-pile foundations to support the grade separation structure may be considered. However, considering the relatively shallow depth to bedrock at the site, pre-augering will likely be required to achieve adequate pile embedment into the shale, and therefore the use of pile foundations is not recommended at this site.

HP 310 X 110 piles extended into shale a minimum 5 m below the pile cap by pre-augering and driving should be designed on the basis of an axial geotechnical resistance of 1,800 kN at factored ULS. The SLS condition will not govern design of piles in rock.

The piles should be driven to practical refusal with a tip depth of at least 2.0 m below the shale surface. The structural capacity of the pile should not be exceeded and driving should be carefully controlled to avoid damaging the pile by overdriving.

Since the piles will be driven into bedrock, the tips of all piles should be fitted with cast steel H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

The Contractor should be alerted to the fact that pre-augering will likely be required to install the piles to an adequate depth.

In general, the soil deposits at the abutments are thin and highly over-consolidated. Settlements induced in the native soils around the piles by construction of the approach embankments will be substantially complete as construction of the embankment is completed. Post-construction downdrag on the piles is therefore not considered to be an issue at this site.

The lateral resistance of the piles may be calculated using the methods outlined previously for caissons, substituting the pile width, B, for the caisson diameter, D. The total horizontal passive resistance of a single pile used in design should not exceed values of 260 kN at factored ULS and 200 kN at SLS.

In the case of conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

8.4 Recommended Foundation System

The recommended foundation system for all foundation elements at this site is spread footings bearing on hard clay till or shale bedrock at relatively shallow depths.

8.5 Abutment Type

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional or semi-integral abutments supported on spread footings. Integral abutments could be considered but would require installation of driven pile foundations by pre-augering into bedrock.

If integral abutment design is desired, the piles must possess flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will typically lie in fill, clay or shale, which may compromise the required flexibility. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP filled with sand (for a "true abutment" supported on piles) or by concentric CSPs in accordance with standard integral abutment design procedures (for a "false abutment").

8.6 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.2 m. Although the shale is geologically defined as rock, protection against frost action must also be provided for the bedrock.

It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation. A 25 mm thickness of rigid, extruded polystyrene insulation is equivalent to 600 mm of earth cover. Synthetic insulation must be covered to provide protection where it is used.

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the fill and upper 1.0 m of native clay at this site may be classed as Type 3 soil. The hard clay till may be classed as Type 1 soil. Near vertical sideslopes may be employed for excavation in the shale.

Roadway protection should be supplied in accordance with OPSS 539 and designed for Performance Level 2. Soil parameters for design are given in Section 14.

Use of a hydraulic excavator should be suitable for trench excavation. Provision should be made for handling of the pavement materials, possible obstructions in the fill, and fragments of shale and limestone in the clay during excavation.

Excavation of the upper 1 to 2 m of the shale should be possible using heavy excavation equipment supplemented by rippers or pneumatic rock breakers to penetrate layers of hard material. The shale becomes less weathered with depth and intensive use of pneumatic/hydraulic breakers will likely be required.

10 UNWATERING

Water was measured at elevations 112.8 to 113.6 m in the piezometers installed at the proposed structure locations. Considering the consistency and relatively low permeability of the soils on site, dewatering using sumps and pumps is considered feasible. The possibility exists that localized zones of perched water may be encountered in the fill, or concentrated seepage may be experienced from seams or fractures in the shale bedrock. The design of any dewatering system that may be required is the responsibility of the Contractor.

11 APPROACH EMBANKMENTS

The soil conditions governing stability of the approach embankments consist of existing stiff to very stiff embankment fill and a thin layer of very stiff to hard silty clay, underlain by bedrock. The proposed embankment height ranges from approximately 2.5 to 5.0 m at the west approach and 8.0 m at the east approach.

The embankment foundation soils are assessed to provide satisfactory resistance to instability under the loading imposed by maximum 5.0 to 8.0 m high embankments. Earth fill slopes inclined at 2H:1V are considered suitable.

Considering the embankment height and consistency of the foundation soils, settlement induced by the embankment loading is not a concern. Completing embankment construction three months in advance of road paving is recommended as a good construction practice to minimize any time-dependent settlement due to consolidation or particle re-organization in the embankment fill itself.

All topsoil and organic soils should be stripped from the footprint of the approach fills. Particular attention should be paid to existing ditches to remove all softened material.

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

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 RETAINED SOIL SYSTEMS

RSS walls used in conjunction with bridge abutments must be "High Performance" and "High Appearance". Therefore it is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill.

Provided proper ground preparation is carried out prior to construction of the walls, RSS systems are considered suitable for the subsurface conditions at this site and are expected to meet the aesthetic and structural requirements. The following recommendations are presented for RSS design if required:

- Topsoil, fill, and any soft/wet native material should be stripped from the footprint of the RSS.
- The RSS must be founded on the very stiff to hard silty clay or on Granular A fill placed to establish the design founding level of the wall. The highest permitted base levels for the underside of the wall or the Granular A fill are indicated in Table 12.1.

Table 12.1 – Maximum Elevation at Underside of Wall Base or Granular A Fill

Location	Borehole Number	Maximum Founding Elevation (m)
West Abutment:		
North End	05-1	115.0
South End	05-5	115.0
Approach	05-13	115.3
East Abutment:		
North End	05-4	113.3
South End	05-8	114.4
Approach	05-14	112.8

- Fill placed under the RSS mass to achieve the design founding level must be placed as engineered fill, consisting of OPSS Granular "A" compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered fill is specified to establish the wall design founding level where the design level is above the elevation indicated in Table 12.1; a minimum granular thickness is not required to achieve the recommended bearing resistances.
- Walls founded on the very stiff to hard silty clay or on Granular A fill placed to establish the wall base level should be designed for a factored bearing resistance of 300 kPa at ULS and a bearing resistance of 200 kPa at SLS.
- If required to accommodate the design, wall base levels may be lower than the maximum elevations indicated in Table 12.1.
- Sliding resistance along the base of the wall or engineered granular fill in contact with the clay may be estimated using an ultimate friction coefficient of 0.55.
- A Non Standard Special Provision (NSSP) for the design, supply and construction of the RSS is provided in Appendix C.
- The global stability of the RSS walls must be assessed prior to finalizing the design.

The supplier of the proprietary RSS system must demonstrate that it will meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design.

13 BACKFILL TO ABUTMENTS

Backfill to the abutments should consist of Granular A or Granular B material. The backfill must be in accordance with OPSS 902 as amended by Special Provision 902S01, and placed to the extents shown in OPSD 3501.000.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with OPSS 501.06.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000.

14 EARTH PRESSURE COEFFICIENTS

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC (2000) but generally are given by the expression:

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

where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 14.1)

γ = unit weight of retained soil (see Table 14.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 14.1.

Table 14.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32^\circ$, $\gamma = 21.2$ 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	0.43*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

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 Table 14.1 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 Canadian Highway Bridge Design Code.

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 1.7 m for Granular A or Granular B Type II.

For integral abutment design, the following values of modulus of horizontal subgrade reaction, k_s, may be used to calculate spring constants for the backfill:

$k_s = 4{,}500\ z/h\ (\text{kN/m}^3)$ for Granular B Type I

$k_s = 5{,}600\ z/h\ (\text{kN/m}^3)$ for Granular A or Granular B Type II

where: z = depth from top of abutment wall to point of interest (m)

h = full height of abutment wall (m)

15 RETAINING WALL ALONG PROPOSED STREET 'A'

Details concerning the alignment, height or design founding levels of the retaining walls have not been established. The subsurface conditions encountered in the boreholes drilled along the south side of proposed Street 'A' comprise a relatively thin layer of silty clay overlying shale bedrock at depths of 0.7 to 1.7 m.

Based on the borehole information, it is recommended that the proposed wall be supported on spread footings founded on the hard silty clay or shale bedrock. Provided a minimum footing width of 1.5 m is maintained, footings bearing on the hard silty clay or at/below the shale surface may be designed for a concentric, vertical geotechnical resistance of 600 kPa at factored ULS and a resistance of 400 kPa at SLS.

The consistency of the overlying silty clay is variable and therefore footings designed using these resistances should be founded at or below the following elevations:

Table 15.1 – Maximum Elevation of Wall Footings

Borehole Number	Maximum Founding Elevation (m)
05-15	111.6
05-16	114.5
05-17	114.0
05-18	114.5
05-19	115.5

The resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC (2000) Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm.

The proposed wall alignment will be at or slightly below the crest of the Sheldon Creek ravine slope. When further details regarding the position of the wall are established, the slope conditions and impact on wall foundation design should be reviewed. In this regard, it may be necessary to increase the embedment depth, increase the setback distance from the slope face, or reduce the design bearing resistance for footings located in close proximity to the slope.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.70 when founded on shale and 0.55 when founded on hard clay. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

Following inspection and approval of the excavation base, the founding surfaces must be covered by placement of a minimum 50 mm thick mud slab of concrete as protection against softening of the clay and degradation of the shale.

Backfill to the retaining walls should consist of Granular A or Granular B material. The lateral earth pressures acting on the wall should be computed using the equation and parameters presented in Section 14.

16 CONSTRUCTION CONCERNS

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

- Undulations in the shale surface or variations in the consistency of the native silty clay. The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete.
- Perched water in the existing embankment fill over the less permeable native clay.
- Excavation of the shale bedrock is likely to require the use of rock excavation methods such as pneumatic rock breakers to penetrate hard limestone interbeds.
- The presence of the limestone interbeds may reduce productivity during caisson augering, and the Contractor should be prepared to use coring equipment or other methods to penetrate thicker bands if necessary.
- Adequate cleaning and unwatering of caissons.
- Exposed bearing surfaces must be protected to prevent softening of the clay or degradation of the shale.
- Pre-augering will be required to install driven piles if a pile foundation system is selected.

Appendix A

Record of Borehole Sheets

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

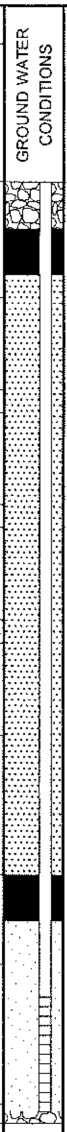
RECORD OF BOREHOLE No 05-1

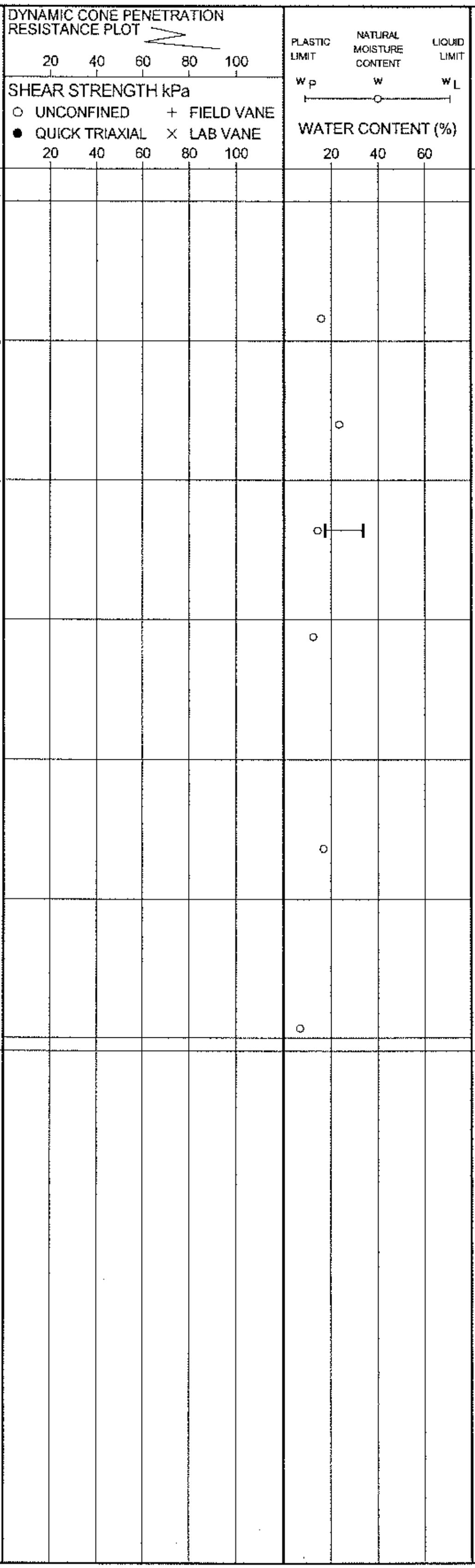
1 OF 1

METRIC

W.P. 19-1351-86
HWY QEW
DATUM Geodetic
LOCATION N 4 806 124.2 E 284 169.5
BOREHOLE TYPE Solid Stem Augers
DATE 16.11.05 - 16.11.05
ORIGINATED BY SLL
COMPILED BY WM
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
117.2 0.0	**TOPSOIL** (100mm)									
0.1	Silty **CLAY**, some sand, trace gravel Stiff Dark Brown Moist (FILL)				117					
		1	SS	11						
					116					
		2	SS	14						
115.0 2.2	Silty **CLAY**, some sand, trace gravel Very Stiff to Hard Brown Moist				115					
		3	SS	29			2	20	50	28
					114					
		4	SS	66						
					113					
		5	SS	65						
112.1 5.1	Highly weathered, very weak, reddish brown **SHALE**				112					
		6	SS	50/.125	111					
110.9 6.3	END OF BOREHOLE AT 6.33m. AUGER REFUSAL AT 6.33m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.76m slotted screen.									





WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
06.12.05	3.60	113.6

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

$\overset{20}{15 \oplus 5}_{10}$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 05-2

1 OF 2 **METRIC**

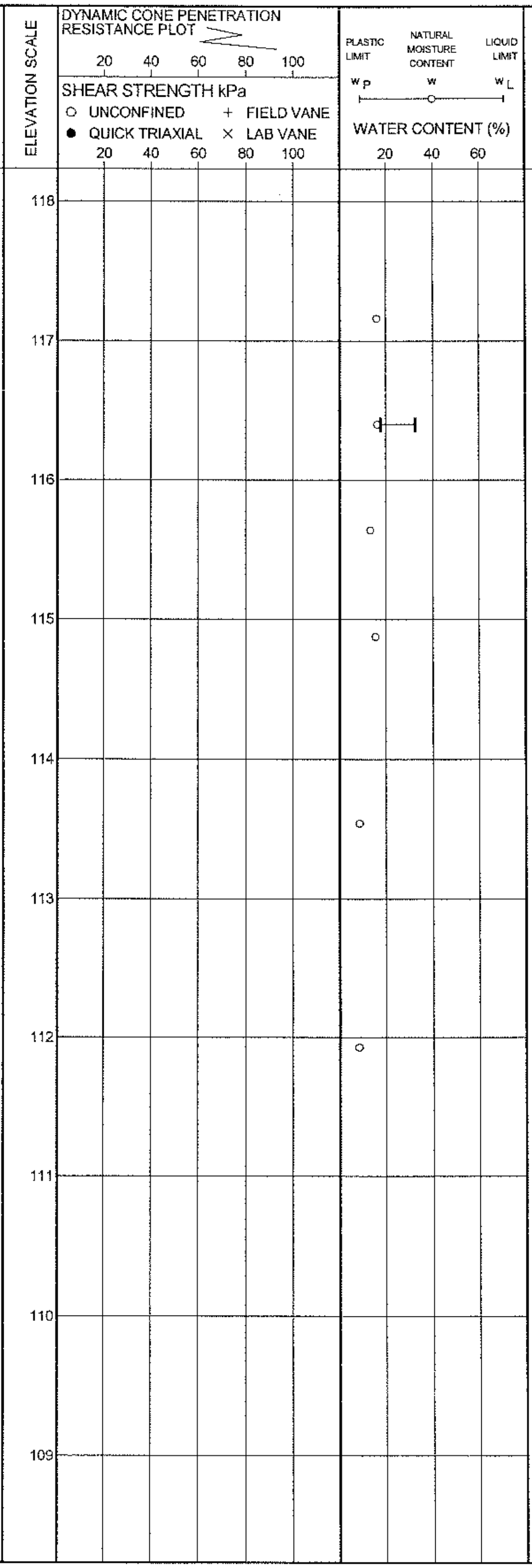
W.P.	19-1351-86	LOCATION	N 4 806 127.5 E 284 172.8	ORIGINATED BY	JHL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	24.11.05 - 24.11.05	CHECKED BY	MRA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
118.2 / 0.0	**CRUSHED LIMESTONE**
117.6 / 0.6	Silty **CLAY**, some sand, trace gravel Stiff to Very Stiff Reddish Brown Moist (FILL)
114.7 / 3.5	Silty **CLAY**, trace sand Firm Brown Moist
113.6 / 4.6	Highly weathered, very weak to weak, reddish brown **SHALE**, with clay seams Moderately weathered limestone interbeds at 7.32m to 7.37m, 7.75m to 7.79m, 7.87m to 7.89m, 8.69m to 8.71m, 8.79m to 8.81m, 8.92m to 8.94m, 9.73m to 9.80m, 9.91m to 9.93m

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11	
2	SS	19	6 21 46 27
3	SS	14	
4	SS	9	
5	SS	95/ .225	
6	SS	79/ .250	
1	RUN		RUN 1# TCR=100%, SCR=97%, RQD=54%, UCS=12MPa at 7.32m, UCS=73MPa at 7.70m
2	RUN		RUN 2# TCR=95%, SCR=92%, RQD=53%, UCS=37MPa at 9.02m, UCS=1.6MPa at 9.65m



GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m3

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-2

2 OF 2

METRIC

W.P.	19-1351-86	LOCATION	N 4 806 127.5 E 284 172.8	ORIGINATED BY	JHL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	24.11.05 - 24.11.05	CHECKED BY	MRA

SOIL PROFILE			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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
108.0											
10.2	END OF BOREHOLE AT 10.21m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.						108				

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

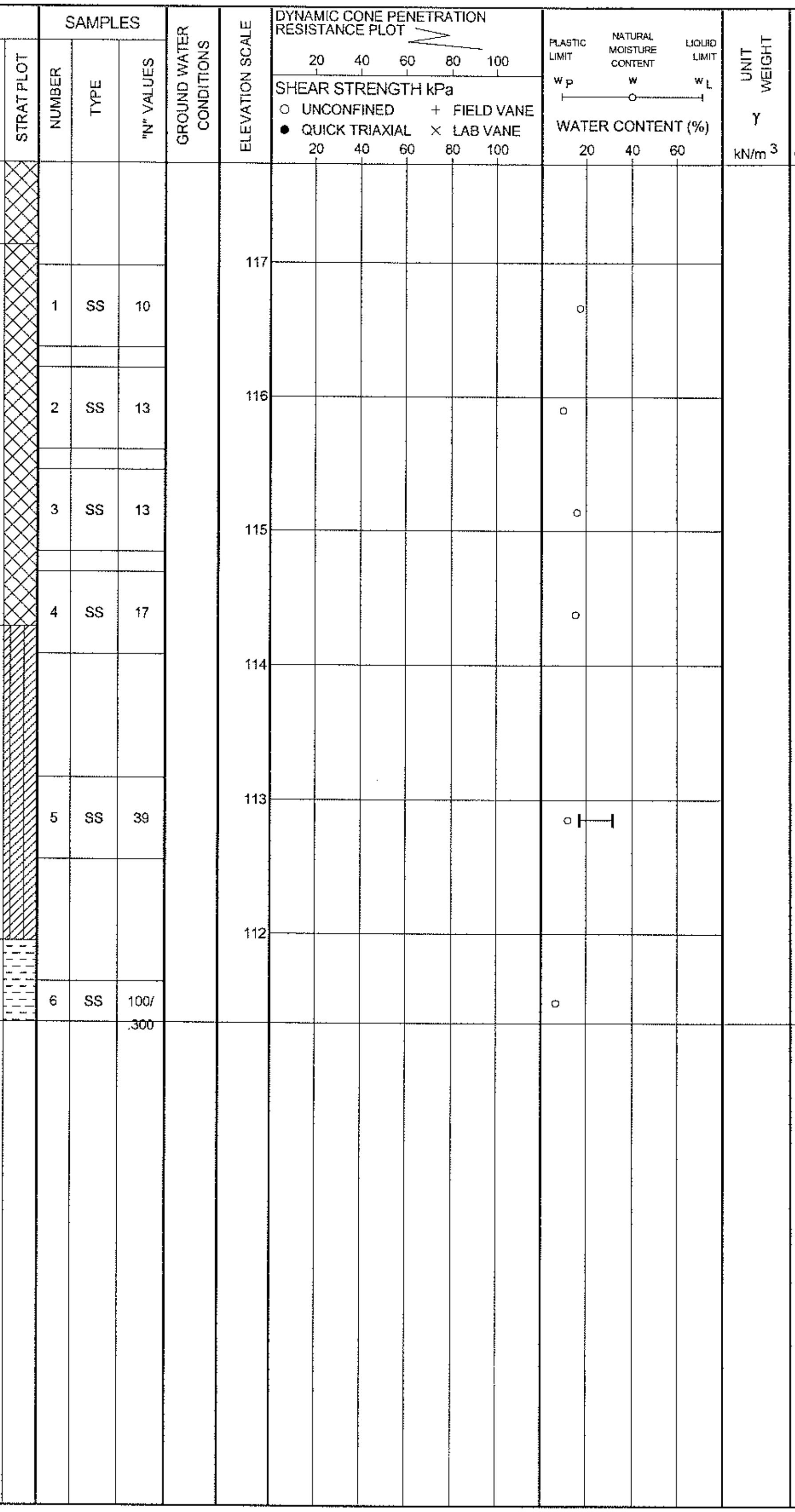
RECORD OF BOREHOLE No 05-3

1 OF 1

METRIC

W.P. 19-1351-86	LOCATION N 4 806 134.6 E 284 179.9	ORIGINATED BY JHL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 23.11.05 - 23.11.05	CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.7 0.0	**CRUSHED LIMESTONE**					
117.1 0.6	Silty **CLAY**, some sand, some gravel Stiff Reddish Brown Moist (FILL)	1	SS	10		
		2	SS	13		
		3	SS	13		
		4	SS	17		
114.2 3.5	Silty **CLAY**, some sand, trace gravel, with shale fragments Very Stiff to Hard Reddish Brown Moist	5	SS	39		1 23 50 26
111.9 5.8	Highly weathered, very weak, reddish brown **SHALE**	6	SS	100/ .300		
111.3 6.4	END OF BOREHOLE AT 6.40m. AUGER REFUSAL AT 6.40m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.					



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-4

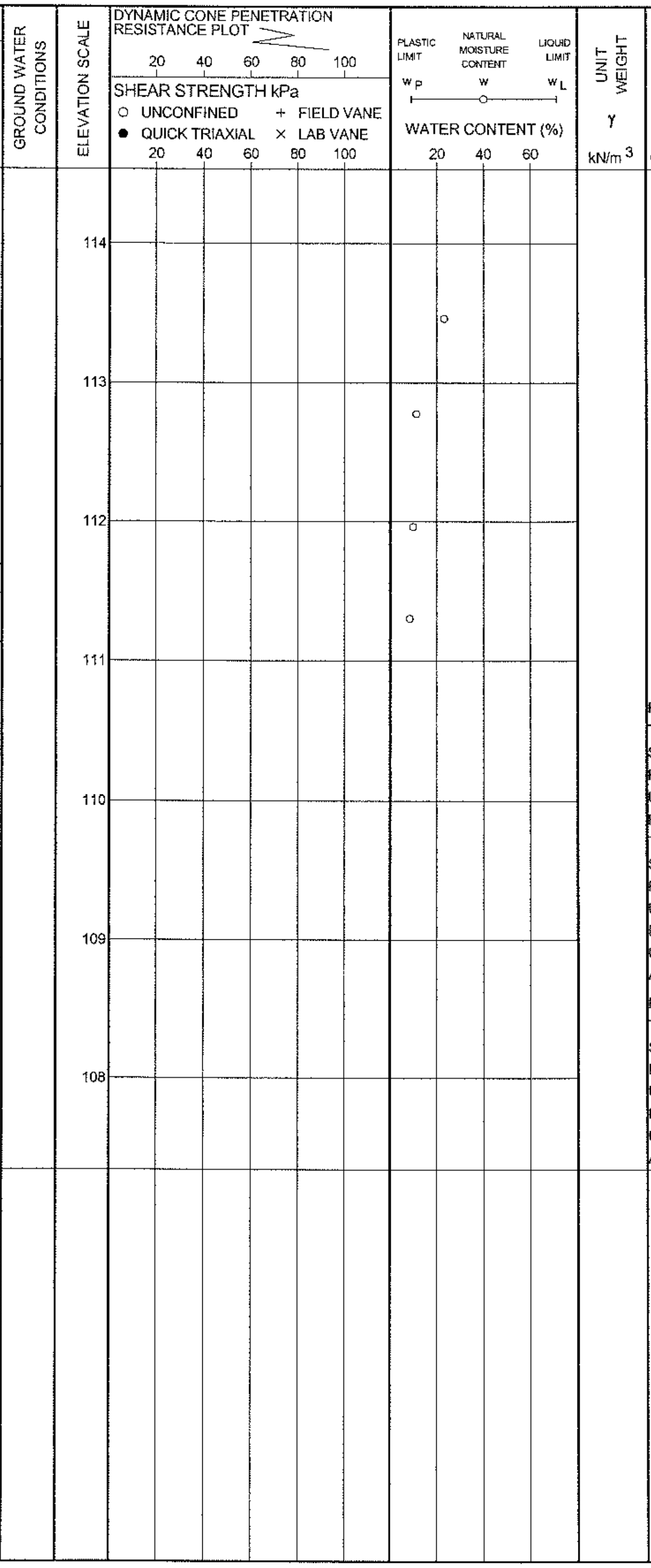
1 OF 1 **METRIC**

W.P. 19-1351-86
LOCATION N 4 806 140.1 E 284 185.6
ORIGINATED BY JHL

HWY QEW
BOREHOLE TYPE Solid Stem Augers and NQ Rock Core
COMPILED BY WM

DATUM Geodetic
DATE 14.11.05 - 14.11.05
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.5					
0.0	**TOPSOIL** (200mm)				
0.2	Silty **CLAY**, some sand Very Stiff Brown Moist (FILL)	1	SS	17	
113.3 1.2	Silty **CLAY**, some sand Very Stiff to Hard Brown Moist	2	SS	100	
112.7 1.8	Highly weathered, very weak to weak, reddish brown **SHALE**, with clay seams	3	SS	53	
		4	SS	100/ ~~200~~	
		1	RUN		RUN 1# TCR=100%, SCR=71%, RQD=0%, UCS=MPa
	Moderately weathered limestone interbeds at 4.17m to 4.21m, 4.42m to 4.44m, 4.93m to 4.98m, 5.18m to 5.23m, 6.02m to 6.12m, 6.14m to 6.19m, 6.63m to 6.73m, 6.86m to 6.91m vertical joints at 5.18m to 5.28m, 5.54m to 5.67m, 5.66m to 5.71m	2	RUN		RUN 2# TCR=100%, SCR=100%, RQD=76%, UCS=26MPa Diametral, UCS=46MPa Axial
		3	RUN		RUN 3# TCR=100%, SCR=99%, RQD=75%, UCS=12MPa Diametral, UCS=19MPa Axial
107.3 7.2	END OF BOREHOLE AT 7.18m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.				



+ 3, × 3: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

ONTMT4S 5186.GPJ 06/02/06

Ministry of Transportation
Ontario

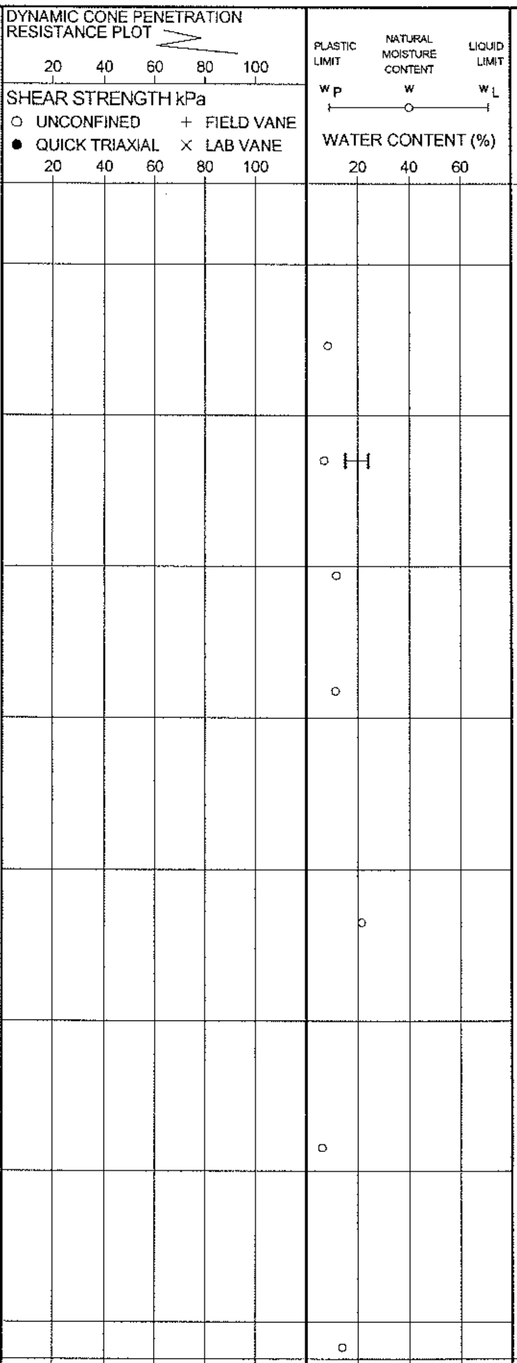
THURBER

RECORD OF BOREHOLE No 05-5

1 OF 1 **METRIC**

W.P. 19-1351-86
LOCATION N 4 806 103.6 E 284 172.9
ORIGINATED BY JHL
HWY QEW
BOREHOLE TYPE Solid Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 25.11.05 - 25.11.05
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
119.5									
0.0	**CRUSHED LIMESTONE**				119				
118.9 0.6	Clayey **SILT** to Silty **CLAY**, trace sand, with shale fragments, occasional limestone fragments Stiff to Firm Reddish Brown Moist (FILL)	1	SS	56					
		2	SS	12	118	0	4	80	15
		3	SS	10	117				
		4	SS	7	116				
115.0 4.5	Silty **CLAY**, some sand, occasional shale fragments Hard Reddish Brown Wet	5	SS	34	115 114				
113.4 6.1	Highly weathered, very weak, reddish brown **SHALE**	6	SS	100/ .100	113 112				
111.7 7.8	END OF BOREHOLE AT 7.77m. AUGER REFUSAL AT 7.77m. BOREHOLE OPEN TO 7.01m AND WATER LEVEL AT 5.13m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.	7	SS	100/ .150					



GROUND WATER CONDITIONS
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)

ONTMT4S 5186.GPJ 20/01/06

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

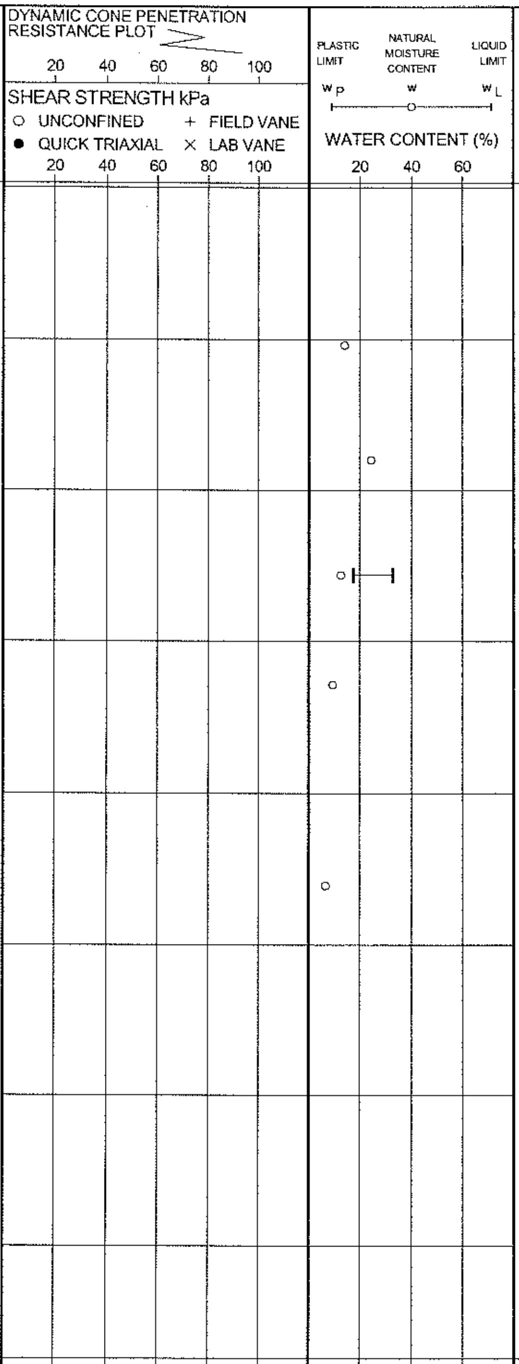
THURBER

RECORD OF BOREHOLE No 05-6

1 OF 1 METRIC

W.P. 19-1351-86
LOCATION N 4 806 108.2 E 284 177.7
ORIGINATED BY SLL
HWY QEW
BOREHOLE TYPE Solid Stem Augers and NQ Rock Core
COMPILED BY WM
DATUM Geodetic
DATE 16.11.05 - 16.11.05
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.0 0.0	**TOPSOIL** (150mm)				117	
0.2	Silty **CLAY**, some sand, trace gravel Stiff Brown Moist (FILL)					
		1	SS	9	116	
		2	SS	11		
115.1 1.9	Silty **CLAY**, some sand, trace gravel Stiff Brown Moist				115	
		3	SS	42		2 22 48 28
		4	SS	68	114	
113.7 3.3	Highly weathered, very weak to weak, reddish brown **SHALE**, with limestone layers and clay seams				113	
		5	SS	70/ .125		
	limestone interbeds at 5.08m to 5.18m, 5.64m to 5.77m, 5.81m to 5.87m, 6.05m to 6.10m, 6.73m to 6.84m, 6.94m Moderately weathered clay seams at 5.38m, 5.64m to 5.69m, 5.87m to 5.92m, 5.96m to 6.05m, 6.15m to 6.17m, 6.45m to 6.50m, 6.53m	1	RUN		112	RUN 1# TCR=90%, SCR=90%, RQD=22%, UCS=120MPa (limestone)
		2	RUN		111 110	RUN 2# TCR=98%, SCR=88%, RQD=61%, UCS=64MPa
109.2 7.8	END OF BOREHOLE AT 7.77m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.					



UNIT WEIGHT γ kN/m³

ONTMT4S 5186.GPJ 06/02/06

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-7

1 OF 1

METRIC

W.P. 19-1351-86	LOCATION N 4 806 116.7 E 284 186.1	ORIGINATED BY JHL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 22.11.05 - 22.11.05	CHECKED BY MRA

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
117.9	
117.9 0.0	**ASPHALT** (75mm)
0.1	**CRUSHED LIMESTONE**
117.3 0.6	Silty **CLAY**, some sand, some gravel, with shale fragments Stiff to Very Stiff Reddish Brown Moist (FILL)
114.9 3.0	Silty **CLAY**, some sand, travel gravel, with shale fragments Stiff Reddish Brown Moist
113.5 4.4	Highly weathered, very weak, reddish brown **SHALE**
111.3 6.6	END OF BOREHOLE AT 6.55m. AUGER REFUSAL AT 6.55m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.76m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
06.12.05	5.13	112.8

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	16				
2	SS	13				
3	SS	32				
4	SS	14	4	17	49	30
5	SS	50/ .125				
6	SS	86/ .225				

STRAT PLOT

GROUND WATER CONDITIONS

ELEVATION SCALE: 117, 116, 115, 114, 113, 112

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

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

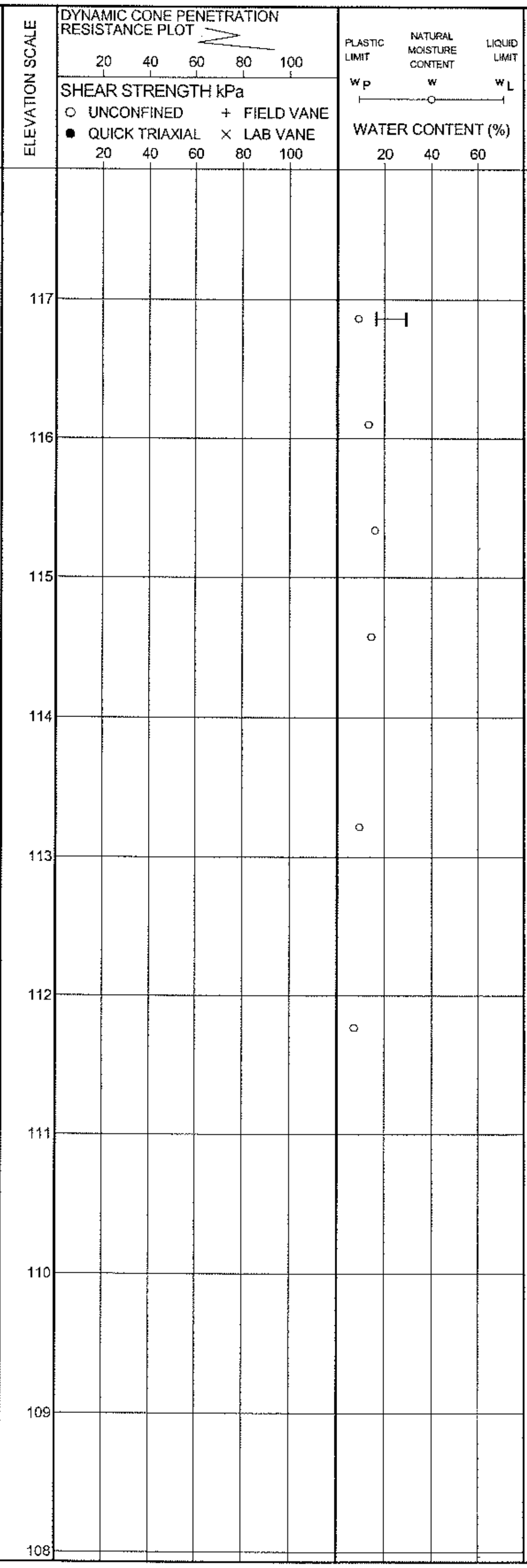
RECORD OF BOREHOLE No 05-8

1 OF 2

METRIC

W.P.	19-1351-86	LOCATION	N 4 806 117.7 E 284 187.2	ORIGINATED BY	JHL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers and NQ Rock Core	COMPILED BY	WM
DATUM	Geodetic	DATE	23.11.05 - 23.11.05	CHECKED BY	MRA

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.9 0.0	**CRUSHED LIMESTONE**							
117.3 0.6	Silty **CLAY**, some sand Stiff to Very Stiff Reddish Brown Moist (FILL)		1	SS	10			0 13 65 23
			2	SS	12			
			3	SS	16			
			4	SS	20			
114.4 3.5	Silty **CLAY**, some sand Very Stiff Reddish Brown Moist							
113.5 4.4	Highly weathered, very weak to weak, reddish brown **SHALE**, with clay seams and limestone interbeds		5	SS	50/ .125			
			6	SS	50/ .125			
	Moderately weathered limestone interbeds at 7.16m to 7.21m, 7.24m to 7.29m, 7.34m to 7.42m, 8.40m to 8.42m, 8.53m to 8.55m, 8.61m to 8.79m, 9.12m to 9.45m, 9.50m to 9.58m, 9.63m to 9.68m		1	RUN				RUN 1# TCR=100%, SCR=90%, RQD=53%, UCS=MPa
			2	RUN				RUN 2# TCR=100%, SCR=100%, RQD=77%, UCS=MPa



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

ONTMT4S 5186.GPJ 20/01/06

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-8

2 OF 2

METRIC

W.P. 19-1351-86

LOCATION N 4 806 117.7 E 284 187.2

ORIGINATED BY JHL

HWY QEW

BOREHOLE TYPE Solid Stem Augers and NQ Rock Core

COMPILED BY WM

DATUM Geodetic

DATE 23.11.05 - 23.11.05

CHECKED BY MRA

SOIL PROFILE			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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
107.8 10.1	END OF BOREHOLE AT 10.06m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.													
							107							

+ 3, × 3: Numbers refer to Sensitivity

$\overset{20}{\underset{10}{15\,\text{○}\,5}}$ (%) STRAIN AT FAILURE

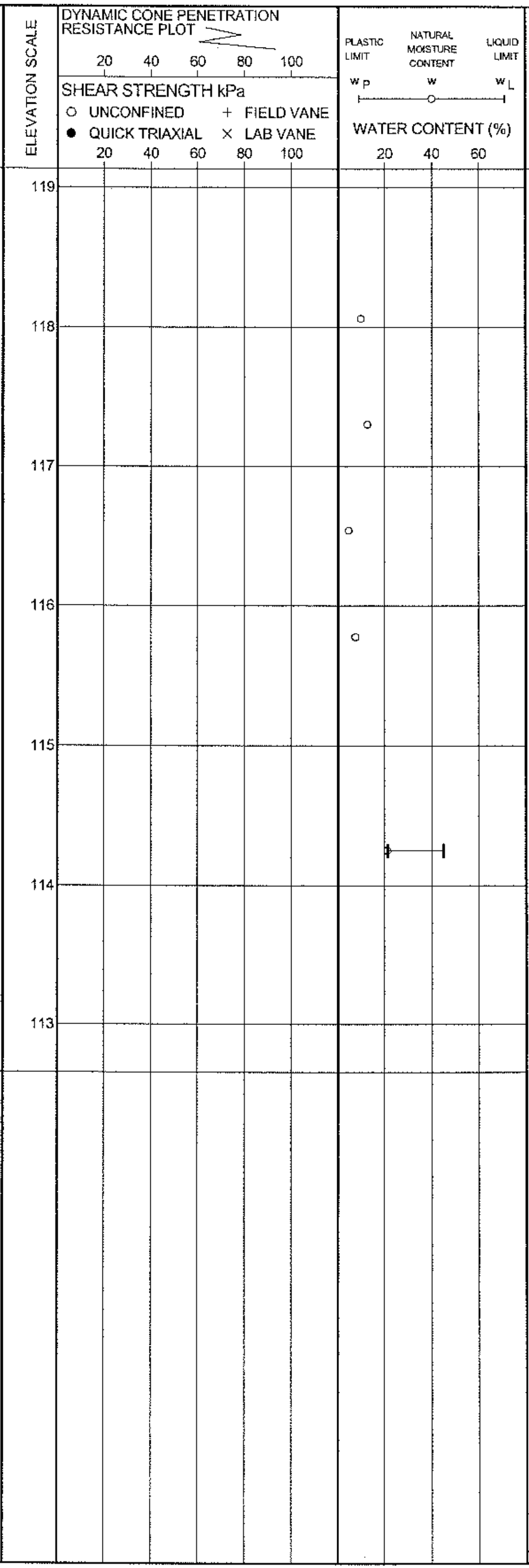
RECORD OF BOREHOLE No 05-9

1 OF 1

METRIC

W.P. 19-1351-86
LOCATION N 4 806 090.5 E 284 180.3
ORIGINATED BY SLL
HWY QEW
BOREHOLE TYPE Solid Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 25.11.05 - 25.11.05
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.1							
0.0	**SAND AND GRAVEL** Compact Brown Moist (FILL)	1	SS	23			
117.7 1.4	Silty **CLAY**, with shale and limestone fragments Very Stiff Reddish brown Moist to Wet (FILL)	2 3 4	SS SS SS	18 33 27			
115.1 4.0	Silty **CLAY**, some sand Very Stiff Brown Moist	5	SS	29			0 20 43 37
113.3 5.8	Highly weathered, very weak, reddish brown **SHALE**, with limestone interbeds	6	SS	102/ .225			
112.6 6.5	END OF BOREHOLE AT 6.48m. AUGER REFUSAL AT 6.48m. BOREHOLE OPEN TO 5.49m AND WATER LEVEL AT 4.88m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.						



+³, ×³: Numbers refer to Sensitivity

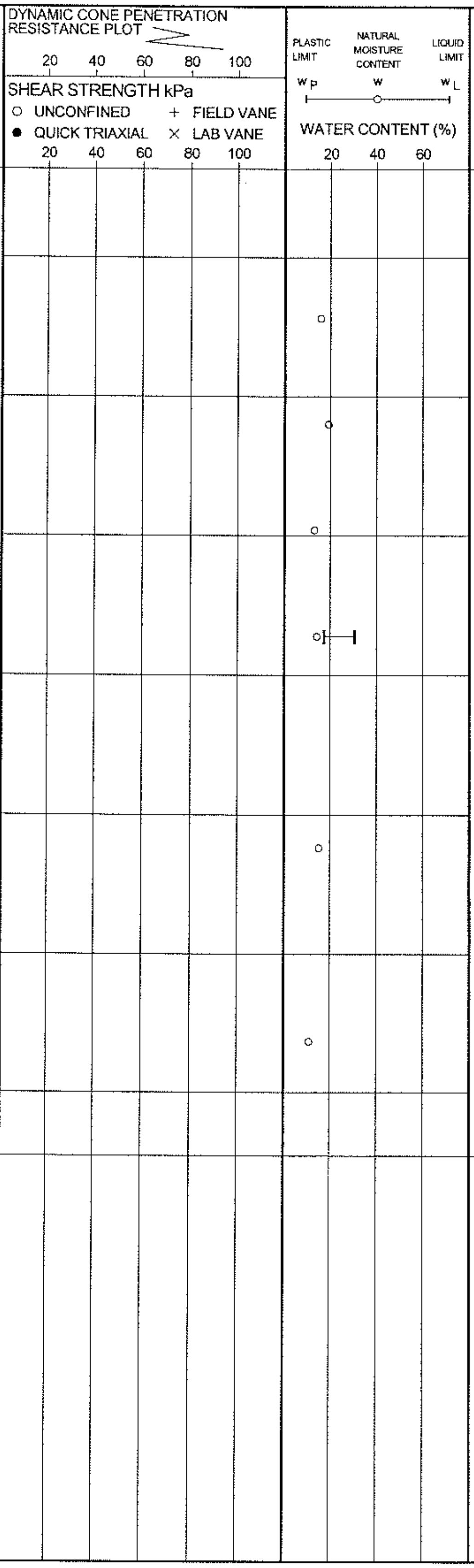
$15\text{-}\overset{20}{\underset{10}{\oplus}}\text{-}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 05-10

1 OF 1 **METRIC**

W.P.	19-1351-86	LOCATION	N 4 806 109.0 E 284 192.5	ORIGINATED BY	JHL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	22.11.05 - 22.11.05	CHECKED BY	MRA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
117.6 0.0	**CRUSHED LIMESTONE**								
117.0 0.6	Silty **CLAY**, some sand, trace gravel Firm Brown Moist (FILL)				117				
		1	SS	5					
		2	SS	7	116				
		3	SS	18	115				
114.9 2.7	Silty **CLAY**, trace sand, with shale and limestone fragments Very Stiff to Hard Reddish Brown Moist								
		4	SS	15	114	0	6	68	26
		5	SS	49	113				
112.1 5.5	Highly weathered, very weak, reddish brown **SHALE**, occasional limestone interbeds				112				
		6	SS	100/ .175	111				
110.5 7.1	END OF BOREHOLE AT 7.09m. AUGER REFUSAL AT 7.09m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.								



+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

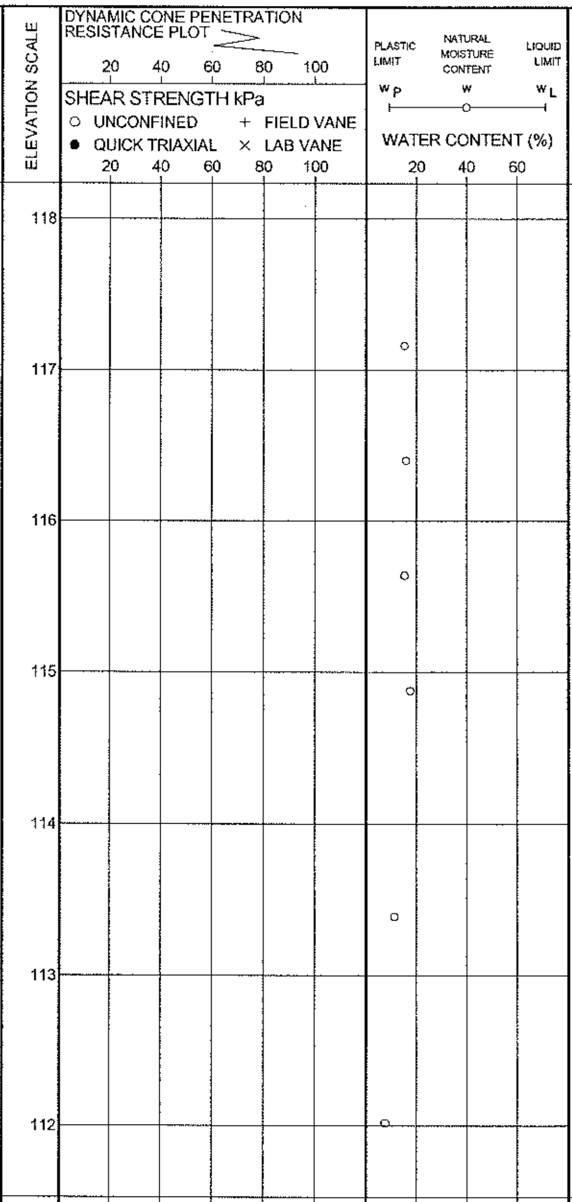
THURBER

RECORD OF BOREHOLE No 05-11

1 OF 1 **METRIC**

W.P. 19-1351-86
LOCATION N 4 806 135.5 E 284 169.0
ORIGINATED BY JHL
HWY QEW
BOREHOLE TYPE Solid Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 24.11.05 - 24.11.05
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
118.2							
0.0	**CRUSHED LIMESTONE**						
117.6 0.6	Silty **CLAY**, some sand, some gravel Stiff to Very Stiff Reddish Brown Moist (FILL)	1	SS	16			
		2	SS	12			
		3	SS	12			
		4	SS	10			
114.6 3.6	Silty **CLAY**, some sand, with shale fragments Firm to Very Stiff Reddish Brown Moist						
113.4 4.8	Highly weathered, very weak, reddish brown **SHALE**, with limestone interbeds	5	SS	86			
		6	SS	100/ .225			
111.5 6.7	END OF BOREHOLE AT 6.71m. AUGER REFUSAL AT 6.71m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.						



+ 3, × 3 : Numbers refer to Sensitivity

20
15-○-5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

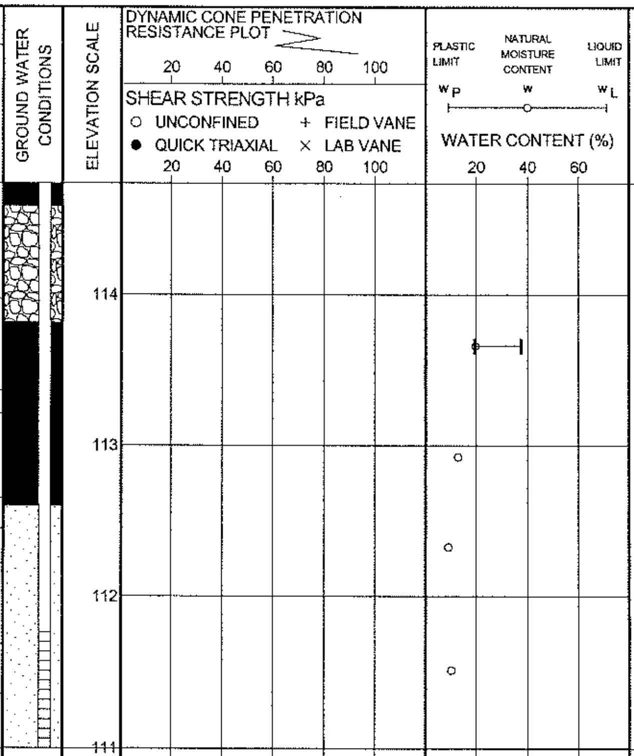
THURBER

RECORD OF BOREHOLE No 05-12

1 OF 1 **METRIC**

W.P. 19-1351-86	LOCATION N 4 806 154.4 E 284 179.1	ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 14.11.05 - 14.11.05	CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
114.7								
0.0	**TOPSOIL** (150mm)							
0.2	Silty **CLAY**, some sand Very Stiff to Hard Brown Moist							
		1	SS	23	0	18	50	31
		2	SS	53				
112.3 2.4	Highly weathered, very weak, reddish brown **SHALE**, with limestone interbeds	3	SS	50/ .075				
		4	SS	100/ .175				
111.0 3.7	END OF BOREHOLE AT 2.74m. AUGER REFUSAL AT 2.74m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.76m slotted screen.							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
06.12.05	1.46	113.2

+ 3 , × 3 : Numbers refer to Sensitivity

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

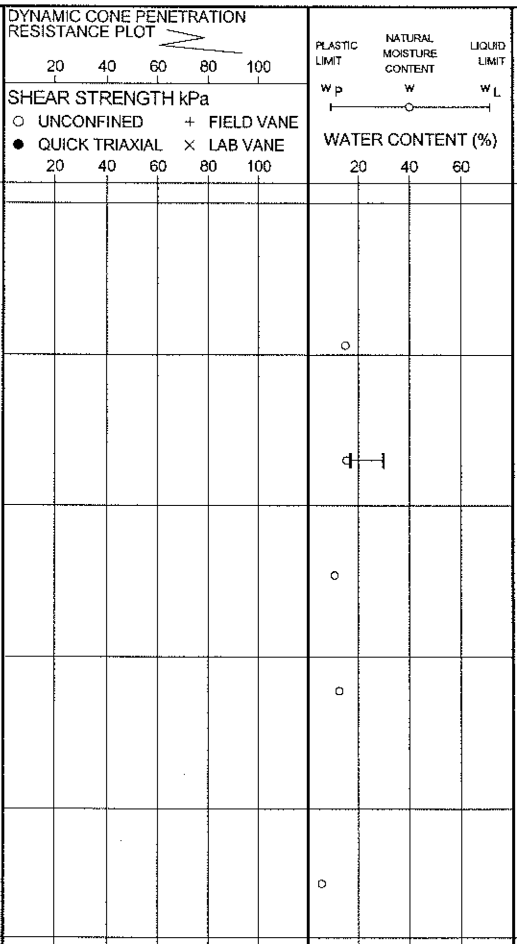
ONTMT4S 5186.GPJ 20/01/06

RECORD OF BOREHOLE No 05-13

1 OF 1 **METRIC**

W.P. 19-1351-86	LOCATION N 4 806 100.5 E 284 155.1	ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 25.11.05 - 25.11.05	CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
116.1 0.0	**TOPSOIL** (200mm)				116				
0.2	Silty **CLAY**, some sand, trace gravel Very Stiff to Hard Brown Moist	1	SS	22	115				
		2	SS	40		3	21	47	29
		3	SS	57	114				
		4	SS	50	113				
112.1 4.0	Highly weathered, very weak, reddish brown **SHALE**	5	SS	100/ .100	112				
111.1 5.0	END OF BOREHOLE AT 4.98m. AUGER REFUSAL AT 4.98m. BOREHOLE OPEN TO 4.57m AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.								



UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

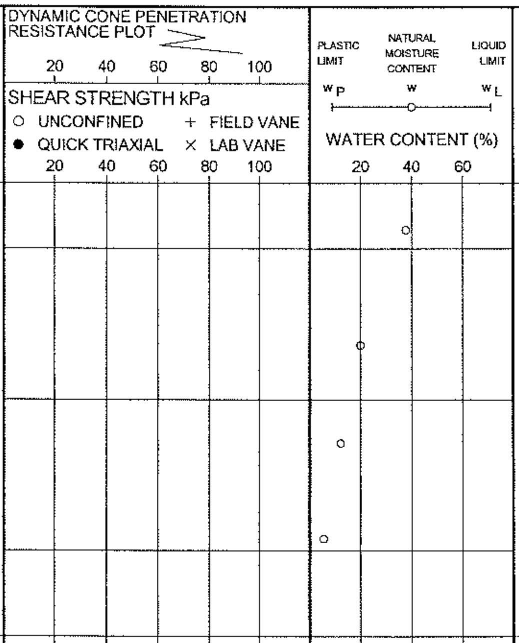
RECORD OF BOREHOLE No 05-14

1 OF 1

METRIC

W.P. 19-1351-86
LOCATION N 4 806 145.1 E 284 199.8
ORIGINATED BY SLL
HWY QEW
BOREHOLE TYPE Solid Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 11.11.05 - 11.11.05
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
113.4									
0.0	**TOPSOIL** (100mm)								
0.1	Silty **CLAY**, trace roots and rootlets, with shale fragments Soft to Hard Brown Moist		1	SS	3		113		
			2	SS	44				
112.0 1.4	Highly weathered, very weak, reddish brown **SHALE**, occasional limestone interbeds		3	SS	100/ 225		112		
			4	SS	60/ 125		111		
110.4 3.0	END OF BOREHOLE AT 3.00m. AUGER REFUSAL AT 3.00m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE.								



+³, ×³: Numbers refer to Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

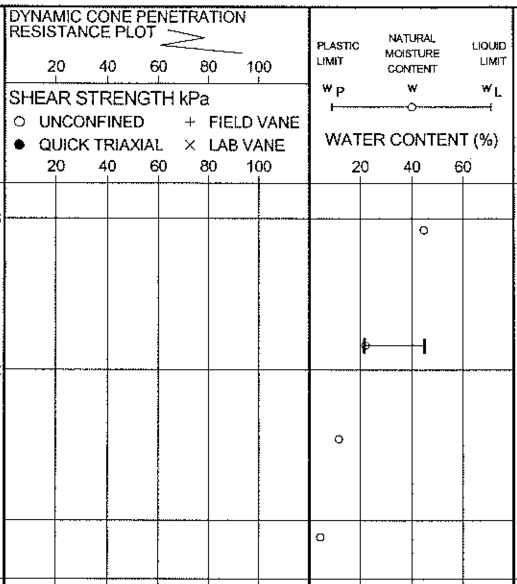
RECORD OF BOREHOLE No 05-15

1 OF 1

METRIC

W.P. 19-1351-86	LOCATION N 4 806 152.9 E 284 220.1	ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 11.11.05 - 11.11.05	CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
113.2												
0.0	**TOPSOIL** (300mm)											
112.9 0.3	Silty **CLAY**, trace sand, with shale fragments Stiff Brown Moist		1	SS	2		113					
			2	SS	9		112		0	2	58	39
111.6 1.6	Highly weathered, very weak, reddish brown **SHALE**		3	SS	89/ .175		111					
			4	SS	50/ .100							
110.6 2.6	END OF BOREHOLE AT 2.62m. AUGER REFUSAL AT 2.62m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE.											



+ ³, × ³ : Numbers refer to Sensitivity

20
15 ◆ 5
10
(%) STRAIN AT FAILURE

ONTMT4S 5186.GPJ 20/01/06

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-16

1 OF 1

METRIC

W.P. 19-1351-86
LOCATION N 4 806 192.3 E 284 259.4
ORIGINATED BY SLL

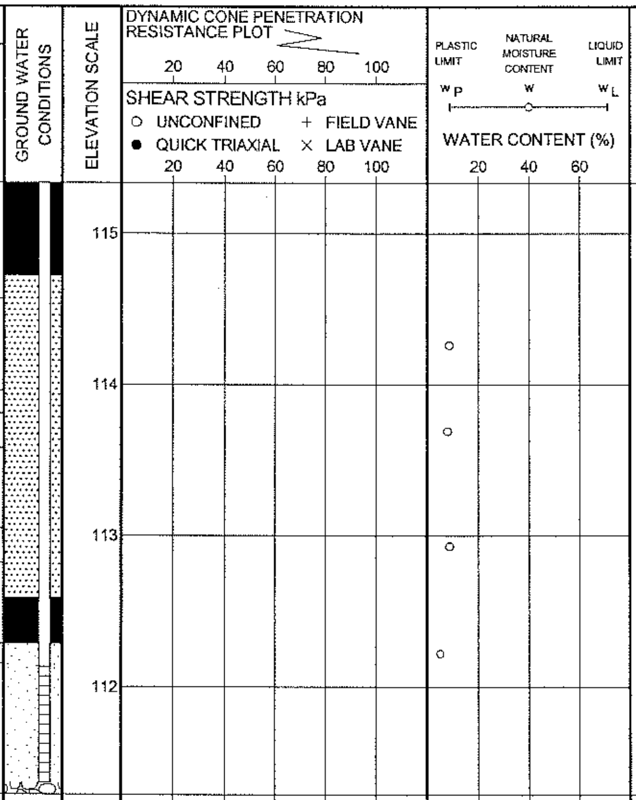
HWY QEW
BOREHOLE TYPE Solid Stem Augers
COMPILED BY WM

DATUM Geodetic
DATE 11.11.05 - 11.11.05
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
115.3				
0.0	**TOPSOIL** (150mm)			
0.2	Silty **CLAY**, trace sand Hard Reddish Brown Moist	1	SS	40
		2	SS	50/ .075
113.6 1.7	Highly weathered, very weak, reddish brown **SHALE**, occasional limestone interbeds	3	SS	50/ .075
		4	SS	50/ .125
111.3 4.0	END OF BOREHOLE AT 4.04m. AUGER REFUSAL AT 4.04m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.76m slotted screen.			

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
06.12.05	2.40	112.9



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 5186.GPJ 20/01/06

RECORD OF BOREHOLE No 05-17

1 OF 1

METRIC

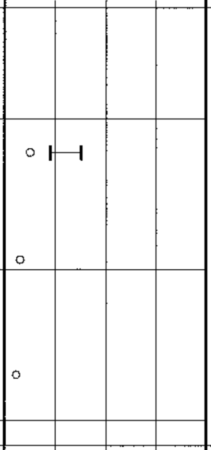
W.P.	19-1351-86	LOCATION	N 4 806 227.9 E 284 295.3	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	11.11.05 - 11.11.05	CHECKED BY	MRA

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

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.7									
0.0	**TOPSOIL** (100mm)								
0.1	Silty **CLAY**, trace sand, with shale fragments Hard Reddish Brown Moist						114		
			1	SS	79/ .225				0 2 74 24
113.2 1.5	Highly weathered, very weak, reddish brown **SHALE**, with limestone interbeds		2	SS	50/ .125		113		
			3	SS	50/ .125		112		
111.8 2.9	END OF BOREHOLE AT 2.90m. AUGER REFUSAL AT 2.90m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE.								



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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 05-18

1 OF 1

METRIC

W.P. 19-1351-86	LOCATION N 4 806 263.8 E 284 331.4	ORIGINATED BY SLL
HWY QEW	BOREHOLE TYPE Solid Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 11.11.05 - 11.11.05	CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
115.5							
0.0	**TOPSOIL** (150mm)						
0.2	Silty **CLAY**, with shale fragments Hard Reddish Brown Moist						115
114.4 1.1	Highly weathered, very weak, reddish brown **SHALE**		1	SS	47		
			2	SS	50/ .150		114
			3	SS	50/ .100		113
			4	SS	50/ .150		112
111.0 4.5	END OF BOREHOLE AT 4.50m. AUGER REFUSAL AT 4.50m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

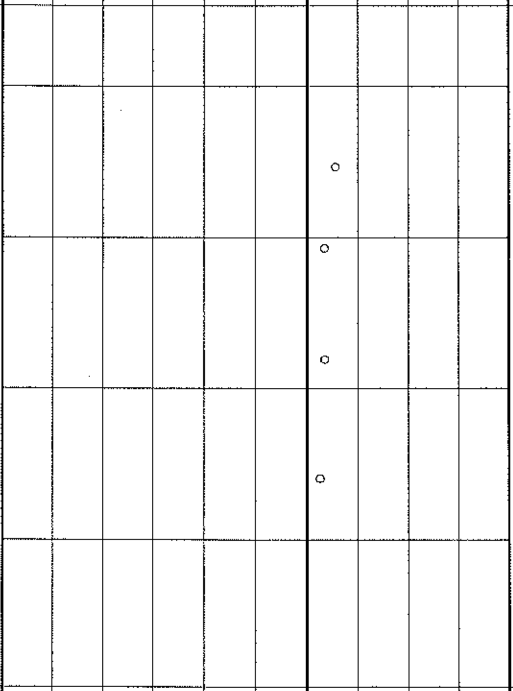
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



ONTMT4S 5186.GPJ 20/01/06

+ 3, × 3 : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

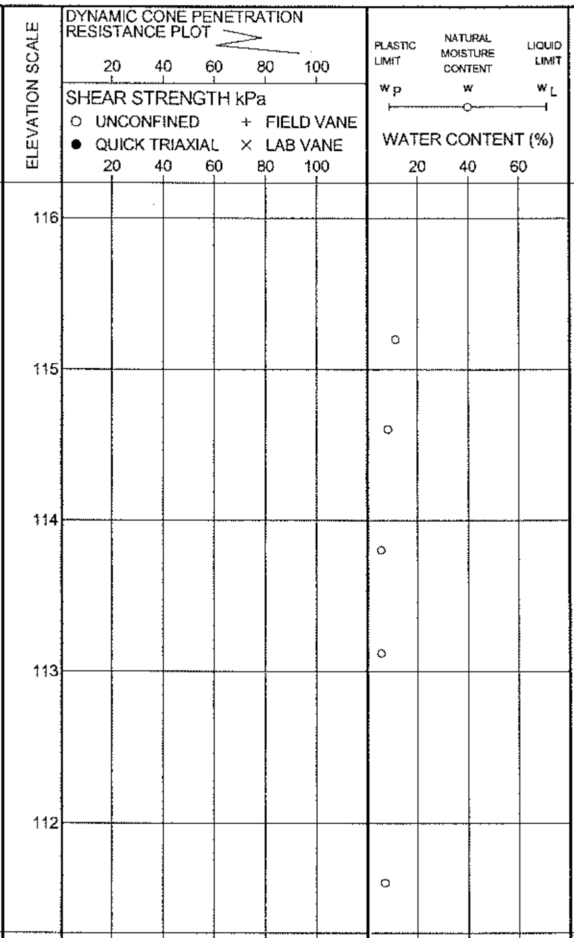
RECORD OF BOREHOLE No 05-19

1 OF 1

METRIC

W.P. 19-1351-86
HWY QEW
DATUM Geodetic
LOCATION N 4 806 299.1 E 284 367.0
BOREHOLE TYPE Solid Stem Augers
DATE 11.11.05 - 11.11.05
ORIGINATED BY SLL
COMPILED BY WM
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.2								
0.0	**SAND AND GRAVEL**	☒						
0.2	Silty **CLAY**, with shale fragments Reddish Brown Moist	▨						
115.5 0.7	Highly weathered, very weak, reddish brown **SHALE**, occasional limestone interbeds		1	SS	57			
			2	SS	50/ .050			
			3	SS	50/ .125			
			4	SS	50/ .125			
			5	SS	50/ .100			
111.2 5.0	END OF BOREHOLE AT 4.95m. AUGER REFUSAL AT 4.95m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.							



ONTMT4S 5186.GPJ 06/02/06

+ ³, × ³: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

Appendix B

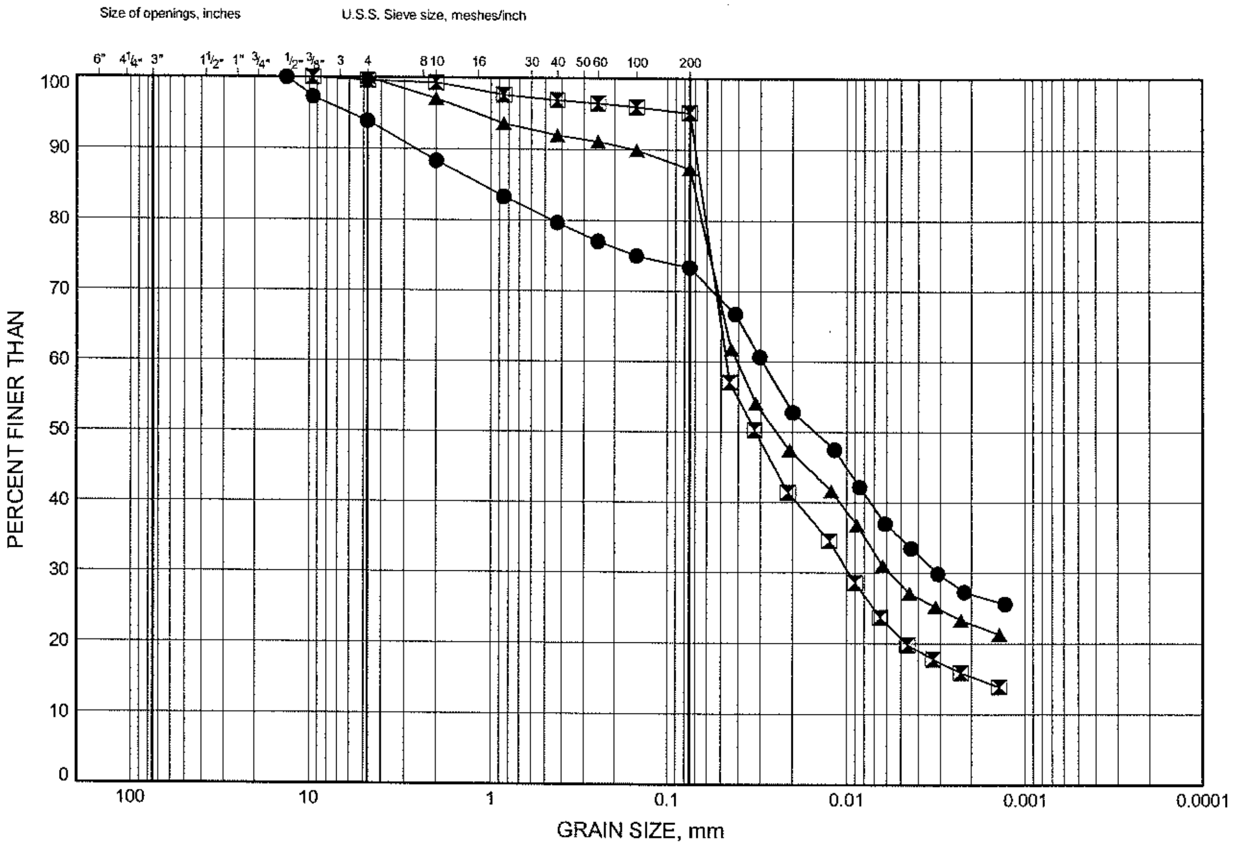
Laboratory Test Results

Burloak Ramp Grade Separation

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-2	1.83	116.37
☒	05-5	1.83	117.67
▲	05-8	1.07	116.83

THURBGSD 5186.GPJ 20/01/06

Date January 2006
Project 19-1351-86

THURBER

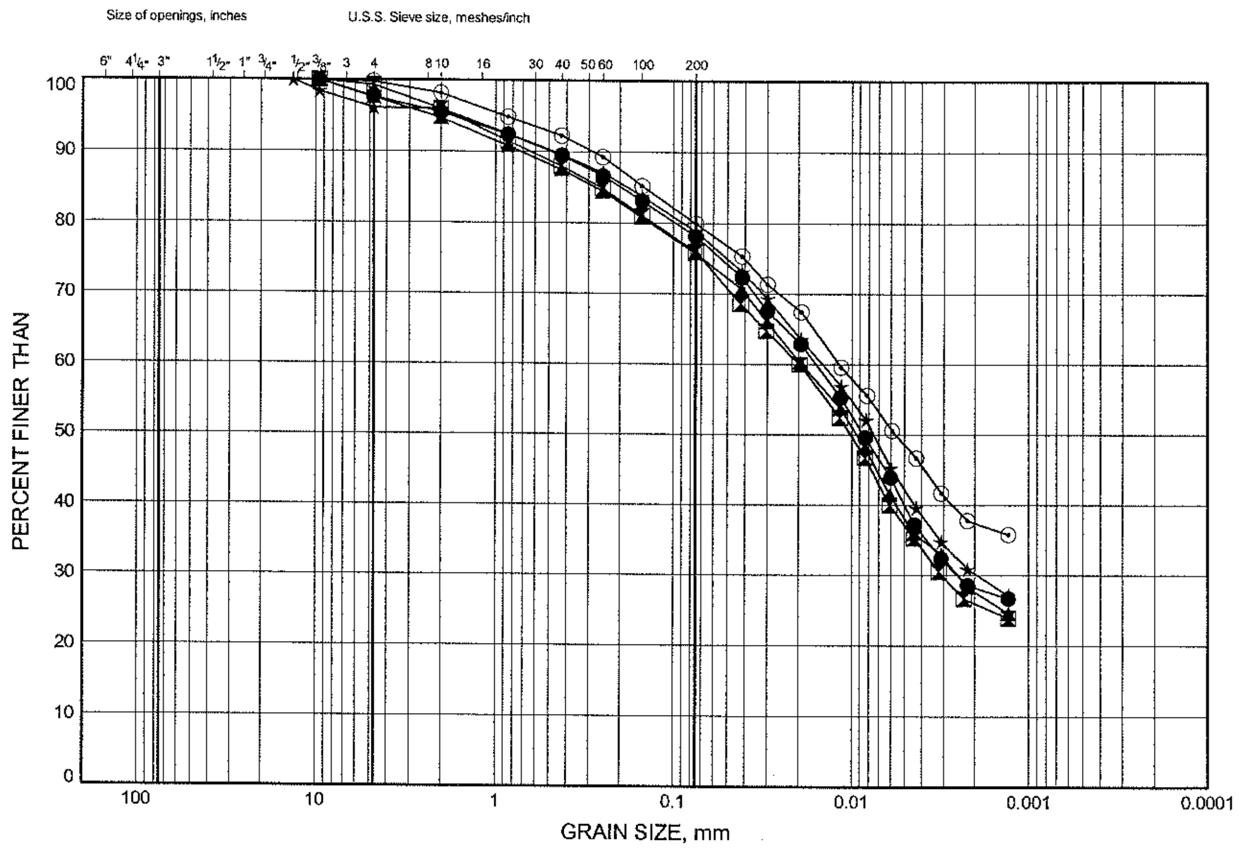
Prep'd JHL
Chkd. MRA

Burloak Ramp Grade Separation

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-1	2.59	114.61
⊠	05-3	4.88	112.82
▲	05-6	2.59	114.41
★	05-7	3.35	114.55
⊙	05-9	4.88	114.22

Date January 2006

Project 19-1351-86

THURBER

Prep'd JHL

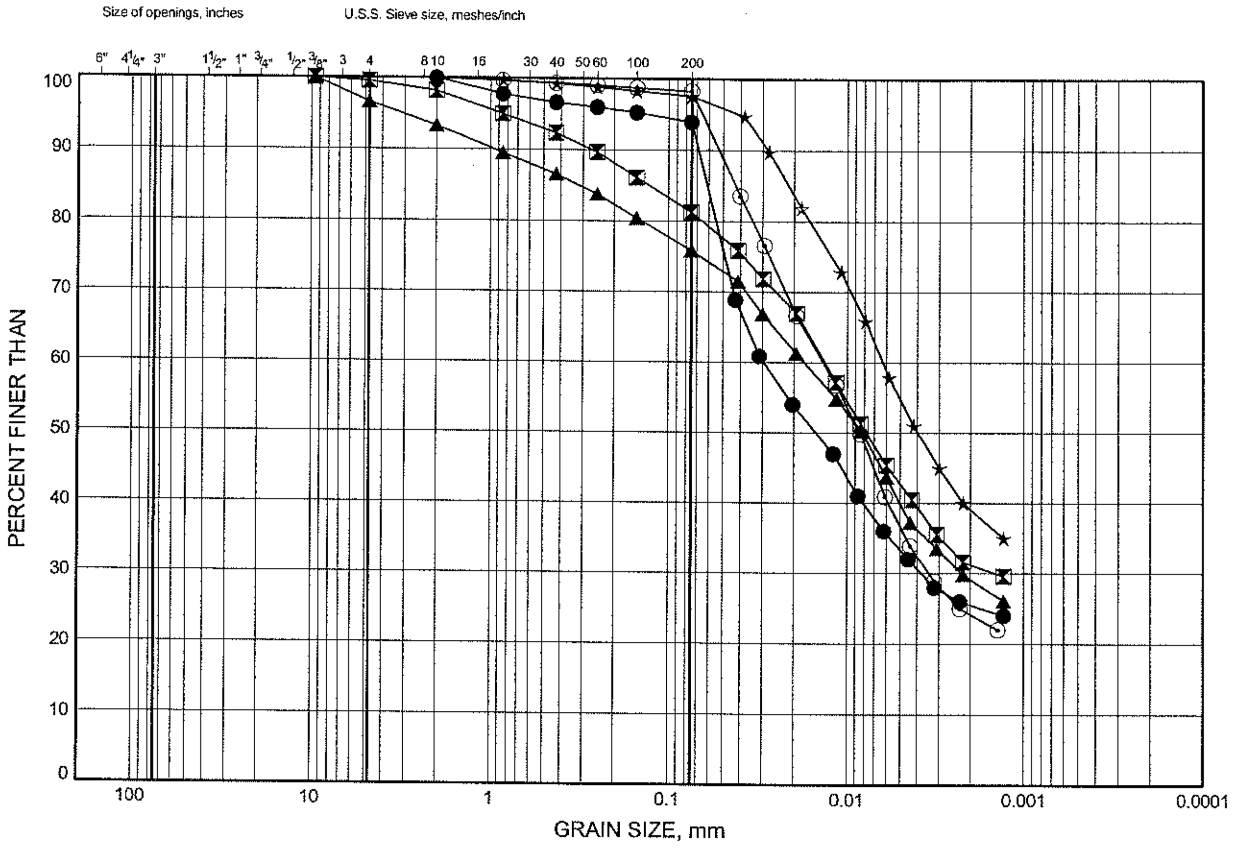
Chkd. MRA

Burloak Ramp Grade Separation

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-10	3.35	114.25
☒	05-12	1.07	113.63
▲	05-13	1.83	114.27
★	05-15	1.07	112.13
⊙	05-17	0.95	113.75

Date January 2006

Project 19-1351-86

THURBER

Prep'd JHL

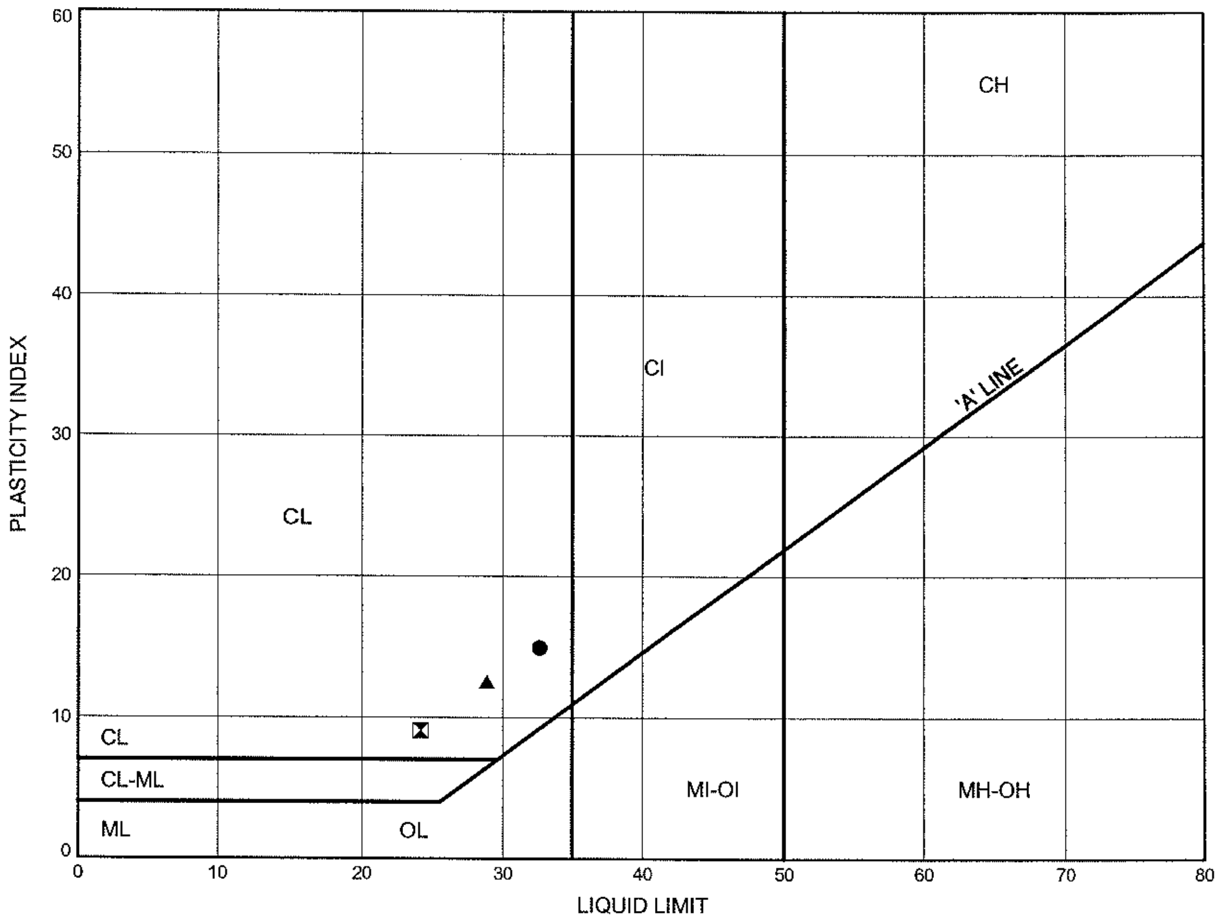
Chkd. MRA

Burloak Ramp Grade Separation

ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SILTY CLAY FILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-2	1.83	116.37
☒	05-5	1.83	117.67
▲	05-8	1.07	116.83

Date January 2006

Project 19-1351-86

THURBER

Prep'd JHL

Chkd. MRA

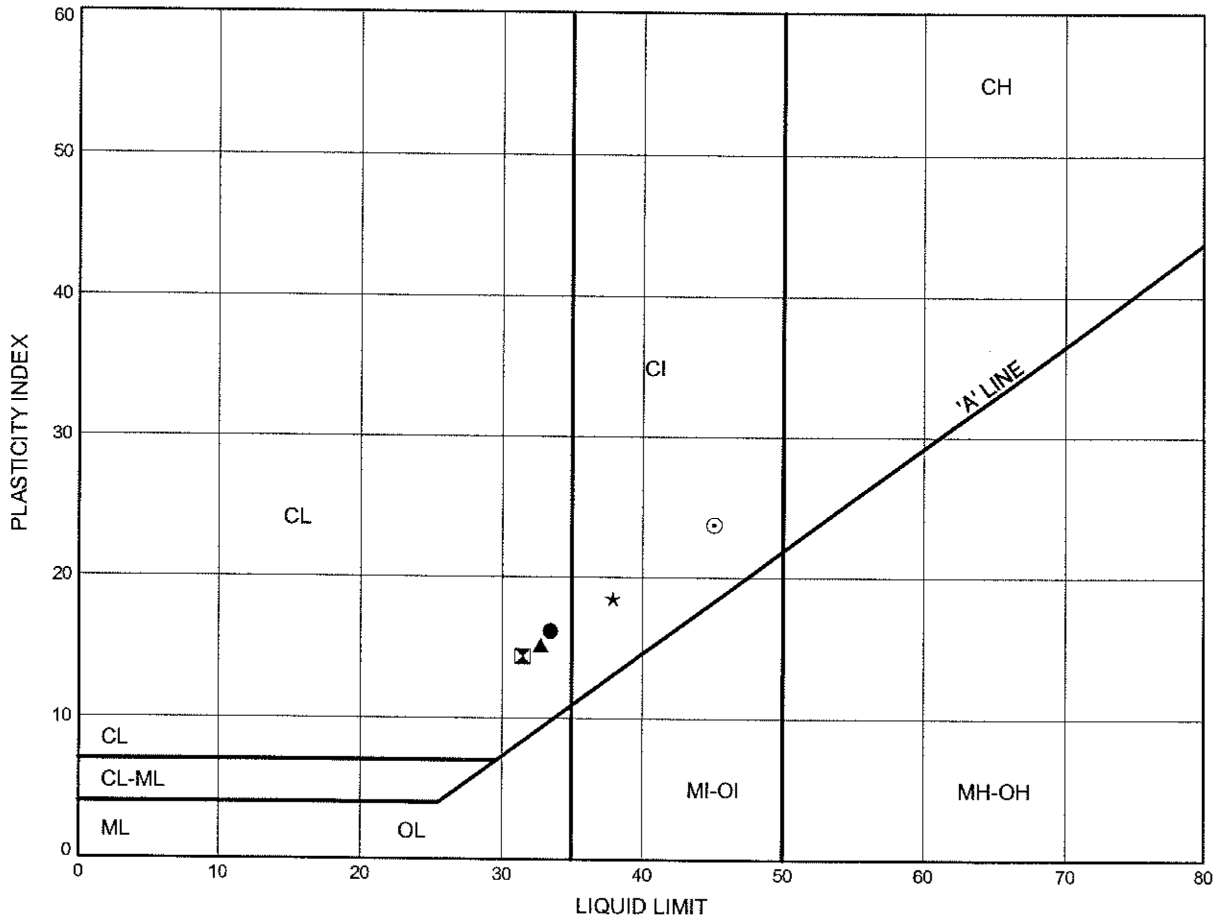
THURBALT 5186.GPJ 20/01/06

Burloak Ramp Grade Separation

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-1	2.59	114.61
⧗	05-3	4.88	112.82
▲	05-6	2.59	114.41
★	05-7	3.35	114.55
⊙	05-9	4.88	114.22

Date January 2006

Project 19-1351-86

THURBER

Prep'd JHL

Chkd. MRA

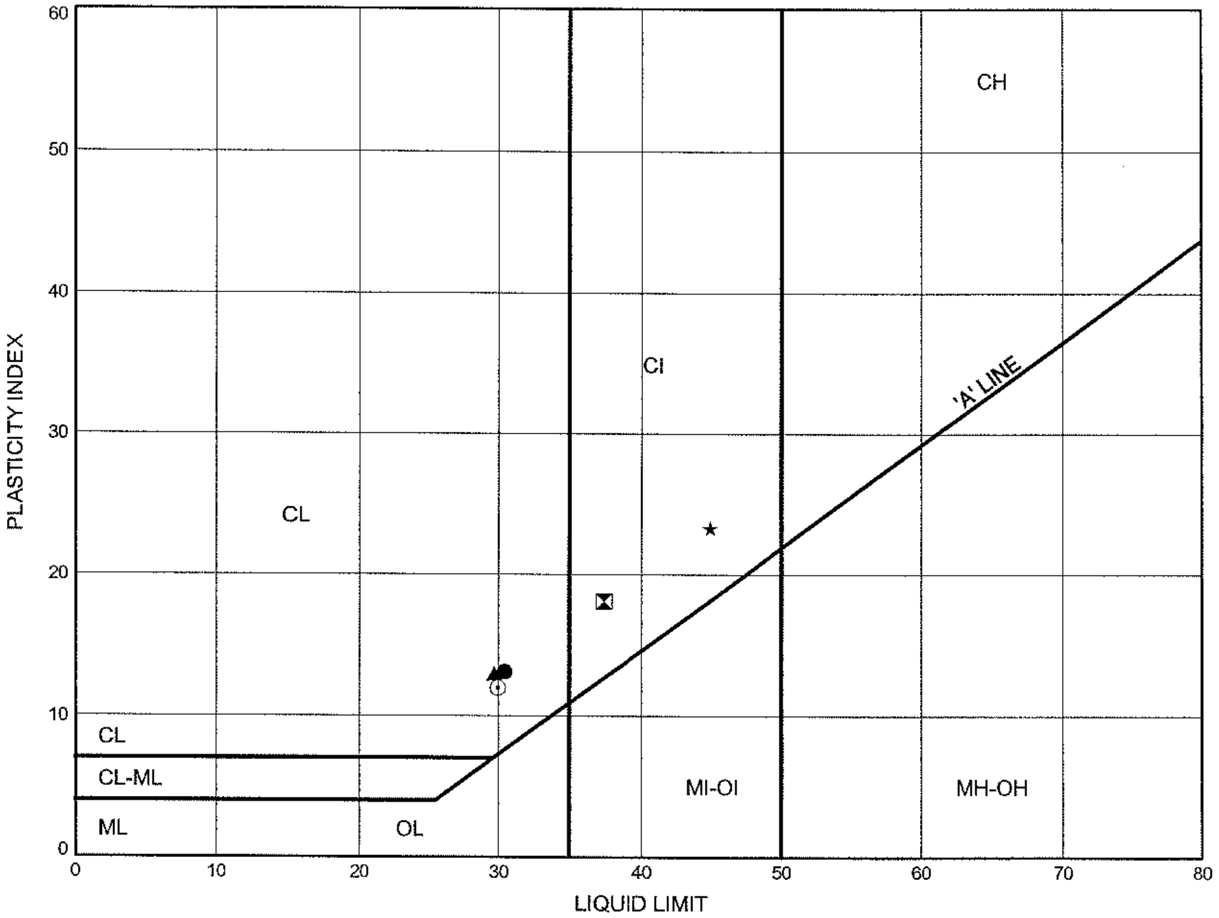
THURBALT 5186.GPJ 20/01/06

Burloak Ramp Grade Separation

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-10	3.35	114.25
☒	05-12	1.07	113.63
▲	05-13	1.83	114.27
★	05-15	1.07	112.13
⊙	05-17	0.95	113.75

Date January 2006

Project 19-1351-86

THURBER

Prep'd JHL

Chkd. MRA

Appendix C

Table: Foundation Comparison

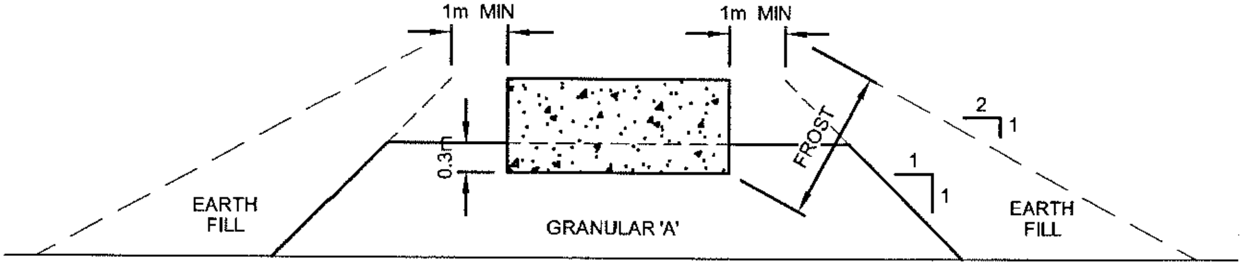
Figure 1: Abutment on Compacted Fill Showing Granular Core

NSSP: Retained Soil System

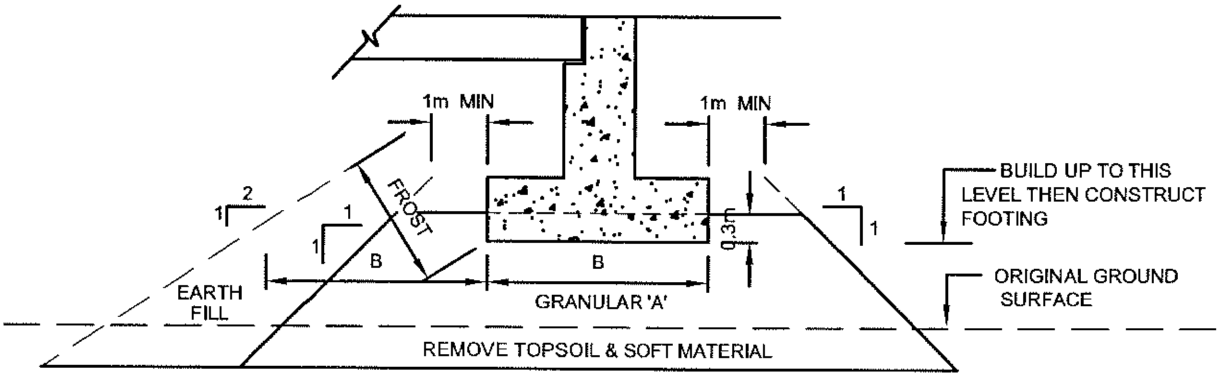
COMPARISON OF FOUNDATION ALTERNATIVES

Driven Piles	Footings on Native Soil or Shale	Footings on Engineered Fill	Caissons
Advantages: i. Piles will develop high geotechnical resistance if driven to refusal in the shale. ii. Allows choice of conventional, integral or semi-integral abutment design. *Disadvantages:* i. Higher unit costs than footings. ii. Pre-augering will be required to install the piles to adequate depth in shale. **NOT RECOMMENDED**	*Advantages:* i. Ease of construction. ii. Lower cost than deep foundations. iii. Relatively high geotechnical resistance is available in the native clay and shale at shallow depth. *Disadvantages:* i. Subexcavation may be required to penetrate fill and upper variable material. **RECOMMENDED**	*Advantages:* i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows use of perched abutments. iii. Lower cost than deep foundations. *Disadvantages:* i. Cost of constructing engineered fill. ii. Subexcavation of existing fill and upper variable material required. **FEASIBLE**	*Advantages:* i. High geotechnical resistance is available for caissons socketed in shale. ii. Subexcavation of fill and variable material not required. *Disadvantages:* i. Higher cost than spread footings. ii. Potential difficulties in cleaning and inspecting bases. iii. Potential difficulties penetrating hard limestone layers in shale. **FEASIBLE**

March , 2004



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER SKP
DRAWN SS
DATE JAN. 2006
APPROVED PKC
SCALE NTS

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

THURBER

DWG. NO. FIGURE 1

TED35146.DWG

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, LOW PERFORMANCE - Item No.
BACKFILL FOR RETAINED SOIL SYSTEM, LOW PERFORMANCE - Item No.

Non Standard Special Provision September, 2005

1.0 SCOPE

This special provision covers the requirements for the design and construction of Retained Soil Systems (RSS) walls and steep slopes.

Additional requirements for RSS precast concrete facing elements shall be as specified in the Contract documents.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 102	Weighing of Materials
OPSS 180	Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction

OPSS 501	Compacting

Canadian Standards Association Standards:

CAN/CSA-S6-00	Canadian Highway Bridge Design Code (CHBDC)

Ministry of Transportation Publications:

MTO Designated Sources of Materials (DSM)
Qualification Criteria for RSS

3.0 DEFINITIONS

For the purposes of this special provision the following definitions apply:

Alignment Elements: means components specified by the manufacturer that are constructed on the foundation for RSS to facilitate placing of the facing elements to the correct lines and grades, such as concrete levelling pads and soldier piles.

Approved Product Drawings: means the documentation for an RSS that has been submitted by the manufacturer and accepted by the Ministry for listing in the DSM, according to the Qualification Criteria for RSS.

Backfill for RSS: means the material specified by the manufacturer as part of the engineered materials comprising the backfill for the RSS.

Constructed Height: means the vertical distance between the foundation for RSS and the top of the currently placed and compacted backfill for RSS, measured at the point of the design height.

Corrective Work: means work carried out by the Contractor to repair deficiencies identified by the Owner during the RSS warranty period.

Design Check Engineer: means an Engineer retained by the Contractor who checks the working drawings; the Design Check Engineer shall have the appropriate experience and expertise to provide design services for the manufacturer's RSS.

Design Engineer: means an Engineer retained by the Contractor who produces the working drawings; the Design Engineer shall have the appropriate experience and expertise to provide design services for the manufacturer's RSS.

Design Height: means the maximum difference in elevation between the foundation for RSS and the corresponding top of backfill for RSS, over the full length or perimeter of the RSS.

External Stability: means stability against deep-seated failure of the foundation for RSS, including adequate bearing capacity at specified settlements of the foundation.

Facing Elements: means components specified by the manufacturer that delineate the front face of the RSS and to which reinforcing elements may be attached, such as precast concrete panels, split-face concrete blocks, and geo-synthetic panels.

Foundation for RSS: means the base on which the RSS is constructed, such as excavation to a specified elevation and construction of a granular 'A' pad.

Internal Stability: means stability against failure of the engineered materials comprising the RSS, including adequate resistance against excessive elongation, breakage and pullout of the reinforcing elements.

Manufacturer: means the firm who supplies the design and proprietary components, and who specifies the backfill and other materials, for the RSS selected by the Contractor.

Manufacturer's Representative: means an individual with continuous full-time employment with the manufacturer for a period of at least three (3) years, and who is knowledgeable in the design and construction of the RSS selected by the Contractor.

Obstruction: means any part of the work and any existing condition within the Contract limits that affects the design, construction and performance of the RSS, such as structures, catch basins and manholes, drainage pipes and sewers, and utilities.

Performance Tolerance – Local: means the joint gap between any two constructed facing elements, measured at any point along the joint between the facing elements and perpendicular to the line of the joint.
Performance Tolerance – Global: means the vector distance between any point on the constructed RSS and the corresponding point on the theoretical RSS surface as defined in the Contract documents.

Placing Tolerances: means tolerances specified by the manufacturer on the placing of the RSS components and backfill for RSS to ensure compliance of the constructed RSS with the performance tolerances.

Quality Verification Engineer (QVE): means an Engineer, other than the Design Engineer and Design Check Engineer, retained by the Contractor to ensure the work conforms with the Contract documents and to issue Certificates of Conformance.

Reinforcing Elements: means components specified by the manufacturer that are placed within the backfill for RSS and connected to the facing elements to mechanically stabilize the backfill for RSS, such as metal tie strips, metal grids and geo-synthetic grids,

Retained Soil System (RSS): means a proprietary system listed in the DSM used to retain horizontal loads for applications such as true and false abutment structures, retaining walls and steep slopes; or, to retain vertical loads for applications such as embankments over soft ground.

RSS Superintendent: means the Contractor's authorized representative in responsible charge of the construction of the RSS.

Structure: means any bridge, culvert, tunnel, retaining wall, overhead sign, high mast light pole, wharf, dock, or any part thereof.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submissions

4.1.1 Working Drawings

The Contractor shall submit working drawings for all RSS. A separate submission shall be made for each RSS in the Contract. All submissions shall bear the seal and signature of the Design Engineer and the Design Check Engineer.

The RSS Superintendent shall have a copy of the working drawings in his possession at all times when on site.

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

4.1.2 Working Drawing Requirements

Working drawings shall include at least the following:

- Statement from the manufacturer confirming the experience and expertise of the Design Engineer and Design Check Engineer to provide design services for the manufacturer's RSS;
- All design, fabrication and construction drawings and specifications for the RSS;
- Location and value of the design height of the RSS;
- Defined lines and grades, type, and quantity in m^3 of the backfill for RSS;
- Details at obstructions, and connections to other structures, where shown in the Contract drawings;
- Statement of bearing resistance required by the RSS foundation according to the CHBDC;
- Statement of satisfactory internal and external stability;
- Placing tolerances for the RSS.

4.1.3 RSS Superintendent

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the RSS Superintendent for each RSS in the Contract.

During construction of an RSS, the Contractor shall not change the RSS Superintendent for that RSS without written permission from the Contract Administrator. The Contractor shall submit in writing to the Contract Administrator the proposed change for RSS Superintendent at least one week prior to the actual change in RSS Superintendent.

4.1.4 Manufacturer's Representative

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the manufacturer's representative for each RSS in the Contract.

For each occasion the Contractor arranges for the manufacturer's representative to be on site, the Contractor shall submit 48 hours advance notice in writing to the Contract Administrator giving the dates and locations the manufacturer's representative will be on site.

4.1.5 Certificates of Conformance

For each RSS in the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the QVE upon completion of each of the following milestones, and prior to commencement of subsequent operations on that RSS:

a) Layout and marking of all lines and grades needed to construct the RSS; and construction of the alignment elements, where applicable;
b) Delivery and storage on site of facing elements and reinforcing elements, where applicable;
c) Installation of the facing elements; placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable;
d) Completion of the RSS.

The Certificate of Conformance shall state that the work has been carried out in general conformance with the Contract documents and stamped working drawings.

For RSS where the design height is greater than 3.0 m, the Contractor shall submit a series of

Certificates of Conformance for milestone c) corresponding to the constructed height of the RSS at 3.0 m, 6.0 m, 9.0 m, 12.0 m and 15.0 m, as applicable, up to and including the design height.

The milestone Certificate of Conformance submissions in no way supersede the inspection and testing intervals required for the construction of the RSS, as specified in the working drawings.

4.1.6 RSS Warranty

The Contractor shall submit a warranty to the Owner to address all deficiencies identified by the Owner related to the performance of the RSS for a period of 36 months from the date of certification of completion of the Contract.

4.2 Design

4.2.1 General

The Contractor shall be responsible for the design of the RSS and for ensuring the RSS as designed is compatible with the work, except that the foundation for RSS shall be as specified in the Contract documents.

The geometric requirements of the RSS, such as lines and grades of the facing elements, typical cross-sections, and other constraints influencing the design shall be as specified in the Contract drawings.

4.2.2 RSS Selection

The Contractor shall select an RSS from the DSM that meets the Application, Performance and Appearance requirements for that RSS, as specified in the Contract drawings.

The Contractor shall select an RSS from the DSM designated as either 'A' (Accepted) or 'DE' (Demonstration). RSS designated as 'DE' status require inspection, instrumentation and monitoring of the constructed RSS, and reporting of the findings to the Ministry by the manufacturer, according to the Qualification Criteria for RSS.

Where there is more than one RSS in the Contract, the Contractor shall select the RSS from the same DSM listing, including type and colour of facing elements, according to the following groupings:

a) All RSS covered under the same tender item number(s) for payment;

b) All RSS with the same Performance and Appearance requirements that abut the same structure, existing and/or part of the work.

4.2.3 Performance Tolerances

Performance tolerances for the RSS shall be according to Table 1.

TABLE 1 – PERFORMANCE TOLERANCES FOR RSS

Performance Requirement	Performance Tolerance (mm)	
	Local	Global
Abutments	Joint Gap[1] ± 5	≤ 20
High	Joint Gap[1] ± 10	≤ 30
Medium	N/A	≤ 50
Low	N/A	≤ 100

Note 1.: Joint Gap shall be as specified in the working drawings.

4.2.4 Obstructions

The Contractor shall be responsible for developing design details of the RSS at obstructions, for all obstructions shown in the Contract drawings.

Where an obstruction is shown in the Contract drawings but not located to sufficient accuracy for the design of the RSS, the Contractor shall locate the obstruction in the field to sufficient accuracy as required to design the RSS.

4.2.5 Foundation Report

A Foundation Investigation Report that describes the subsurface conditions at the RSS is available, as specified in the Contract documents.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

5.0 MATERIALS

5.1 General

All materials for the selected RSS shall be according to the Approved Product Drawings for that RSS.

6.0 EQUIPMENT

6.1 Restriction on Skid-Steer Vehicles

Skid-steer vehicles will not be permitted on any area where the depth of backfill for RSS over installed reinforcing elements is less than 0.5 m.

7.0 CONSTRUCTION

7.1 General

The RSS shall be constructed according to the working drawings and this Special Provision.

Construction of the RSS shall not commence until the Contractor has submitted all applicable Certificates of Conformance for the foundation for RSS.

7.2 RSS Superintendent

The Contractor shall schedule his operations such that the construction of an RSS is at all times under the responsible charge of an RSS Superintendent who has been advised on site by the manufacturer's representative as to the required procedures for the construction of that RSS, for the specified operations and time periods.

7.3 Manufacturer's Representative

The manufacturer's representative shall be on site to advise the RSS Superintendent as to the procedures and placing tolerances required for the construction of the RSS.

For each RSS in the Contract, the Contractor shall arrange for the manufacturer's representative to be on site at commencement of each of the following operations, for a time period of three (3) working days per operation or until the operation is complete, whichever is less:

a) Layout of the RSS; and construction of the alignment elements, where applicable;
b) Installation of the facing elements;
c) Placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable.

Whenever there is a change in the RSS Superintendent during construction of an RSS, the Contractor shall arrange for the manufacturer's representative to return to the site for the same operations and time periods as at commencement.

7.4 Backfill for RSS

Backfill for RSS shall be placed within the lines and grades shown on the working drawings. All backfill for RSS shall be compacted according to OPSS 501.

The Contractor shall coordinate the placing and compacting of the backfill for RSS with his other backfilling operations.

Prior to placing backfill for RSS against an adjacent concrete structure that is part of the work, the Contractor shall ensure that the concrete in that structure has obtained a compressive strength at least 70% of the value specified in the Contract drawings.

7.5 Management of Excess Materials

Management of excess materials shall be according to OPSS 180.

8.0 QUALITY ASSURANCE

8.1 Acceptance Criteria at End of the RSS Warranty Period

The Owner will accept the RSS at the end of the RSS warranty period if none of the deficiencies listed in Table 2 are found during the warranty inspections. Where deficiencies are found, the RSS will not be accepted until the Contractor has carried out approved corrective work to repair the deficiencies.

TABLE 2 – RSS DEFICIENCIES

Number	Description of Deficiency
1.	Performance tolerance exceeds tolerances given in Table 1.
2.	Damaged facing elements and damaged alignment elements, where applicable.
3.	Dead and dying vegetative elements that are an integral part of the RSS.

8.2 Warranty Inspections

Throughout the warranty period and no later than twelve weeks prior to the expiry of the RSS warranty period, the Owner will carry out warranty inspections of the RSS for deficiencies as per Table 2. The Owner will notify the Contractor as to the date and time of the inspection(s) and the Contractor may, at his discretion, be present during the inspection(s).

Within two weeks following a warranty inspection the Owner will notify the Contractor in writing of all deficiencies that require corrective work.

8.3 Proposal for Corrective Work

At least two weeks prior to commencement of any corrective work at an RSS, the Contractor shall submit to the Manager of Contracts, for approval, three copies of his proposal for corrective work for that RSS.

The proposal for corrective work shall establish the cause, and fully detail the repair procedures required to correct, each deficiency identified by the Owner.

The proposal for corrective work shall bear the seal and signature of an Engineer (who may be different than the Design Engineer and Design Check Engineer), and be signed by the manufacturer's representative.

8.4 Corrective Work

At least one week prior to commencement of any corrective work at an RSS, the Contractor shall submit written notice of commencement to the Manager of Contracts.

The Contractor shall repair all deficiencies according to the approved proposal for corrective work. All corrective work shall be done within the RSS warranty period, unless prevented by seasonal shutdown, in which case the corrective work shall be done during the first eight weeks of the following construction season.

The Contractor shall provide access to the corrective work for inspection by the Owner when requested.

9.0 MEASUREMENT FOR PAYMENT

9.1 Backfill for Retained Soil System, High Performance
Backfill for Retained Soil System, Medium Performance
Backfill for Retained Soil System, Low Performance

The quantity of backfill for RSS shall include only those quantities measured within the theoretical lines and grades shown in the stamped working drawings.

9.1.1 Actual Measurement

Measurement will be of the mass in tonnes and the method of determining the mass shall be according to OPSS 102.

10.0 BASIS OF PAYMENT

10.1 Retained Soil System, True Abutment - Item
Retained Soil System, False Abutment - Item
Retained Soil System, Wall/Slope, High Performance – Item
Backfill for Retained Soil System, High Performance – Item
Retained Soil System, Wall/Slope, Medium Performance – Item
Backfill for Retained Soil System, Medium Performance – Item
Retained Soil System, Wall/Slope, Low Performance – Item
Backfill for Retained Soil System, Low Performance – Item

Payment at the contract price for the above tender items shall be full compensation for all labour, equipment and material to do the work, including all costs associated with the manufacturer's representative on site.

Payment for construction of the foundation for RSS will be made under the appropriate tender items in the Contract.

No payment will be made for corrective work, including investigation of deficiencies, design of repairs, site access, traffic staging and removal of existing work, except where the corrective work is required as a result other than an act or fault of the Contractor.

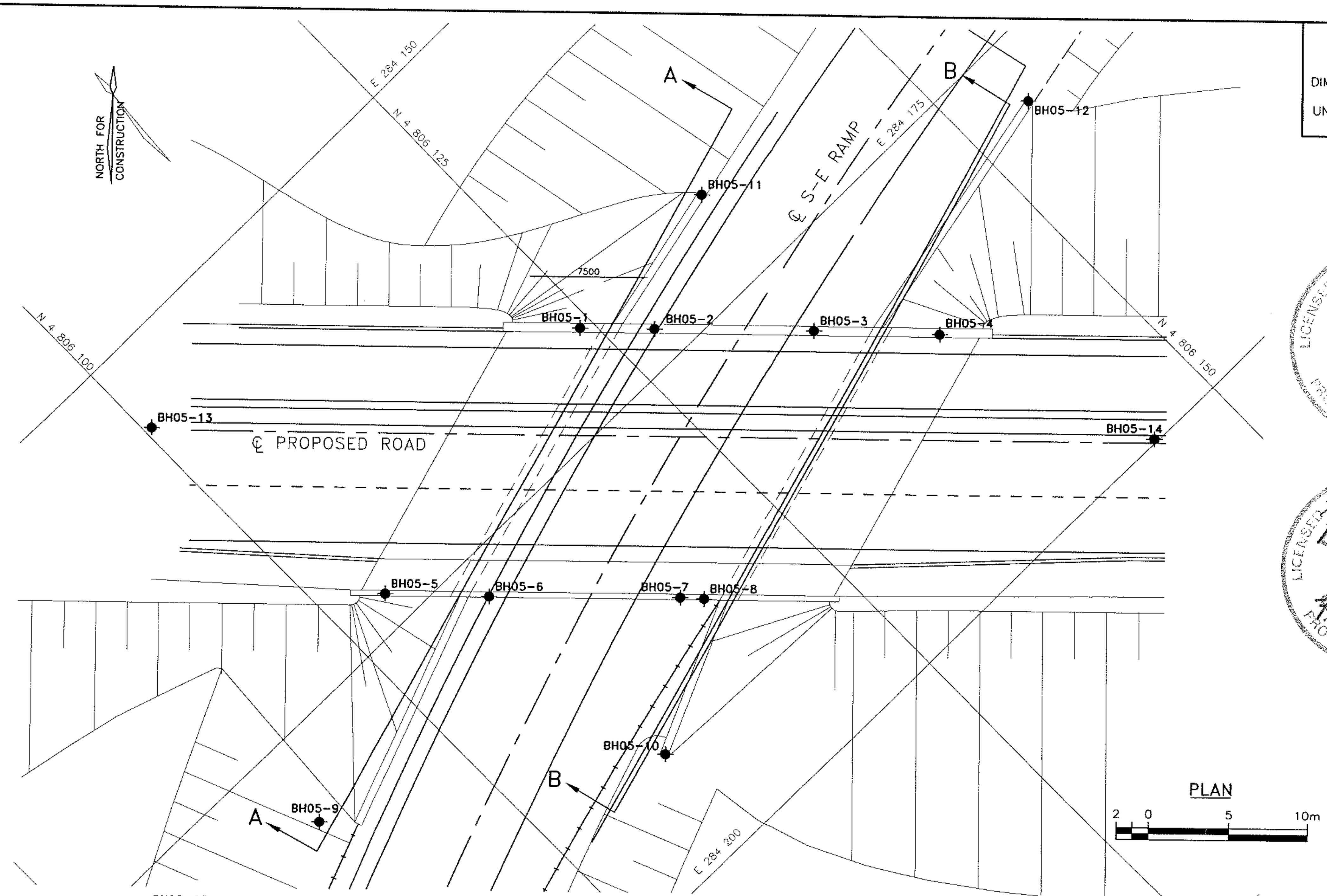
WARRANT: Always with these tender items.
NOTES TO DESIGNER:

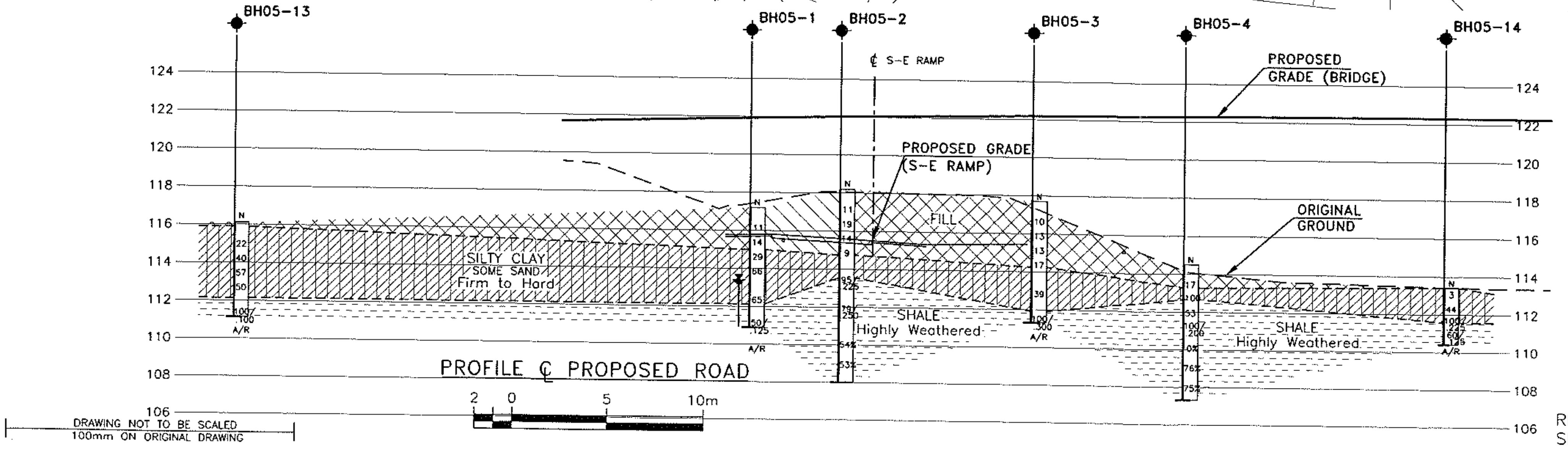
- *Do not use the generic RSS tender items and SP for sole-source RSS construction.*

- *Include quantities for excavation for RSS under the appropriate "Earth Excavation for Structure" and "Rock Excavation for Structure" tender items.*
- *Show limits of earth/rock excavation for RSS on the "Structural Excavation" Grading drawing; confirm plan dimensions of excavation with Pavements & Foundations Section.*
- *Include tender items "Roadway Protection", "Track Protection", and "Unwatering Structure Excavation" where recommended by the Pavements & Foundations Section.*
- *Include appropriate "Backfill for RSS" tender items and estimated quantities; confirm method of calculating backfill quantity with Pavements & Foundations Section.*
- *Where the MTO-designed L-shaped footing is used to support a concrete barrier wall or parapet wall on RSS, show the dimensions and reinforcing for the L-shaped footing as well as the barrier wall or parapet wall, and include an NSSP to the appropriate barrier wall or parapet wall tender item for payment.*
- *Where the L-shaped footing supports a concrete barrier wall/parapet wall that is continuous with a barrier wall/parapet wall on structure, consider an NSSP to restrict construction timing of the L-shaped footing.*
- *For each RSS False Abutment tender item include a "Scope" NSSP to cover i) both abutments, and ii) all contiguous wingwalls for payment under the item.*
- *Where several RSS are grouped under a single RSS tender item for payment, ensure all such RSS have the same Application, Performance and Appearance attributes.*

Appendix D

Drawings





REFER TO DWG. 2 FOR SECTION A-A AND B-B

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST NO.

GWP NO.

SHEET

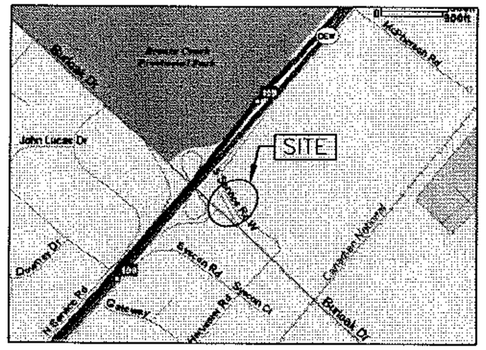
QEW - BURLOAK DRIVE INTERCHANGE
STREET 'A' OVER S-E RAMP
BOREHOLE LOCATIONS AND SOIL STRATA

McCORMICK RANKIN CORPORATION

THURBER ENGINEERING LTD.

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Feb 14/06
PROVINCE OF ONTARIO

P. J. BRANCO
Feb-14/06



KEYPLAN

LEGEND

●	Bore Hole
◆	Dynamic Cone Penetration Test (cone)
◈	Bore Hole & Cone
N	Blows/ 0.3m (Std Pen Test, 475 J/ blow)
CONE	Blows/ 0.3m (60° Cone, 475 J/ blow)
PH	Pressure, Hydraulic
▼	WL
⊤	Head Artesian Water
│	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	EASTING	NORTHING
BH05-1	117.2	284169.5	4806124.2
BH05-2	118.2	284172.8	4806127.5
BH05-3	117.7	284179.9	4806134.6
BH05-4	114.5	284185.6	4806140.1
BH05-5	119.5	284172.9	4806103.6
BH05-6	117.0	284177.7	4806108.2
BH05-7	117.9	284186.1	4806116.7
BH05-8	117.9	284187.2	4806117.7
BH05-9	119.1	284180.3	4806090.5
BH05-10	117.6	284192.5	4806109.0
BH05-11	118.2	284169.0	4806135.5
BH05-12	114.7	284179.1	4806154.4
BH05-13	116.1	284155.1	4806100.5
BH05-14	113.4	284199.8	4806145.1

— NOTE —

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

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK PJB		LOAD	DATE JAN. 2006
DRAWN JHL	CHK MRA	SITE	STRUCT	DWG. 1

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

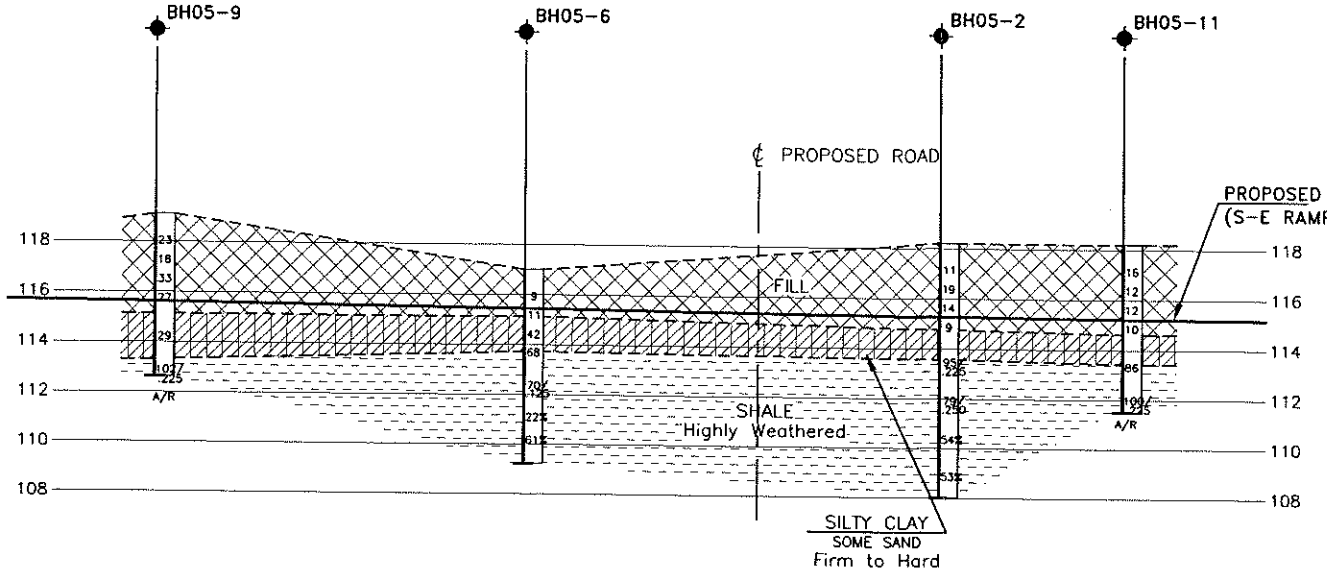
DIST NO.
GWP NO.

QEW – BURLOAK DRIVE
INTERCHANGE
STREET 'A' OVER S-E RAMP
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

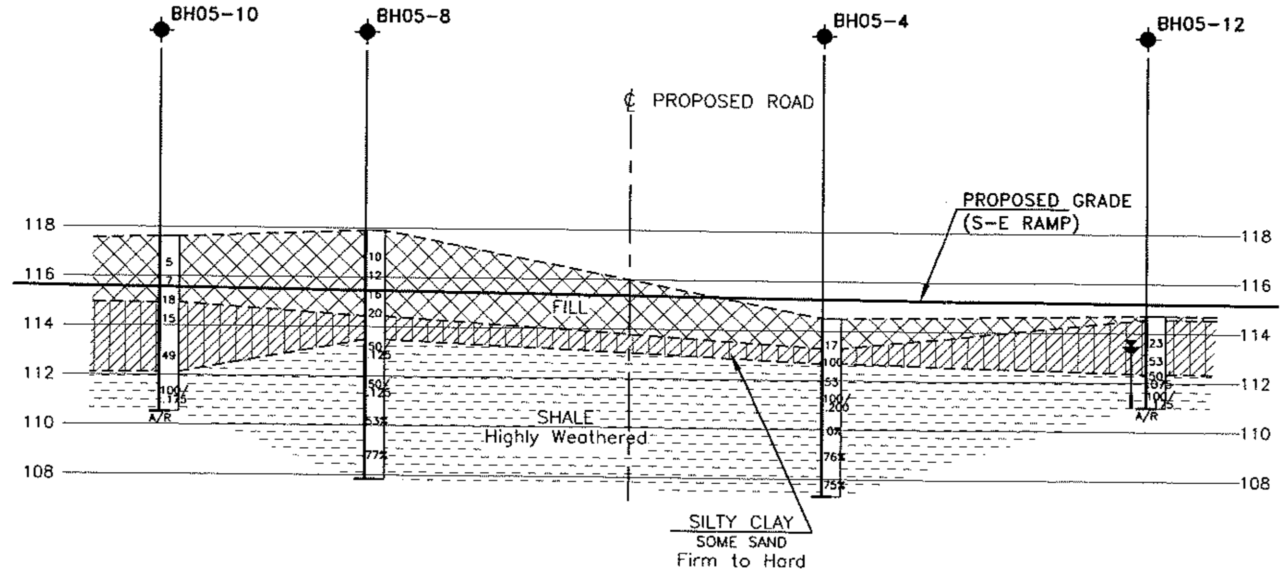
McCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.



SECTION A–A

2 0 5 10m

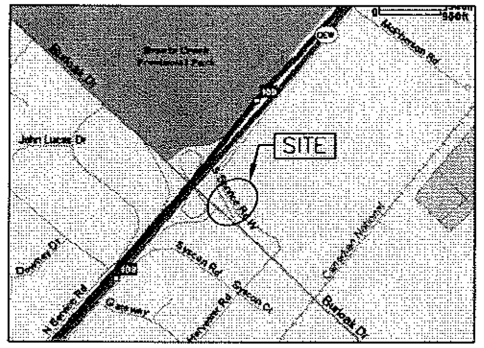


SECTION B–B

2 0 5 10m

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Feb 14/06
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. J. BRANDO
Feb 14/06
PROVINCE OF ONTARIO



KEYPLAN

LEGEND

●	Bore Hole
✦	Dynamic Cone Penetration Test (cone)
⊕	Bore Hole & Cone
N	Blows/ 0.3m (Std Pen Test, 475 J/ blow)
CONE	Blows/ 0.3m (60° Cone, 475 J/ blow)
PH	Pressure, Hydraulic
▼	WL
⊤	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	EASTING	NORTHING
BH05-1	117.2	284169.5	4806124.2
BH05-2	118.2	284172.8	4806127.5
BH05-3	117.7	284179.9	4806134.6
BH05-4	114.5	284185.6	4806140.1
BH05-5	119.5	284172.9	4806103.6
BH05-6	117.0	284177.7	4806108.2
BH05-7	117.9	284186.1	4806116.7
BH05-8	117.9	284187.2	4806117.7
BH05-9	119.1	284180.3	4806090.5
BH05-10	117.6	284192.5	4806109.0
BH05-11	118.2	284169.0	4806135.5
BH05-12	114.7	284179.1	4806154.4
BH05-13	116.1	284155.1	4806100.5
BH05-14	113.4	284199.8	4806145.1

— NOTE —

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

REFER TO DWG. 1 FOR
LOCATIONS OF SECTIONS.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN MRA	CHK PJB		LOAD	DATE JAN. 2006	
DRAWN JHL	CHK MRA	SITE	STRUCT	DWG. 2	

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

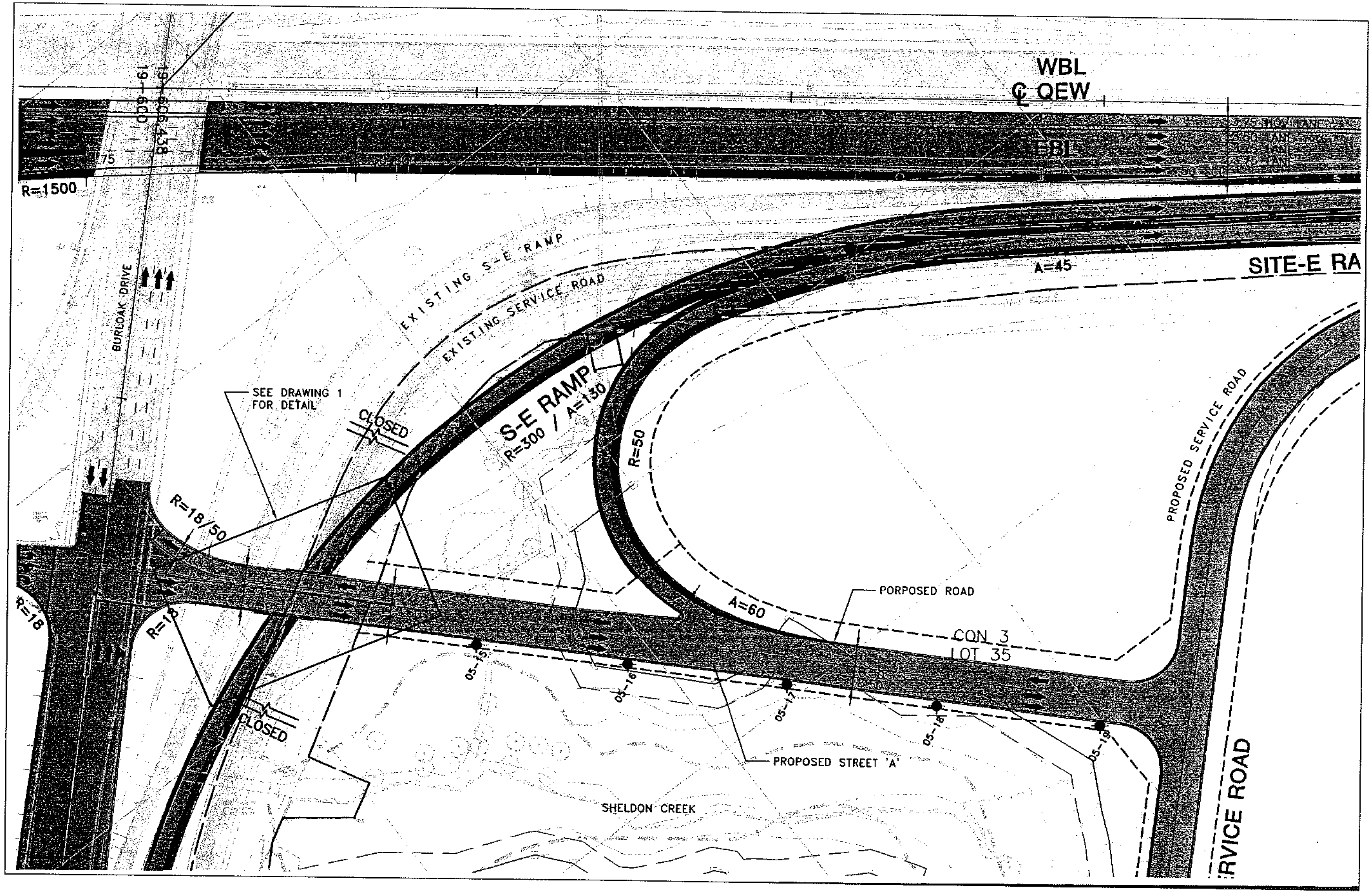
DIST NO.
GWP NO.

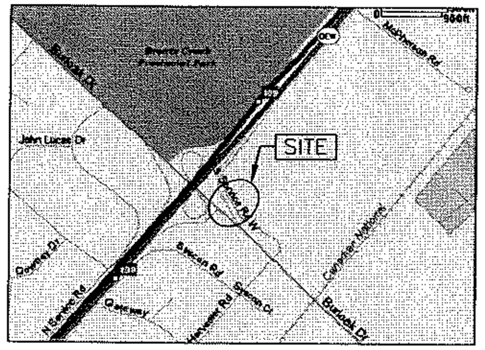
SHEET

QEW – BURLOAK DRIVE
INTERCHANGE
STREET 'A' OVER S–E RAMP
BOREHOLE LOCATIONS

McCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.





KEYPLAN

	LEGEND
●	Bore Hole
◆	Dynamic Cone Penetration Test (cone)
⊕	Bore Hole & Cone
N	Blows/ 0.3m (Std Pen Test, 475 J/ blow)
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PH	Pressure, Hydraulic
▼	WL
⊤	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	EASTING	NORTHING
BH05–15	113.2	284220.1	4806152.9
BH05–16	115.3	284259.4	4806192.3
BH05–17	114.7	284295.3	4806227.9
BH05–18	115.5	284331.4	4806263.8
BH05–19	116.2	284367.0	4806299.1

– NOTE –

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

M. R. ANDERSON
Feb 14/06

P. J. BRANCO
Feb 14/06

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK PJB		LOAD	DATE JAN. 2006
DRAWN JHL	CHK MRA	SITE	STRUCT	DWG. 3