

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
REPLACEMENT OF CPR OVERHEAD STRUCTURE  
HIGHWAY 401 WESTBOUND COLLECTORS  
JANE STREET TO KIPLING AVENUE  
G.W.P. 2147-01-00**

**Geocres Number: 30M11-231**

**Report to**

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**PART A: FOUNDATION INVESTIGATION REPORT**

## **1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted for the proposed replacement of the Highway 401 Westbound Collector - CPR Overhead structure located between Wendell Avenue and Weston Road. The new structure will accommodate widening of the Highway 401 westbound collector lanes.

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 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 foundations for the structure replacement.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation, under the Ministry of Transportation Ontario (MTO) Agreement Number 2005-E-0023.

## **2 SITE DESCRIPTION**

The existing CPR overhead structure carries the westbound collector lanes of Highway 401 over the CP Railway. The structure has a single span varying in length from approximately 20 m at the south end to near 30 m at the north end. Grades are near elevation 144 m on Highway 401 over the structure and elevation 136 m at the CPR level.

Photographs of the north end of the structure are provided in Appendix C.

The surrounding lands are primarily commercial and light industrial.

The general site area is located within the physiographic region referred to as the Peel Plain, a level to undulating region of massive to laminated, glaciolacustrine clay and silt. The clay and silt are underlain by glacial till deposits of silty clay to clayey silt.

Grey shale bedrock of the Georgian Bay Formation underlies the site. Bedrock topography maps indicate the bedrock surface is near elevation 100 m, sloping down towards a northwest to southeast trending bedrock valley immediately northeast of the site.



### 3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing adjacent to the north end of the existing structure on November 22 and 23, 2007 and January 11 and 14, 2008 as permissions to enter the adjoining commercial properties became available. One borehole was drilled near each abutment in conjunction with investigation for associated retaining wall realignment. Borehole RW-05 was located near the west abutment and terminated in bedrock at 36.7 m depth. Borehole RW-06 was drilled near the east abutment and terminated in very dense till at 36.9 m depth.

A supplementary investigation comprising three additional boreholes, including two near the south end of the abutments, was carried out between October 29 and November 4, 2008 to further evaluate foundation requirements. The additional boreholes were designated 08-01 to 08-03 and were advanced to depths of 16.5 to 31.1 m. Borehole 08-03 was drilled adjacent to the previous borehole RW-05 to obtain thin wall tube samples only.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawings in Appendix D.

Thurber positioned the boreholes in the field relative to local site features. The coordinates and ground surface elevations at the boreholes were subsequently established by J.D. Barnes Limited. The coordinates and elevations of the boreholes are given on the Borehole Locations and Soil Strata Drawing and on the individual Record of Borehole Sheets in Appendix A.

Solid stem augers were used to advance the boreholes. Samples were obtained using a split spoon sampler in conjunction with Standard Penetration Tests (SPT). Where soft to very stiff cohesive soils were encountered, the undrained shear strength was evaluated using the MTO shear vane. Thin wall tube samples of the cohesive deposits were recovered from the boreholes drilled during the supplementary investigation.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The inspector logged the soil and groundwater conditions encountered in the boreholes, and collected, labelled and arranged for transport of the samples to Thurber's laboratory.

Standpipe piezometers were installed in two boreholes to monitor groundwater levels. The completion details of the piezometers are presented in Table 3.1. The piezometers were subsequently decommissioned in accordance with the abandonment requirements of MOE Reg. 903. The boreholes without piezometers were grouted upon completion in accordance with the Regulation.

**Table 3.1 – Piezometer Details**

Location	Tip Position (m)		Completion Details
	Depth	Elevation	
RW-05	21.3	115.7	Borehole sidewalls caved to 21.3 m, sand filter and screen from 21.3 to 19.5 m, bentonite to surface.
RW-06	10.4	125.2	Bentonite to 10.4 m, sand filter and screen from 10.4 to 8.5 m, bentonite 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.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing. Thin wall tube samples were also selected for consolidation testing and triaxial compression tests. The results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

## 5 DESCRIPTION OF SUBSURFACE CONDITIONS

### 5.1 General

Reference is made to the Record of Borehole sheets in Appendix A and to the Borehole Locations and Soil Strata Drawings in Appendix D. An overall description of the stratigraphy based on the conditions encountered in the boreholes is given in the following paragraphs. However, the factual data presented in the borehole logs takes precedence over this general description and interpretation of the site conditions.

The soil stratigraphy encountered at this site generally consists of surficial pavement, topsoil and fill layers overlying silty clay, underlain by a thick deposit of silty clay till. Various deposits of sand, silt and till, as well as shale bedrock, were encountered within or below the clay till.

More detailed descriptions of the individual strata are presented below.

### 5.2 Pavement and Clay Fill

A 75 mm thick asphalt surface was encountered in borehole RW-06 located near the north end of the east abutment. The asphalt was underlain by a 225 mm thick concrete slab.

A 100 mm thick topsoil layer underlain by silty clay fill was encountered in borehole RW-05 located near the north end of the west abutment. The clay fill extended to 0.8 m depth (elevation 136.3 m). An SPT N-value of 8 blows/0.3 m was obtained in the fill, indicating a firm to stiff consistency. A moisture content of 17 % was measured.

In the boreholes drilled on the south shoulder of the Westbound Collector Lanes (boreholes 08-01 and 08-02), a rigid pavement structure was encountered, comprising 50 and 80 mm of asphalt over 200 and 230 mm of concrete overlying 300 mm of gravel.

The pavement structure on the Collector Lanes was underlain by silty clay approach fill to depths of 9.7 and 9.6 m depth (elevation 134.8 and 134.9 m). SPT N-values of 5 to 24 blows/0.3 m obtained in the clay fill indicate a firm to very stiff consistency. Moisture contents ranged from 17 to 25%. The results of grain size distribution analyses and Atterberg Limits tests carried out on two samples of the fill are presented in Figures B1 and B6 of Appendix B, respectively.

### 5.3 Silty Clay

Silty clay was encountered below the pavement and fill materials. The clay typically contained a trace of sand and was described as brown or grey.

SPT N-values of 10 to 15 blows/0.3 m were obtained in the clay in boreholes 08-02 and RW-05. Values of 25 and 31 blows/0.3 m were encountered in borehole RW-06. In situ vane testing measured undrained shear strengths of 196 to over 200 kPa. Based on these results, the consistency of the clay is stiff to hard.

The natural moisture content of the silty clay ranged from 15 to 29%.

The results of laboratory tests carried out on two samples were as follows:

Gravel %	0 to 1
Sand %	1 to 8
Silt %	32 to 39
Clay %	59 to 60
Liquid Limit	45 to 46
Plastic Limit	20

The results of these tests indicate that the silty clay is a CI soil (medium plasticity).

The grain size distribution curves are shown in Figure B2, Appendix B. The Atterberg Limits are plotted on Figure B7.

The thickness of the clay layer ranged from 2.0 to 2.6 m. The lower boundary was contacted at depths of 3.0 and 2.3 m (elevation 134.0 and 133.3 m) at the north end of the west and east abutments, respectively. Below the Collector Lanes, the lower boundary was encountered at 11.7 and 12.2 m depth (elevation 132.8 and 132.3 m) at the west and east abutment, respectively.

### 5.4 Silty Clay Till

The silty clay was underlain by silty clay till. The clay till was described as grey, contains trace gravel, and has a sand content typically ranging from some sand to sandy.

SPT N-values obtained in the clay till ranged from 15 to 56 blows/0.3 m. Slightly lower N-values of 10 to 22 blows/0.3 m (stiff to very stiff) were obtained in the supplementary boreholes (08-01 and 08-02). In situ vane testing in the clay till measured undrained shear strengths of 196 to over 200 kPa. Based on these results, the consistency of the clay till is stiff to hard.

The natural moisture content of recovered samples ranged from 11 to 20%.

The grain size distribution curves for ten samples tested are shown in Figures B3 and B4, Appendix B. The Atterberg Limits are plotted on Figures B8 and B9. Although rarely encountered in the boreholes, till soils often contain cobbles and boulders, and isolated occurrences should be anticipated during construction.

The results of the laboratory tests were as follows:

Gravel %	0 to 4
Sand %	18 to 51
Silt %	35 to 51
Clay %	11 to 33
Liquid Limit	22 to 37
Plastic Limit	13 to 19

The results of these tests indicate that the silty clay till is a CL to CI soil (low to medium plasticity).

Consolidated undrained (CU) triaxial tests with pore pressure measurements were conducted on two samples of the clay till. The results of the testing are presented in Appendix B and summarized in Table 5.1.

**Table 5.1 – CU Triaxial Test Parameters**

Borehole	Sample Depth (m)	w <sub>o</sub> (%)	Cell Pressure (kPa)	Maximum Deviator Stress (kPa)	Axial Strain at Maximum Deviator Stress (%)	Initial Tangent Modulus E <sub>i</sub> (MPa)	Secant Modulus at ½ Deviator Stress E <sub>50</sub> (MPa)
08-01	15.2-15.8	16	505	263	5.9	32	20
08-03	6.1-6.7	15	255	298	12.5	34	13

The results of consolidation testing conducted on four samples of the silty clay till (including one from Wendell Avenue obtained as part of the overall widening project) are included in Appendix B and summarized in Table 5.2.

**Table 5.2 – Consolidation Test Parameters**

Borehole	Sample Depth (m)	w <sub>o</sub> (%)	γ (kN/m <sup>3</sup> )	e <sub>o</sub>	p <sub>o</sub> ' (kPa)	p <sub>c</sub> ' (kPa)	OCR	C <sub>c</sub>	C <sub>r</sub>	c <sub>v</sub> (cm <sup>2</sup> /s)
08-01	15.2-15.8	15	21.7	0.442	310	650	2.1	0.15	0.018	0.050
08-03	6.1-6.7	16	21.0	0.499	130	450	3.5	0.17	0.020	0.100
08-03	15.2-15.8	14	22.0	0.381	280	550	2.0	0.20	0.019	0.004
08-04	9.1-9.8	16	21.5	0.438	200	625	3.1	0.19	0.018	0.003

Comparison of the existing and preconsolidation pressures (p<sub>o</sub>' and p<sub>c</sub>') derived from the test results indicate that the natural silty clay till is overconsolidated. The coefficient of consolidation, c<sub>v</sub>, reported in the table applies to the typical pressure range anticipated in the field. The compressibility characteristics will vary with depth in accordance with the moisture content and shear strength profiles.

In boreholes RW-05 and RW-06, the lower boundary of the till was encountered at depths of 33.5 and 27.4 m (elevation 103.5 and 108.2) respectively, indicating a unit thickness of 30.5 and 25.1 m. However, a 2.3 to 3.0 m thick layer of silty sand was encountered within



the till in both boreholes, and a thin layer of clayey silt was encountered above the sand layer at the east abutment.

In boreholes 08-01 and 08-02, a silt/sandy silt layer was encountered within the till at depths of 24.4 and 24.5 m (elevation 120.1 and 120.0 m). Borehole 08-01 was terminated in the sandy silt. In borehole 08-02, the silt layer was penetrated at 30.8 m depth (elevation 113.7 m), and drilling was terminated in clay till at 31.1 m (elevation 113.4 m).

### **5.5 Discontinuous Non-Cohesive Deposits**

A layer of silty sand to silt was encountered within the clay till at depths of 18.3 to 24.5 m (elevation 115.0 to 120.1 m). Where fully penetrated, the sand/silt layer was 2.3 to 6.3 m thick with a lower boundary at depths of 21.3 to 30.8 m (elevation 112.7 to 115.7 m). SPT N-values ranged from 20 to 42 blows/0.3 m, indicating a compact to dense condition.

Deposits of sandy silt to silty sand till, silt till and silty sand were encountered below the clay till in boreholes RW-05 and RW-06. SPT N-values obtained in these deposits ranged from 30 blows/0.3 m to 76 blows/0.15 m, indicating a dense to very dense condition. The lower boundary of the non-cohesive material in borehole RW-05 was encountered at 36.0 m depth (elevation 101.0 m). Borehole RW-06 was terminated in the very dense cohesionless deposits at 36.9 m depth (elevation 98.7 m).

The natural moisture content of recovered samples ranged from 10 to 16 %.

The results of laboratory tests carried out on five samples were as follows:

Gravel %	0 to 10
Sand %	8 to 73
Silt %	23 to 79
Clay %	4 to 10

The grain size distribution curves for the samples tested are shown in Figure B3, Appendix B. Although not encountered in the boreholes, glacial till may contain cobbles and large boulders.

### **5.6 Discontinuous Cohesive Deposit**

A 0.8 m thick layer of clayey silt was encountered within the clay till unit, immediately above the silty sand layer in borehole RW-06 at the east abutment. An SPT N-value of 26 blows/0.3 m (very stiff) and a moisture content of 18 % were measured in this layer.

### **5.7 Bedrock**

At the west abutment (borehole RW-05), weathered grey shale was encountered below silty sand till at 36.0 m depth (elevation 101.0 m). The borehole was terminated in the shale at 36.7 m depth.

## 5.8 Groundwater

The initial and final groundwater depths and elevations measured in the boreholes and piezometers are shown in Table 5.3.

**Table 5.3 – Groundwater Depths and Elevations**

Borehole	Piezometer Tip Depth (m)	Date	Water Level (m)		Event
			Depth	Elevation	
RW-05	21.3	23-Dec-07	22.6	114.4	Upon completion
		09-Jan-08	17.7	119.3	In piezometer
		21-Jan-08	17.5	119.5	In piezometer
		03-Mar-08	17.6	119.4	In piezometer
		04-Apr-08	17.1	119.9	In piezometer
		08-May-08	17.0	120.0	In piezometer
RW-06	10.4	14-Jan-08	24.1	111.5	Upon completion
		21-Jan-08	10.4	125.2	In piezometer
		03-Mar-08	7.8	127.8	In piezometer*
		08-May-08	9.0	126.6	In piezometer* *cap missing

The above water levels reflect the unstabilized conditions in the boreholes upon completion of drilling or the piezometric head at the level of the piezometer tips at the time of the readings. The measurements are short-term observations and seasonal fluctuations of the groundwater level are to be expected.

## 6 MISCELLANEOUS

J.D. Barnes Limited determined the co-ordinates and ground elevations at the boreholes following completion of the site investigation.

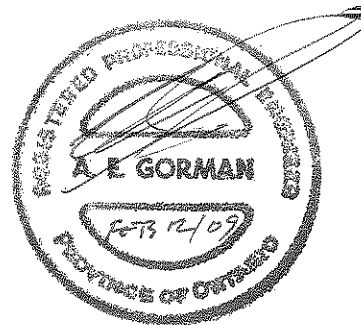
DBW Drilling Limited and Walker Drilling Ltd. supplied and operated the drilling and sampling equipment for the initial and supplementary field programs, respectively. Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. George Azzopardi, Mr. David Elwood and Mr. Will Ball of Thurber.

Supervision of the field program, interpretation of the field data, and preparation of the report was performed by Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair Gorman, P.Eng., and by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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**PART B: FOUNDATION DESIGN REPORT**

## **7 INTRODUCTION**

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist selection and design of the foundation system for replacement of the existing Highway 401 Westbound Collector - CPR overhead structure.

The existing CPR overhead structure carries the westbound collector lanes of Highway 401 over the CP Railway. The structure has a single span varying in length from approximately 20 m at the south end to near 30 m at the north end. Grades are near elevation 144 m on Highway 401 over the structure and elevation 136 m at the CPR level. An approximate 9 m high counterfort retaining wall supports the north side of the approach fill adjacent to the structure.

Contract drawings for the previous structure widening (Contract No. 65-55) indicate that the existing overpass is supported on spread footings founded at elevation 134.4 m. The footing widths vary from 3.7 to 4.3 m.

The new Westbound Collector structure will have a width of 25.4 m and a single span length ranging from approximately 21.3 m at the south end to 28.8 m at the north end. The minimum overhead clearance will be 7.0 m.

Current plans call for the new approach fills to be constructed as an RSS wall, effectively realigning the existing retaining wall to the north of the current alignment. Foundation recommendations regarding design and construction of the new retaining wall and approaches are presented in a separate report.

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 FOUNDATIONS

In general terms, the soil stratigraphy encountered at this site consists of a surficial pavement, topsoil and fill layer overlying silty clay, underlain by a thick deposit of silty clay till. Discontinuous deposits of silty sand, silt and till, as well as shale bedrock were encountered at depth.

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

A comparison of the technical advantages and disadvantages of alternative foundation schemes is presented in Appendix C. Initial consideration was given to spread footings on native soil or engineered fill, driven steel H-piles, and caissons (drilled shafts).

As noted above, the existing structure is supported on spread footings bearing on the native undisturbed silty clay soil.

### 8.1 Spread Footings on Native Soil

The new structure footings should be founded on undisturbed native soil at the same level as the existing footings, elevation 134.4 m. The founding level of the existing footings is based on previous contract drawings and should be verified during design.

Where required to penetrate existing fill, footings should be stepped down away from the edge of the existing footings to avoid undermining the existing foundation.

Footings bearing on very stiff to hard native silty clay at the same level as the existing footings may be designed using the following resistance values:

Factored Geotechnical Resistance at ULS	330 kPa
Geotechnical Resistance at SLS	220 kPa

The SLS value is based on a maximum footing width of 4.0 m and a total settlement not exceeding 25 mm due to the applied footing load. However, additional settlement of the footing at the north end of the structure will result from compression of the clay and clay till founding soils under the load applied by the new approach fill adjacent to the structure. For a footing width of 4 m applying a bearing pressure of 220 kPa, and an approach fill height of 9 m, the estimated maximum settlement of the footing is approximately 65 mm. Additional discussions regarding settlement of the structure and approaches are presented in Section 8.5.

The new footings should be tied into the existing foundation of the core lane structure to minimize differential movement between the structures.

The resistance values are for vertical, concentric loads. In accordance with the CHBDC Clauses 6.7.3 and 6.7.4, the design must also account for the effects of any eccentric or inclined loads applied.

The lateral resistance of the footings founded on silty clay may be computed using an unfactored friction coefficient of 0.45. 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, has been adequately prepared to receive concrete, and comprises native soil below the level of all fill.

All footings should be provided with a minimum of 1.2 m of earth cover over the footing base as protection against frost action. It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation. Typically, 25 mm of extruded polystyrene insulation is equivalent to 600 mm of soil cover.

## 8.2 Spread Footings on Engineered Fill

Construction of an engineered fill pad for support of spread footings would require excavation below the level of the existing footings supporting the core lane structure, the existing retaining wall, and part of the new structure during construction staging. This excavation could potentially result in undermining of the existing structure foundations. Construction of spread footings on engineered fill is therefore not recommended.

## 8.3 Driven Steel Piles

The geotechnical conditions encountered at this site are considered suitable for driven steel H-pile foundations.

The piles should be driven to refusal in the very dense silty sand/sandy silt till. The estimated pile tip elevations and the geotechnical resistances recommended for design of piles founded in the very dense native soils are as follows:

**Table 8.1 – Pile Geotechnical Resistance**

Pile Section	Geotechnical Resistance (kN)		Estimated Pile Tip Elevation (m)
	Factored ULS	SLS (25 mm settlement)	
HP 310 X 110	1 600	1 400	West Abutment: 101.5
HP 360 X 132	1 800	1 600	East Abutment: 100.0

Higher resistances could be achieved by driving the piles to the underlying shale bedrock. Coring of the bedrock would be required to confirm the depth and quality of the shale.

The pile tip elevations are presented for estimating purposes only and may vary along the abutment locations. The actual pile tip elevations will be controlled as described in the next section.

The structural resistance of the pile must be checked by the structural designer.

### 8.3.1 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

Pre-augering of pile installations within 6 m of the existing core lane structure is recommended to minimize any impact of driving operations on the existing structure. The pre-auger hole should be 150 to 200 mm in diameter and extend to depth of approximately 10 m (elevation 125 m). The existing structure should be monitored during pile driving operations.

Pile driving should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. “R” must have minimum values shown of 3,200 kN for HP 310x110 and 3,600 kN for HP 360x132.

The tips of all piles should be fitted with H-section driving shoes as per OPSD 3000.100.

### 8.3.2 Downdrag

Downdrag forces will develop along the length of pile embedded in the silty clay and silty clay till deposits due to increased approach embankment loads. For design purposes, an unfactored downdrag force of 650 kN per pile is recommended to evaluate the impact of downdrag.

In accordance with Section 6.8.4 of the CHBDC, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. A check should be performed to confirm that the factored permanent plus downdrag loads do not exceed the factored below-ground structural resistance of the pile at the neutral plane. At this site, the neutral plane will be close to the lower clay till boundary, near elevation 115.0 m. As per the CHBDC, live loads and downdrag loads are not combined.

The factored structural resistance of the pile (factored for structural design and below-ground design) to be used in the downdrag check are:

HP 310 X 110	2,800 kN
HP 360 X 132	3,400 kN

### 8.3.3 Lateral Resistance of Piles

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

$$\begin{aligned}
 k_s &= n_h \cdot z / D \quad (\text{kN/m}^3) \quad \text{for cohesionless soils} \\
 &= 67 \cdot c_u / D \quad (\text{kN/m}^3) \quad \text{for cohesive soils} \\
 p_{ult} &= 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})
 \end{aligned}$$

where

$$\begin{aligned}
 z &= \text{depth of embedment of pile in metres} \\
 D &= \text{pile width in metres} \\
 n_h &= \text{coefficient of horizontal subgrade reaction (Table 8.2)} \\
 c_u &= \text{undrained shear strength (Table 8.2)} \\
 \gamma &= \text{bulk unit weight (Table 8.2), use submerged unit weight below water table} \\
 K_p &= \text{passive earth pressure coefficient (Table 8.2)}
 \end{aligned}$$

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times D$  ( $\text{kN/m}$ ), where  $k_s$  is the coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  $D$  is the pile width (m) and  $L$  is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \times L \times D$ .

**Table 8.2 – Parameters for Lateral Pile Resistance**

Location	Elevation	Soil	$n_h$ ( $\text{kN/m}^3$ )	$c_u$ ( $\text{kPa}$ )	$K_p$	Unit Weight* ( $\text{kN/m}^3$ )
West Abutment (RW-05)	136 to 134	Clay	-	90	2.8	19
	134 to 119	Clay Till	-	150	2.8	19
	119 to 116	Silty Sand	6,000	-	3.5	10
	116 to 104	Clay Till	-	200	3.0	10
East Abutment (RW-06)	135 to 133	Clay	-	150	2.8	19
	133 to 128	Clay Till	-	150	2.8	19
	128 to 115	Clay Till	-	150	2.8	9
	115 to 113	Silty Sand	4,000	-	3.3	10
	113 to 108	Clay Till	-	200	3.0	10

\*Buoyant unit weight below the water table.

The total horizontal passive resistance of a single pile in very stiff clay/till, for a top of pile at or below elevation 134.0 m, should not exceed the following values:

**Table 8.3 – Maximum Lateral Passive Resistance of Piles**

Pile	Maximum Lateral Resistance	
	Factored ULS	SLS
HP 310X110	200 kN	110 kN
HP360X132	240 kN	140 kN



The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.4. Intermediate values may be obtained by linear interpolation.

**Table 8.4 – Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

\* where D is the width of pile

#### 8.4 Caissons

The use of augered caissons is not recommended in view of the depth to suitable bearing material (greater than 30 m) and the presence of water-bearing sands below the clay till deposit. Constructing caissons would require use of a liner sealed below the sand layer and/or slurry methods to control groundwater, support the sidewalls of the shaft, and prevent heave in the base.

#### 8.5 Settlement Considerations

The site is underlain by over 20 m of stiff to hard silty clay and clay till. Construction of new spread footings and widening of the structure approach fill over previously unloaded areas will apply additional loads to the cohesive foundation soils and result in compression of these deposits, with consequent settlement of the new structure and approach fills.

The preliminary General Arrangement drawing for the new structure indicates that the alignment of the east abutment will be rotated inwards at the north end of the structure, narrowing the north opening. New foundations will therefore be constructed over previously unloaded areas and additional abutment backfill will be required, in addition to the structure and approach fill widening.

If footings are selected as the foundation type, significant differential settlement would be anticipated along the length of the new abutments due to the revised loading conditions, with the most settlement at the north end of the new structure. Relatively little settlement would be expected at the south end where loading conditions will not change significantly, and the existing structure carrying the Express Lanes should experience negligible effect. The settlement variation is difficult to predict due to the complexity of the new loading conditions.

The relative magnitude of settlement to be expected under the loading of new spread footings and the weight of the RSS wall fill was estimated using elastic analysis and one-

dimensional consolidation theory. The analyses were conducted using an in-house spreadsheet program as well as Rocscience Phase<sup>2</sup> and Settle3D finite element software. Geotechnical parameters for the analysis were deduced from the results of the oedometer and triaxial testing, and moisture content correlations developed by Thurber.

Following initial settlement computations, discussions were held with structural designers from McCormick Rankin to evaluate foundation options in view of the preliminary findings. From the discussions, we understand that bridge construction will be carried out in stages:

1. In Stage 1A, the new RSS wall will be constructed along the north side of the existing retaining wall supporting the north side of the collector lanes, and the north section of the new bridge footing and abutment will be constructed. The existing bridge structure will remain in use during this stage.
2. Stage 1B-1D will involve demolition of the north part of the existing structure and construction of the middle section of the new structure. Traffic will be maintained over the south part of the existing structure. The deck slab will then be constructed over both the Stage 1A and Stage 1B-1D sections.
3. For Stage 2A-2C, traffic will be shifted to the new structure and the south part of the existing structure will be replaced.

The predicted settlement along the existing and new east abutment during the various stages of construction are illustrated on Figures C1 and C2 of Appendix C. The figures indicate the following:

- Figure C1 shows the estimated foundation settlement along the existing east abutment from construction of the new RSS wall and footing, assuming the existing structure will remain in place over one construction season (6 months). Settlement of the foundation soils under the north end of the existing east abutment is expected to be in the order of 25 to 30 mm.
- Figure C2 (Stage 1A) indicates a maximum settlement in the order of 45 mm under the new footing over the first construction season (6 months).
- Figure C2 (Stage 1B) indicates an approximate settlement of 30 mm under the middle section of the new structure during Stage 1B abutment construction, prior to placement of the deck slab (assuming 3 months for Stage 1B construction).
- Figure C2 (Stage 1D) shows the anticipated settlement under the Stage 1 sections immediately before Stage 2 construction begins, assuming another construction season passes (6 months).
- Figure C2 (Final Stage) indicates that the maximum total long-term settlement anticipated under the new structure will range between approximately 65 mm under the Stage 1A section, 50 mm under the Stage 1B-1D section, and 20 mm under the Stage 2 section.

Settlements along the west abutment are expected to be less than along the east abutment, as the existing and new abutments on the west side are essentially on the same alignment.

The capability of the existing and new structure to accommodate the anticipated settlement during each stage of construction should be confirmed by the structural designer.

The applied loads are expected to remain below the preconsolidation pressure of the silty clay and clay till on site. The settlements were therefore computed using an estimated recompression index,  $C_r$ , and are expected to occur relatively quickly during and after load application.

The estimated settlement values should be considered approximate and indicative only of the relative magnitude of settlement anticipated. A monitoring program should be implemented to measure actual settlement of the existing and new structures during and after construction, to confirm the settlement predictions, and to assess the performance of the structures and determine if any remedial measures are required. An NSSP for the monitoring program should be included in the Contract documents; the suggested NSSP is included in the Foundation Design Report for the Retaining Wall.

## **8.6 Recommended Foundation**

To address the potential issues noted above with regards to settlement of new spread footings, pile foundations driven to refusal would be the preferred foundation type to support the proposed replacement structure. However, provided the proposed structure can tolerate the estimated settlements, the use of spread footings is recommended based on the following considerations:

- Spread footings are substantially less costly than deep pile foundations.
- Ease of construction.
- Pile foundations would reduce settlement of the new structure but would not decrease settlement experienced by the existing structure due to the RSS loading.
- Maneuvering the large pile driving equipment required to install long piles within confined areas during staged construction, particularly battered piles and piles near the ends of the bridge segments, may not be practical.
- Flatter access slopes would be required to mobilize large pile driving equipment into the foundation area, requiring increased excavation into the existing approach fill and a greater length of roadway protection.

The new structure should be tied in to the existing core structure foundation to minimize the potential for differential movement between the adjacent structures. A settlement monitoring program is recommended to confirm the performance of the foundations.

### **8.7 Abutment Type**

Use of a rigid frame abutment type is recommended to be compatible with the recommended spread footing foundation system and the existing structure to the south.

## **9 EXCAVATION AND DEWATERING**

Excavation for pile cap or spread footing construction will extend through surficial pavement and fill layers and into native silty clay. The excavation depth is expected to be less than 3 m below grade at the railway level, and approximately 10 m below the level of Highway 401.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and in accordance with Special Provision 902S01. For the purposes of the OHSA, the surficial fill materials are classified as Type 3 soils and the underlying very stiff native soils are classified as Type 2 soils.

Use of a hydraulic excavator should be suitable for excavation in the overburden. Provision should be made for handling of the pavement materials, potential obstructions in the fill, and possible cobbles or boulders.

Care must be taken to avoid undermining or otherwise disturbing the existing foundations during excavation.

Roadway and railway protection should be supplied in accordance with SP 105S19 and designed for Performance Level 2. The following soil parameters are recommended for design of the temporary shoring system, assuming a level surface behind the wall:

Unit Weight = 20 kN/m<sup>3</sup>

Active Earth Pressure Coefficient,  $K_a$  = 0.3

Passive Earth Pressure Coefficient,  $K_p$  = 3.0

Water was not observed in the boreholes within the anticipated excavation depths, during or upon completion of drilling. Perched water may be encountered locally within the surficial granular materials or fill. Considering the consistency and low permeability of the predominant clay soils on site, dewatering using sumps and pumps is considered feasible.

The design of any dewatering system that may be required is the responsibility of the Contractor.

## **10 BACKFILL AND LATERAL EARTH PRESSURES**

Granular backfill is recommended adjacent to the abutments. The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150.

The design of the abutment walls must incorporate a subdrain as shown in OPSD 3190.100.

Compaction equipment to be used adjacent to abutment walls must be restricted in accordance with OPSS 501.07.

Earth pressures acting on the abutment walls may be assumed to be triangular and to be governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC (2006) 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 below)

$\gamma$  = unit weight of retained soil (see Table below)

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

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the walls are dependent on the material used as backfill. Typical values for granular backfill are shown in Table 10.1.

**Table 10.1 – Earth Pressure Coefficients (K)**

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II		OPSS Granular B Type I	
	$\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		$\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	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	-

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.

The factors in Table 10.1 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.

## 11 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.09

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 1.0 should be used in seismic design.

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method<sup>1</sup>. Using this method, it was determined that the foundation soils are not in danger of liquefaction.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. The seismic earth pressure coefficients to be used in design at this site are shown in Table 11.1.

**Table 11.1 – Earth Pressure Coefficients (K) for Seismic Design**

Condition	Earth Pressure Coefficient (K) for Earthquake Loading			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active*, $K_{AE}$ (Unrestrained Wall)	0.28	0.47	0.32	0.60
At rest**, $K_{OE}$ (Restrained Wall)	0.55	-	0.59	-
Passive*, $K_{PE}$ (Movement Towards Soil Mass)	6.5	-	5.1	-

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods

<sup>1</sup> Seed, H.B. and Idriss, I.M. 1971, “Simplified Procedure for Evaluating Soil Liquefaction Potential” *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 101, No. SM9, September, pp. 1249 – 1273.

## 12 CONSTRUCTION CONCERNS

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

- Excavation adjacent to the existing abutment and retaining walls must not undermine the existing footings or otherwise disturb the existing wall.
- The founding level of the existing footings may vary from that shown on the previous contract drawings and should be confirmed prior to tendering.
- The possibility of encountering obstructions in the fill or cobbles and boulders during excavation or pile driving.
- Monitoring of the existing and new structures should be carried out during and following construction, to confirm the settlement estimates presented in this report and assess the need for remedial action.
- Confined working space during staged construction may limit the size of equipment employed during excavation or pile driving.

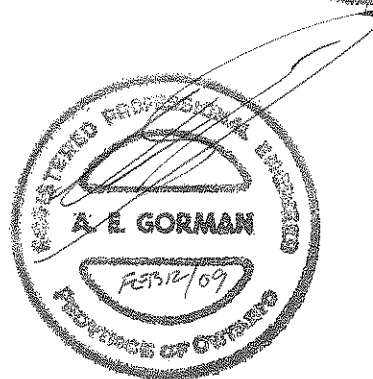
## 13 CLOSURE

Engineering analysis and preparation of the foundation design report was conducted by Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair Gorman, P.Eng., and by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

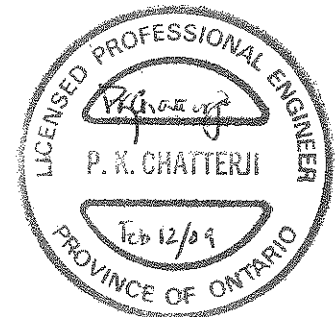
Thurber Engineering Ltd.  
Murray R. Anderson, P.Eng., M.Eng.  
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Project Manager



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Review Principal



## **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 <sup>(1)</sup> '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

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
<b>Fresh (FR)</b>	No visible signs of weathering.		CLAYSTONE
<b>Fresh Jointed (FJ)</b>	Weathering limited to the surface of major discontinuities.		SILTSTONE
<b>Slightly Weathered (SW)</b>	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SANDSTONE
<b>Moderately Weathered (MW)</b>	Weathering extends throughout the rock mass, but the rock material is not friable.		COAL
<b>Highly Weathered (HW)</b>	Weathering extends throughout the rock mass and the rock is partly friable.		Bedrock (general)
<b>Completely Weathered (CW)</b>	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	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

<u>TERMS</u>	
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.

# RECORD OF BOREHOLE No RW-05

1 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 699.5 E 302 397.1 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2007.11.22 - 2007.11.23 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								20 40 60 80 100										
137.0																		
0.0	TOPSOIL: (100mm)		1	SS	8		137											
0.1	Silty CLAY, trace to some sand, trace gravel, occasional wood fibres																	
136.3	Stiff Brown (FILL)																	
0.8	Silty CLAY, trace sand		2	SS	10		136											
	Stiff Brown to Mottled Brown-Grey																	
			3	SS	14		135											
			4	SS	15		134							1 8 32 59				
134.0	Silty CLAY, some sand to sandy, trace gravel		5	SS	22		134											
3.0	Very Stiff Brown (TILL)																	
	Grey		6	SS	25		133											
			7	SS	25		132											
			8	SS	23		131											
							130											
			9	SS	15		129							1 24 51 24				
							128											
			10	SS	19													

Continued Next Page

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

# RECORD OF BOREHOLE No RW-05

2 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 699.5 E 302 397.1 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2007.11.22 - 2007.11.23 CHECKED BY MRA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	20 40 60		
Continued From Previous Page													
	Silty CLAY, some sand to sandy, trace gravel Very Stiff Brown (TILL)		11	SS	23								
			12	SS	27								
			13	SS	29								
			14	SS	16								
			15	SS	34								
118.7 18.3	Silty SAND, fine grained Dense Grey Wet		16	SS	42								

Continued Next Page

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

# RECORD OF BOREHOLE No RW-05

3 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 699.5 E 302 397.1 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2007.11.22 - 2007.11.23 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
	Continued From Previous Page														
115.7	Silty SAND, fine grained Dense Grey Wet		17	SS	35		117							0 65 29 6	
21.3	Silty CLAY, some sand to sandy, trace gravel Hard to Very Stiff Grey (TILL)		18	SS	45		116								
							115								
							114								
							113								
			19	SS	52		112								
							111								
							110								
	occasional sand seams		20	SS	16		109							3 51 35 11	
							108								

Continued Next Page

+<sup>3</sup>, x<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No RW-05

4 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 699.5 E 302 397.1  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers  
 DATUM Geodetic DATE 2007.11.22 - 2007.11.23  
 ORIGINATED BY GA  
 COMPILED BY ES  
 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20	40	60	80	100		
								PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT				
								W <sub>p</sub>	W	W <sub>L</sub>				
103.5	Silty CLAY, some sand to sandy, trace gravel Hard Grey (TILL)		21	SS	50		107							
							106							
							105							
							104							
33.5	Silty SAND, some gravel, trace clay, occasional shale fragments Very Dense Grey (TILL)		22	SS	105		103							10 47 34 9
							102							
101.1			23	SS	125/ .250		101							
36.0	Highly weathered, grey SHALE													
100.3			24	SS	100/ .100									
36.7	END OF BOREHOLE AT 36.7m. BOREHOLE OPEN TO 29.6m AND WATER LEVEL AT 22.6m UPON COMPLETION. Piezometer installation consists of 30mm diameter schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2008.01.09 17.7 119.3 2008.01.21 17.5 119.5 2008.03.03 17.6 119.4 2008.04.04 17.1 119.9 2008.05.08 17.0 120.0													

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No RW-06

1 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 713.9 E 302 441.6 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2008.01.11 - 2008.01.14 CHECKED BY MRA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
135.6													
0.0	ASPHALT: (75mm)												
0.1													
135.3	CONCRETE: (225mm)												
0.3													
	Silty CLAY, trace sand, oxide staining Very Stiff to Hard Brown		1	SS	25								
			2	SS	31								
133.3													
2.3	Silty CLAY, some sand to sandy, trace gravel Very Stiff to Hard Brown (TILL)		3	SS	39								0 22 46 32
			4	SS	30								
			5	SS	20								
			6	SS	15								
			7	SS	15								0 22 47 31
			8	SS	21								
			9	SS	25								

Continued Next Page

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



# RECORD OF BOREHOLE No RW-06

2 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 713.9 E 302 441.6 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2008.01.11 - 2008.01.14 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								20 40 60 80 100	20 40 60 80 100						20 40 60	
	Continued From Previous Page															
	Silty CLAY, some sand to sandy, trace gravel Very Stiff to Hard Grey (TILL)		10	SS	35		125									
							124									
			11	SS	24		123									
							122									
			12	SS	25		121									
							120									
			13	SS	35		119									
							118									
			14	SS	21		117									
							116									
			15	SS	52											
115.8																
19.8	Clayey SILT															

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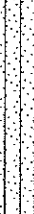


+ 3 x 3 : Numbers refer to  
Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No RW-06

3 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 713.9 E 302 441.6 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2008.01.11 - 2008.01.14 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
Continued From Previous Page								20 40 60 80 100										
115.0	Very Stiff Grey		16	SS	26													
20.6	Silly SAND, fine grained Compact Grey Wet																	
			17	SS	22										0 73 23 4			
112.7																		
22.9	Silty CLAY, some sand to sandy, trace gravel Hard Grey (TILL)																	
			18	SS	56													
108.2																		
27.4	SILT, some sand, trace gravel Dense Grey Damp (TILL)		19	SS	47													

Continued Next Page

+ 3, x 3. Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No RW-06

4 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION Weston Road to Highway 400, N 4 841 713.9 E 302 441.6 ORIGINATED BY GA  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY ES  
 DATUM Geodetic DATE 2008.01.11 - 2008.01.14 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								20	40	60	80	100		
								WATER CONTENT (%)						
								20	40	60				
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT						
								W <sub>p</sub>	W	W <sub>L</sub>				
105.3	SILT, some sand, trace gravel													
30.3	Dense Grey Damp (TILL)		20	SS	30									3 61 32 4
	Silty SAND, trace gravel													
	Dense Grey Wet													
102.1														
33.5	Sandy SILT, trace gravel		21	SS	64/									
	Very Dense Grey Wet (TILL)				150									
	occasional shale fragments		22	SS	73/									
					150									
98.7			23	SS	76/									
					150									
36.9	END OF BOREHOLE AT 36.9m. BOREHOLE OPEN TO 36.6m AND WATER LEVEL AT 24.1m UPON COMPLETION. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2008.01.21 10.4 125.2 2008.03.03 7.8 127.8* 2008.05.08 9.0 126.6* * Cap missing													

ONTMT4S 1122.GPJ 8/19/08

# RECORD OF BOREHOLE No 08-01

1 OF 3

METRIC

G.W.P. 2147-01-00 LOCATION CPR Overpass, N 4 841 668.1 E 302 389.0 ORIGINATED BY DEE  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2008.11.03 - 2008.11.03 CHECKED BY MRA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	W <sub>p</sub> W W <sub>L</sub>	W <sub>p</sub> W W <sub>L</sub>		
144.5													
0.0	PAVEMENT: 80mm asphalt over 230mm concrete over 300mm gravel												
143.9													
0.6	Silty CLAY, trace to some sand Firm to Very Stiff Brown (FILL)												
			1	SS	5								
			2	SS	9								
			3	SS	9								
			4	SS	15								
			5	SS	17								
			6	SS	15								
134.8													
9.7	Silty CLAY, trace sand Stiff												

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

## METRIC

ORIGINATED BY DEE

COMPILED BY AN

CHECKED BY MRA

Continued Next Page

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 08-01

3 OF 3

METRIC

G.W.P. 2147-01-00 LOCATION CPR Overpass, N 4 841 668.1 E 302 389.0 ORIGINATED BY DEE  
HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2008.11.03 - 2008.11.03 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  Y  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
Continued From Previous Page							0 20 40 60 80 100					PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT W P W W L WATER CONTENT (%) 20 40 60					
	Silty CLAY, some sand to sandy, trace gravel Stiff to Very Stiff Grey (TILL)		13	TW	PH		124										
			14	SS	15		123										
			15	SS	20		122										
							121										
120.1							120										
24.4	Sandy SILT, some clay to clayey Compact Grey Wet		16	SS	20												
119.5																	
25.0	END OF BOREHOLE AT 25.0m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN ASPHALT TO SURFACE.																

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE

# RECORD OF BOREHOLE No 08-02

1 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION CPR Overpass, N 4 841 681.9 E 302 432.8 ORIGINATED BY WB  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2008.10.29 - 2008.10.30 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
144.5														
0.0	PAVEMENT: 50mm asphalt over 200mm concrete over 300mm gravel													
143.9														
0.6	Silty CLAY, trace to some sand Stiff to Very Stiff Brown (FILL)		1	SS	12		144							
			2	SS	12		143							
			3	SS	9		142							
			4	SS	7		141							
			5	SS	23		140							
			6	SS	24		139							
			7	SS	16		138							
							137							
							136							
134.9							135							
9.6	Silty CLAY, trace sand Stiff Brown													

Continued Next Page

+<sup>3</sup> . X<sup>3</sup> : Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

ORIGINATED BY WB

COMPILED BY AN

CHECKED BY MRA

Continued Next Page

(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 08-02

3 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION CPR Overpass, N 4 841 681.9 E 302 432.8 ORIGINATED BY WB  
HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2008.10.29 - 2008.10.30 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					WATER CONTENT (%) w <sub>p</sub> w w <sub>L</sub>				
	Continued From Previous Page		18	SS	22		124										
			19	SS	17		123									2	33 43 22
							122										
							121										
120.0							120										
24.5	SILT, trace sand, trace clay Compact Brown Moist		20	SS	30		119										
							118										
			21	SS	29		117									3	8 79 10
							116										
							115										

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 08-02

4 OF 4

METRIC

G.W.P. 2147-01-00 LOCATION CPR Overpass, N 4 841 681.9 E 302 432.8 ORIGINATED BY WB  
 HWY 401 Westbound Collectors BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2008.10.29 - 2008.10.30 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								<div><div></div><div>20406080100</div></div>										<div><div></div><div>204060</div></div>		
	Continued From Previous Page																			
	SILT, trace sand, trace clay Compact Brown Moist																			
113.7																				
30.8	Silty CLAY, some sand, trace gravel		22	SS	55															
113.4	Hard																			
31.1	Grey (TILL)																			
	END OF BOREHOLE AT 31.1m. WATER LEVEL AT 24.3m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN ASPHALT TO SURFACE.																			

+ 3 . x 3 : Numbers refer to  
Sensitivity

20  
15  
10


(%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity

## METRIC

ELEV DEPTH	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100						
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) 20 40 60					

[illegible]

+ 3, X 3: Numbers refer to Sensitivity

## **Appendix B**

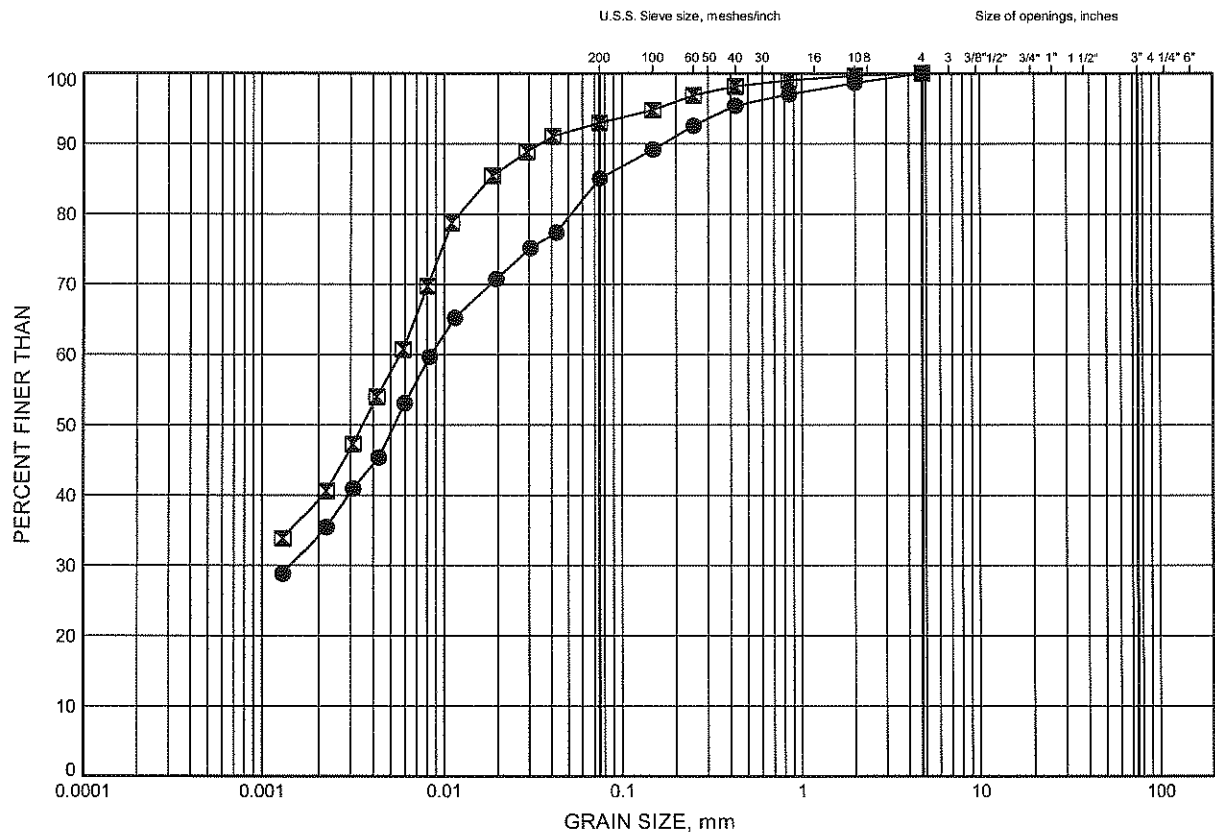
### **Laboratory Test Results**

# Hwy 401 Westbound Collectors, Jane to Kipling

## GRAIN SIZE DISTRIBUTION

FIGURE B1

### SILTY CLAY FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-01	6.40	138.10
×	08-02	4.88	139.62

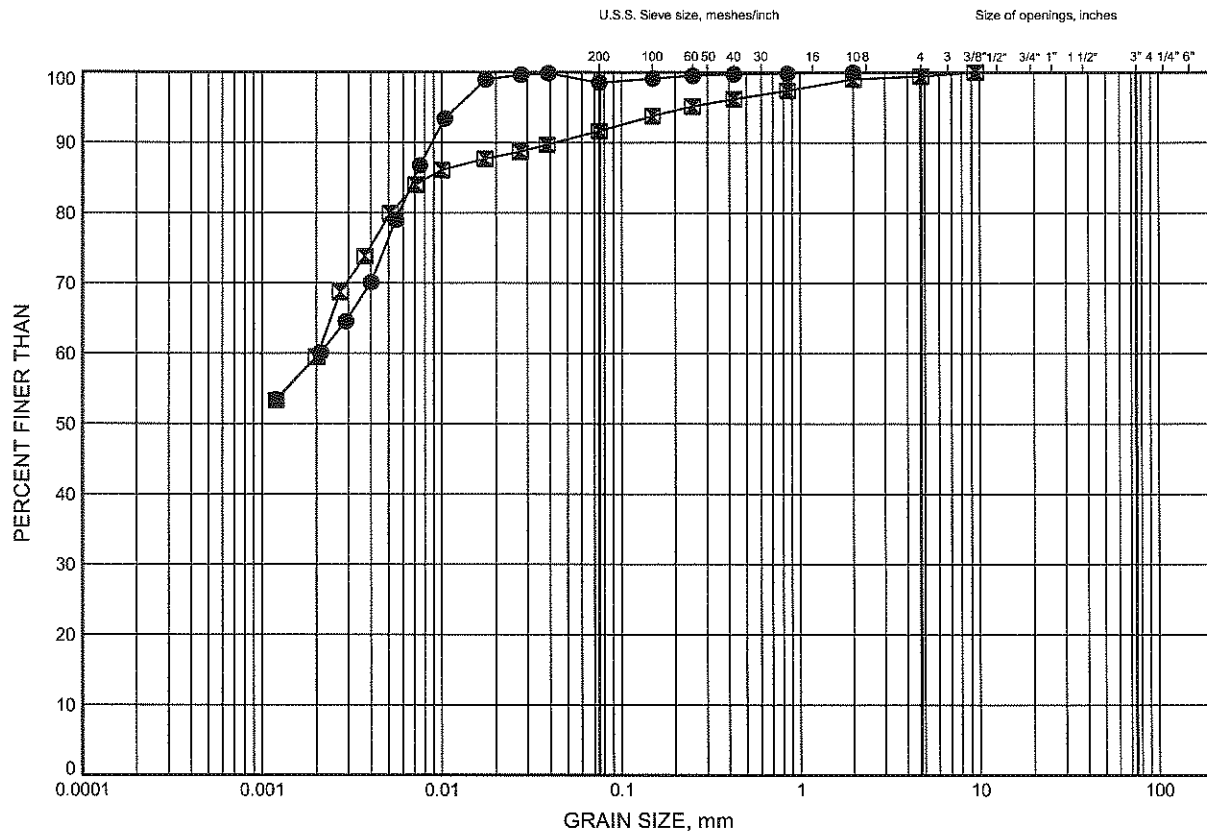


W.P.# .2147-01-00.....  
 Prepared By .AN.....  
 Checked By .MEF.....

Hwy 401 Westbound Collectors, Jane to Kipling  
**GRAIN SIZE DISTRIBUTION**

FIGURE B2

**SILTY CLAY**



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

**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-02	9.75	134.75
◻	RW-05	2.59	134.43



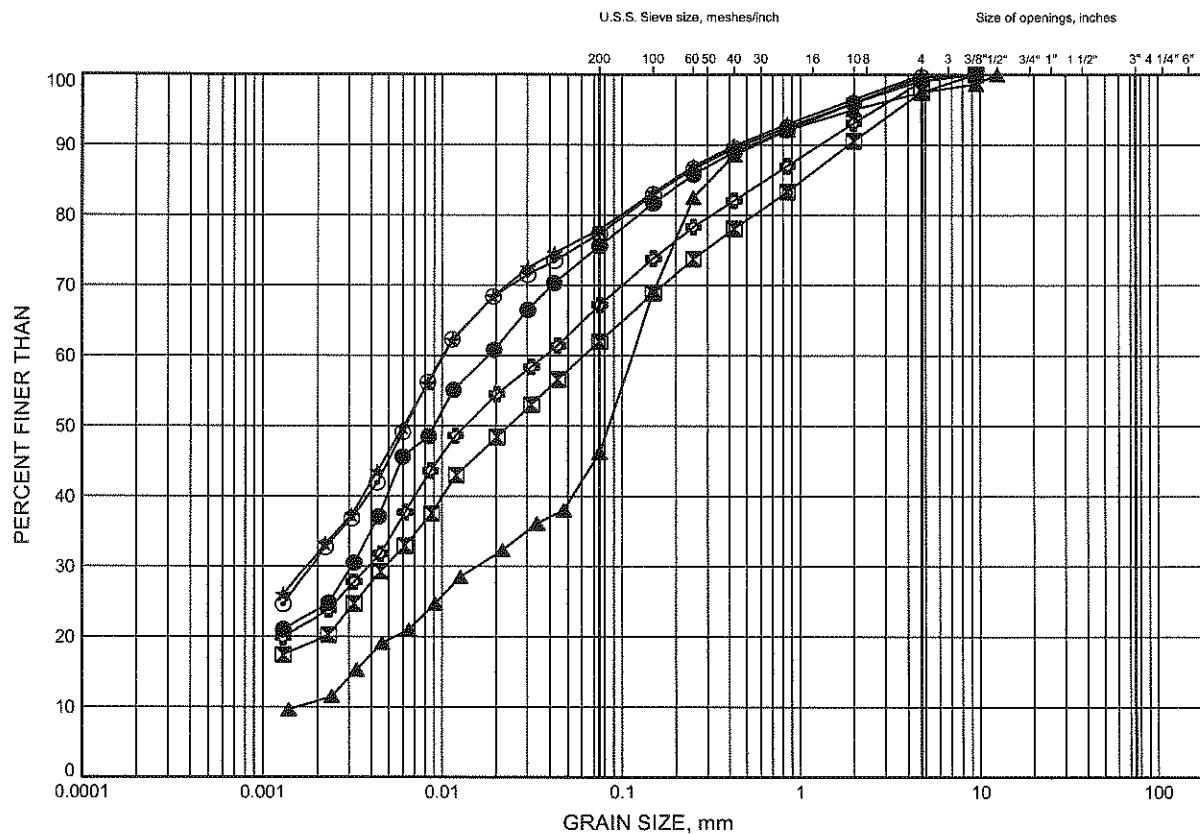
W.P.# .2147-01-00.....  
 Prepared By .AN.....  
 Checked By .MEF.....

# Hwy 401 Westbound Collectors, Jane to Kipling

## GRAIN SIZE DISTRIBUTION

FIGURE B3

### SILTY CLAY TILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	RW-05	7.92	129.10
⊠	RW-05	15.54	121.48
▲	RW-05	27.74	109.28
★	RW-06	2.59	133.00
⊙	RW-06	6.40	129.19
⊕	RW-06	15.54	120.05



W.P.# .2147-01-00.....  
 Prepared By .AN.....  
 Checked By .MEF.....

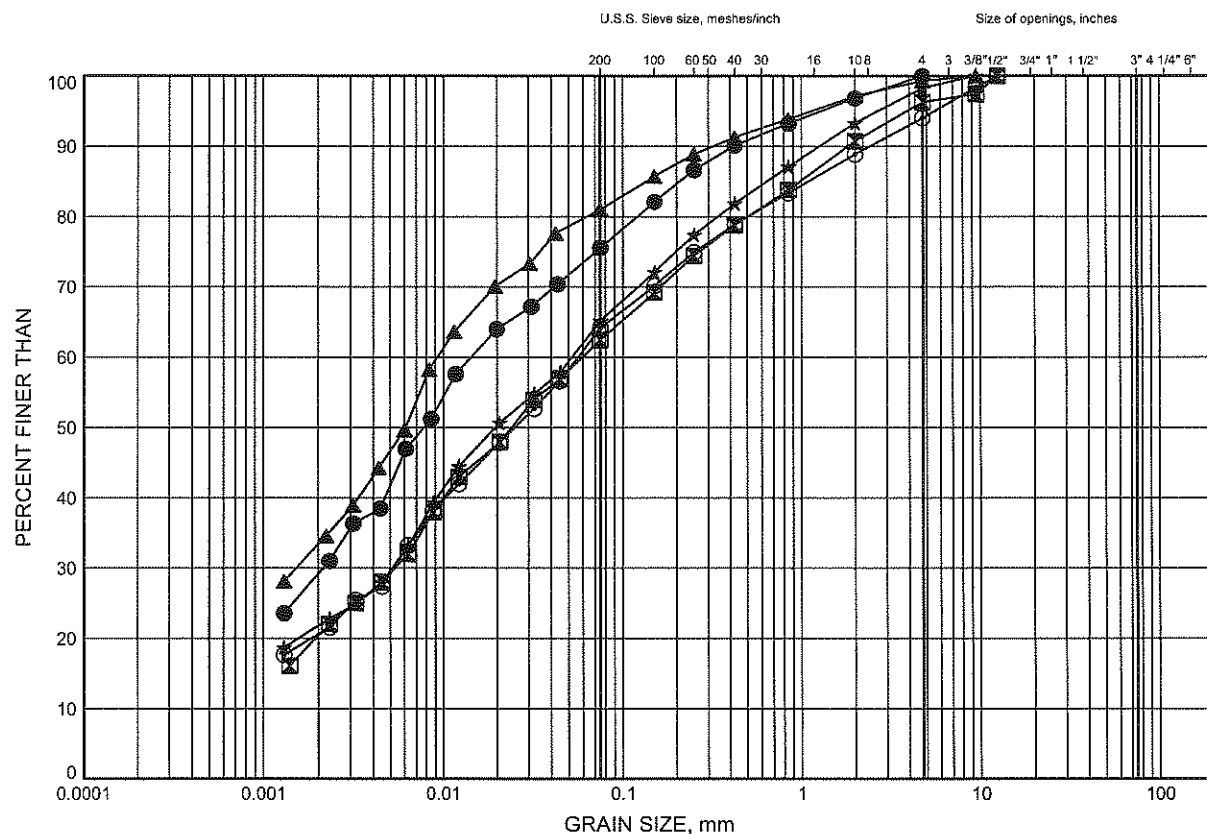


# Hwy 401 Westbound Collectors, Jane to Kipling

## GRAIN SIZE DISTRIBUTION

FIGURE B4

### SILTY CLAY TILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-01	12.50	132.00
⊠	08-01	18.59	125.91
▲	08-02	14.78	129.72
★	08-02	21.64	122.86
⊙	08-03	15.54	121.46

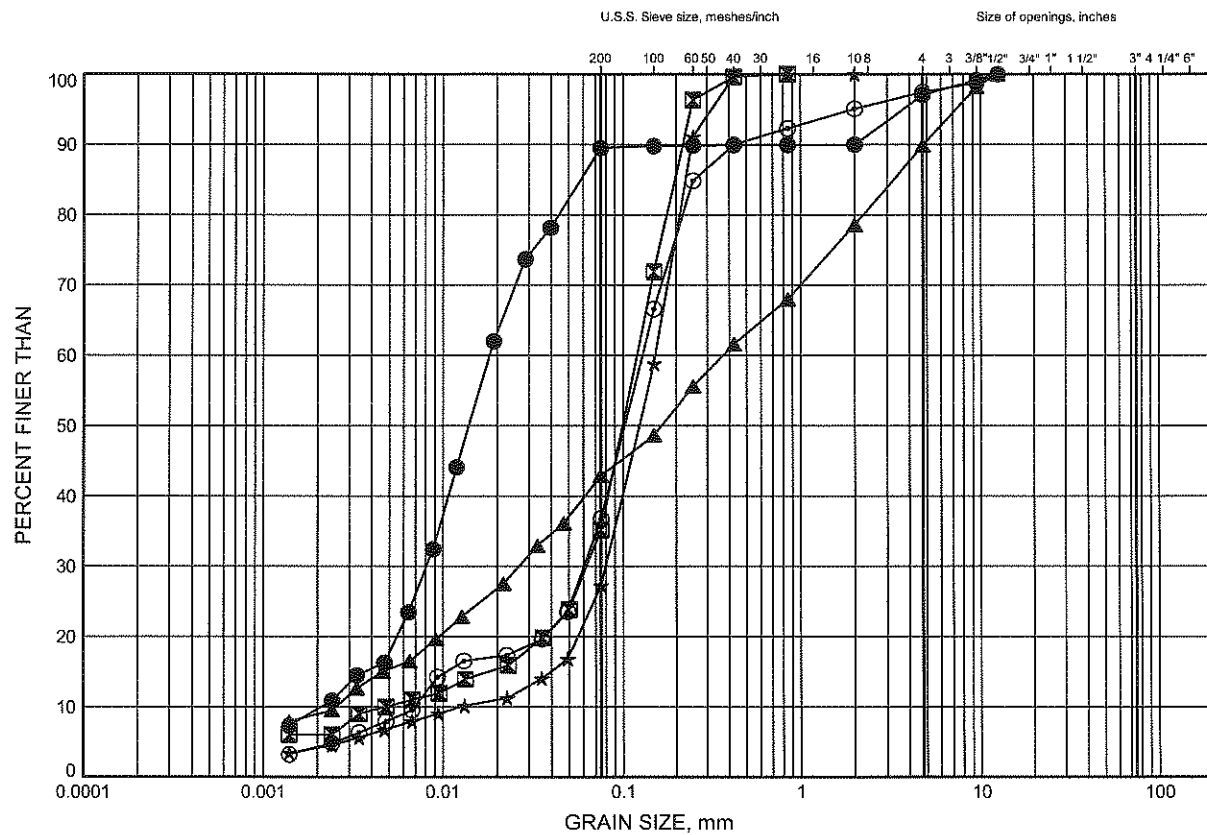


W.P.# .2147-01-00.....  
 Prepared By .MFA.....  
 Checked By .MRA.....

Hwy 401 Westbound Collectors, Jane to Kipling  
**GRAIN SIZE DISTRIBUTION**

FIGURE B5

**SILT, SILTY SAND AND SILTY SAND TILL**



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

**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-02	27.74	116.76
⊠	RW-05	20.12	116.90
▲	RW-05	33.83	103.19
★	RW-06	21.64	113.95
⊙	RW-06	30.78	104.81

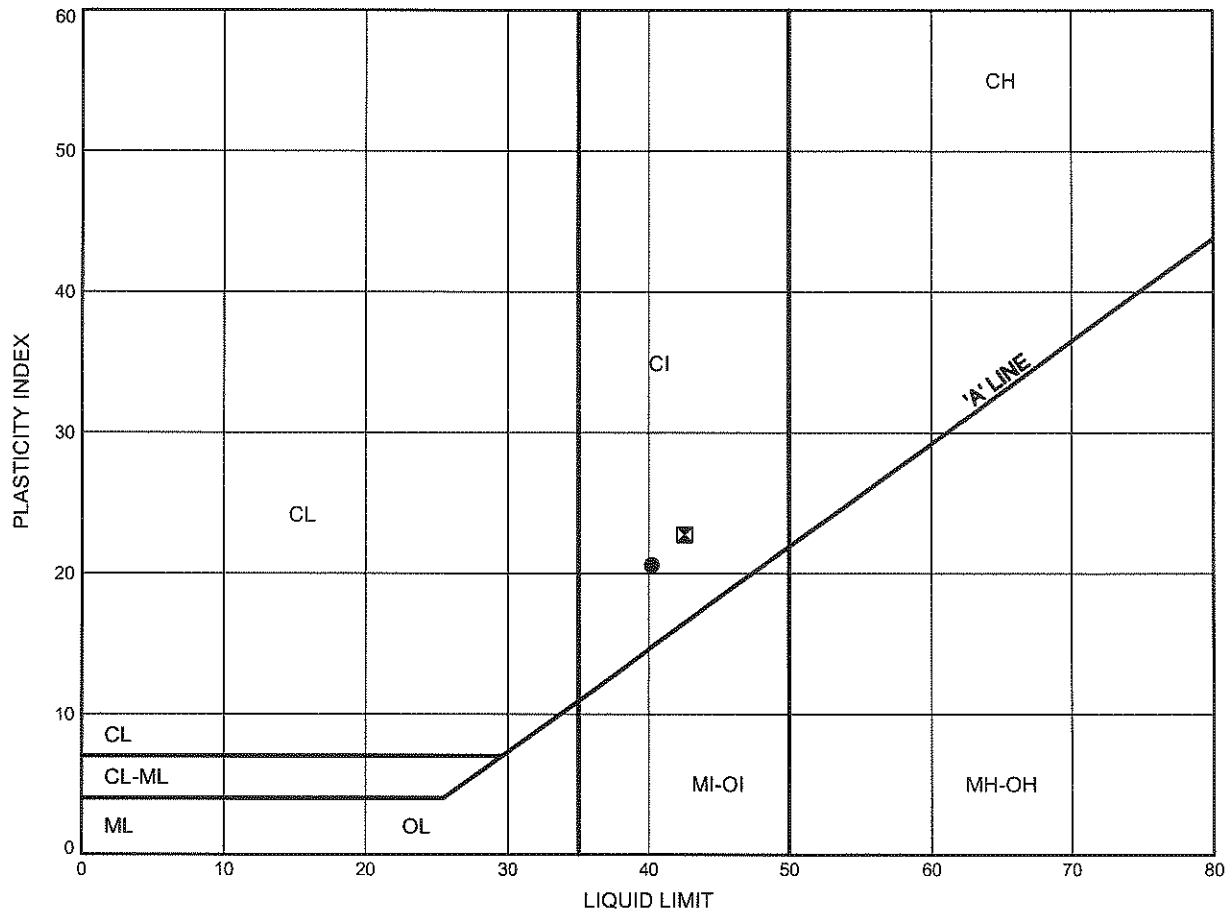


W.P.# .2147-01-00.....  
 Prepared By .MFA.....  
 Checked By .MRA.....

Hwy 401 Westbound Collectors, Jane to Kipling  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B6

**SILTY CLAY FILL**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-01	6.40	138.10
⊠	08-02	4.88	139.62



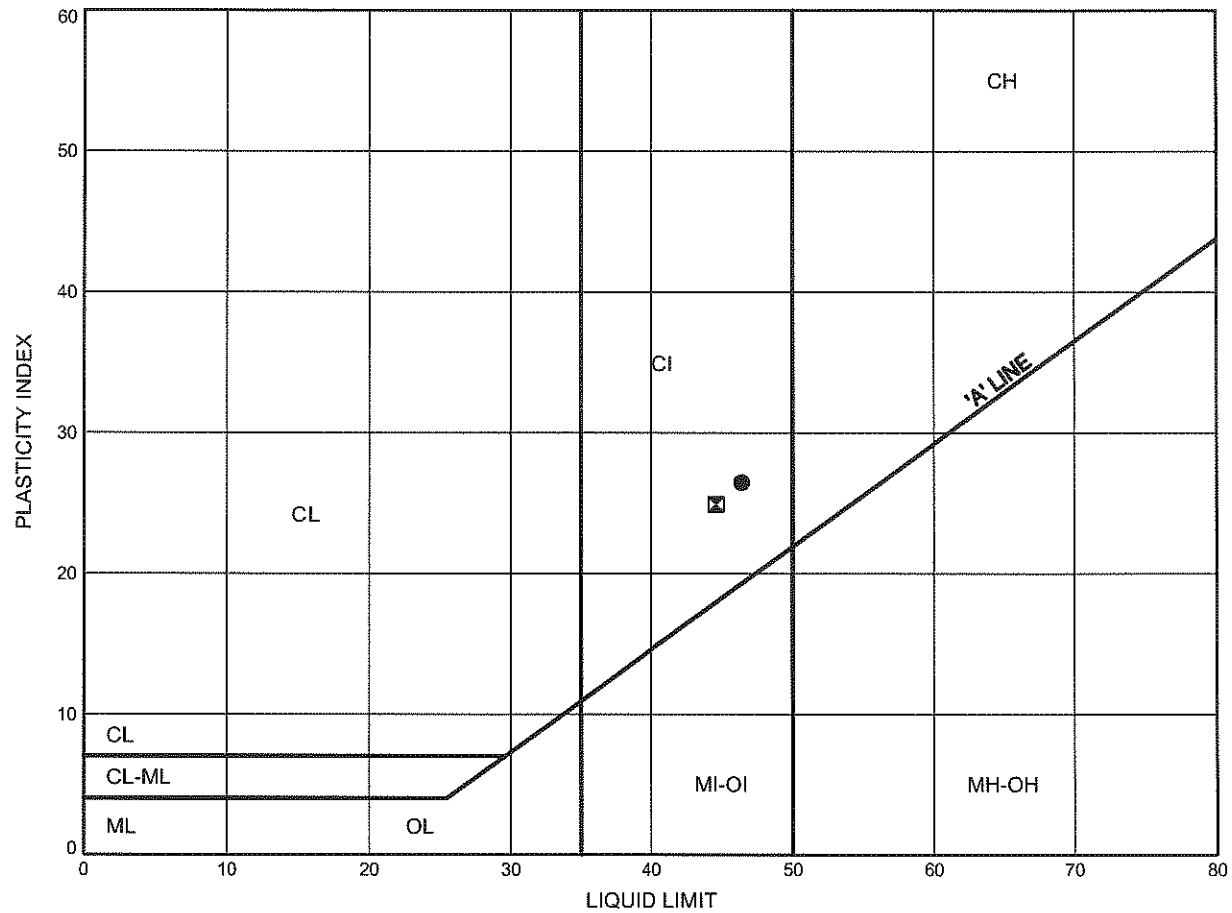
Date December 2008  
 Project 2147-01-00

Prep'd AN  
 Chkd. MEF

Hwy 401 Westbound Collectors, Jane to Kipling  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B7

**SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-02	9.75	134.75
⊠	RW-05	2.59	134.43

Date December 2008  
 Project 2147-01-00

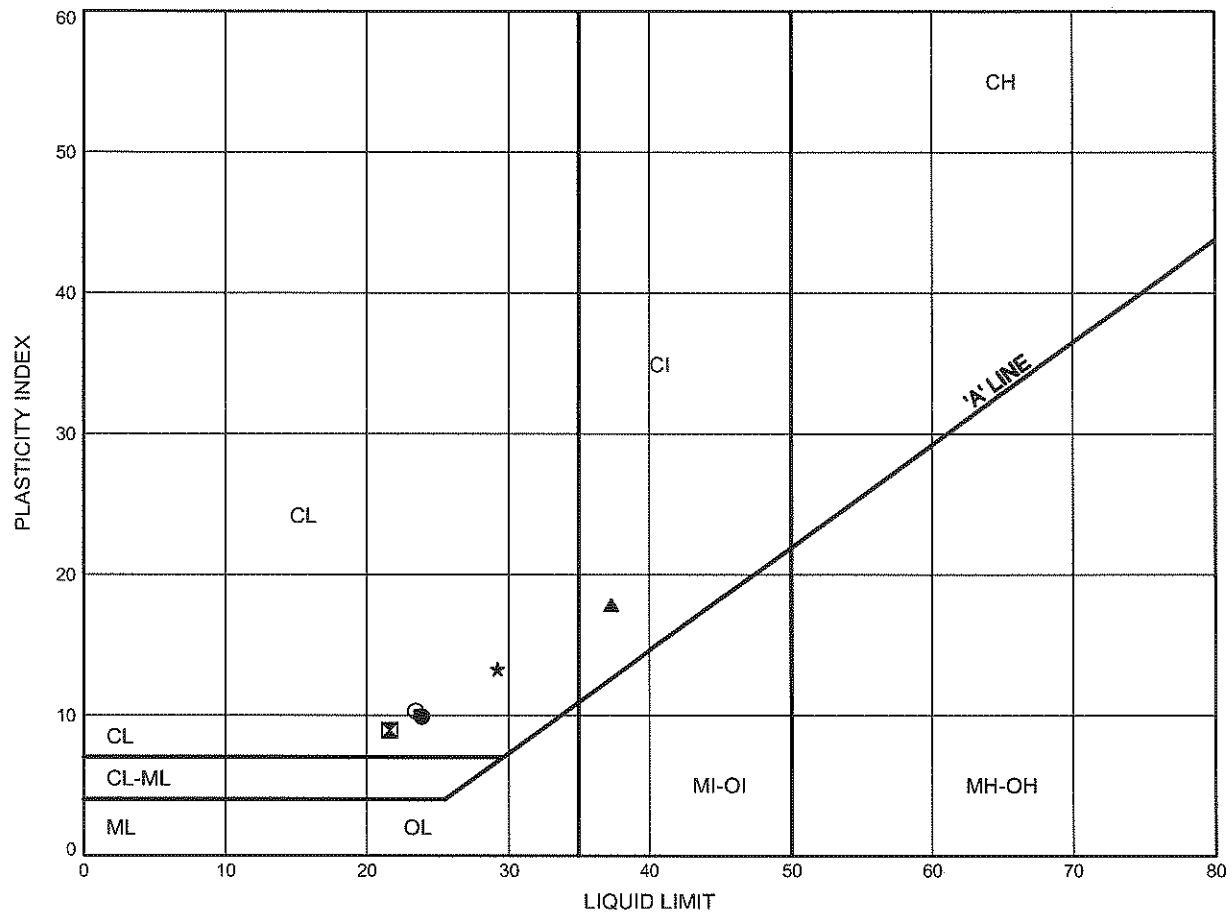


Prep'd AN  
 Chkd. MEF

Hwy 401 Westbound Collectors, Jane to Kipling  
**ATTERBERG LIMITS TEST RESULTS**

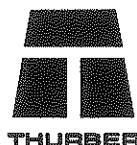
FIGURE B8

**SILTY CLAY TILL**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	RW-05	7.92	129.10
⊠	RW-05	15.54	121.48
▲	RW-06	2.59	133.00
★	RW-06	6.40	129.19
⊙	RW-06	15.54	120.05

Date December 2008  
 Project 2147-01-00

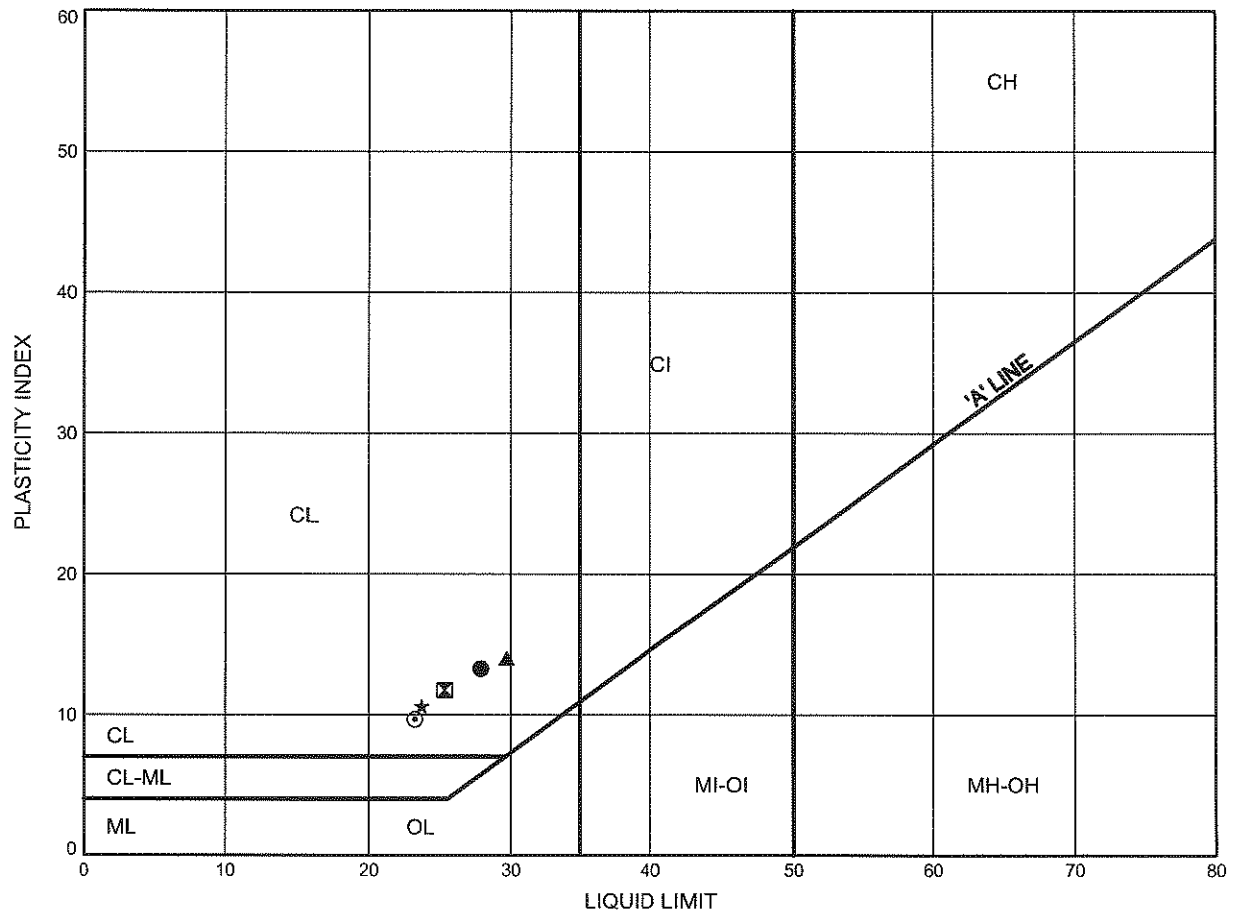


Prep'd AN  
 Chkd. MEF

Hwy 401 Westbound Collectors, Jane to Kipling  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B9

**SILTY CLAY TILL**

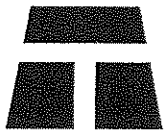


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-01	12.50	132.00
⊠	08-01	18.59	125.91
▲	08-02	14.78	129.72
★	08-02	21.64	122.86
⊙	08-03	15.54	121.46

Date December 2008  
 Project 2147-01-00



Prep'd AN  
 Chkd. MEF



## Consolidation Test Report

CLIENT: **McCormick Rankin Corporation**

FILE NUMBER: 19-1351-122

PROJECT: **HWY 401 Westbound Collectors, Jane to Kipling** REPORT DATE: 10-Dec-08

TEST DATES: November 12, 2008 - November 29, 2008

SAMPLE: BH08-3-TW4 (50'-52')  
Clay, Silty, Sandy, trace Gravel, grey, (CL), 21% Clay, 43% Silt, 30% Sand & 6% Gravel

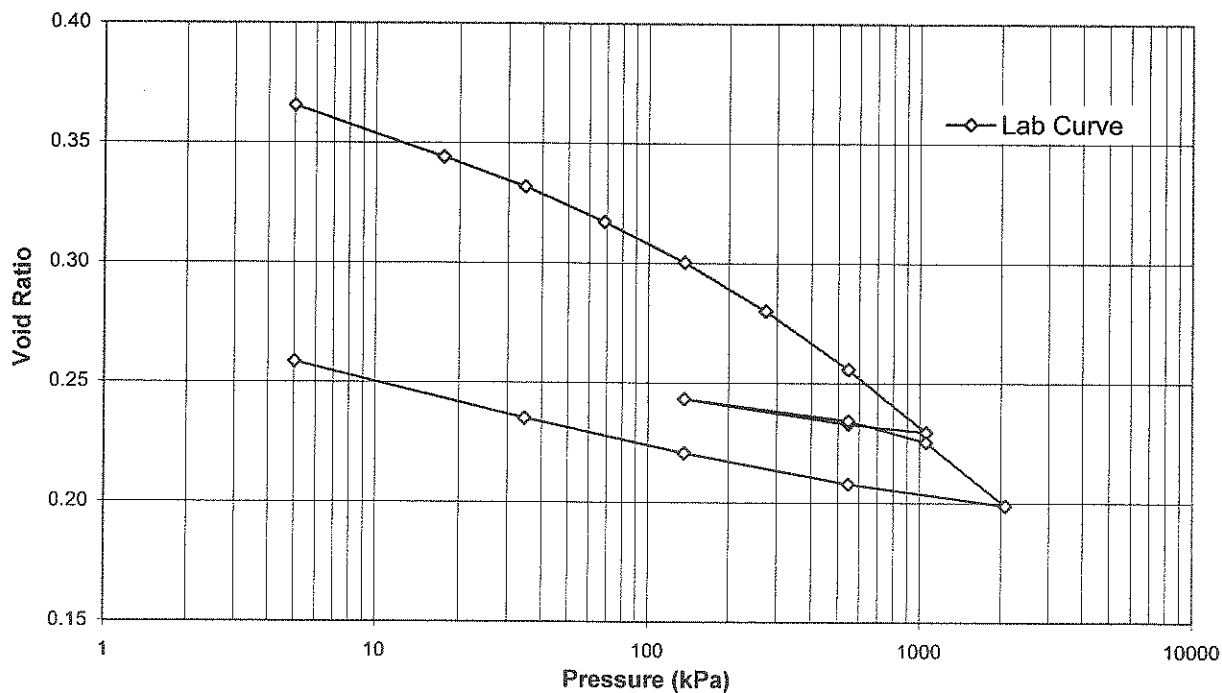
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

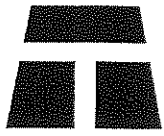
	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m <sup>3</sup> )	2246.3	2416.1
Dry Dens. (kg/m <sup>3</sup> )	1953.0	2142.7
Moisture Cont. (%)	14.1	12.8
Void Ratio	0.381	0.259
Saturation (%)	100.0	

Note: A Specific Gravity of 2.70 was measured for the void ratio and saturation calculations.

Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-3-TW4 (50'-52')  
Oedometer Consolidation Test

**Void Ratio vs Pressure**





## Consolidation Test Report

HWY 401 Westbound Collectors, Jane to Kipling  
19-1351-122

BH08-3-TW4 (50'-52')

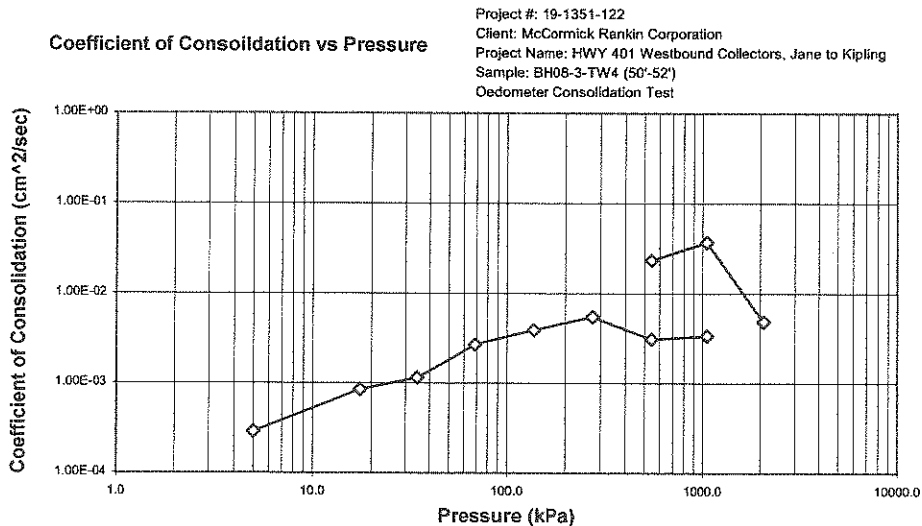
**TRIMMING:** The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

**LOADING:** A seating load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied and the duration of each load step was 24 hours.

**CALCULATIONS:** Coefficients of Consolidation were calculated by the square root time method.

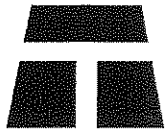
Pressure (kPa)	Corr. Hgt (mm)	Avg. Hgt. (mm)	D90 (mm)	T90 (min)	Cv (cm <sup>2</sup> /s)	Void Ratio	mv (m <sup>2</sup> /kN)	k (cm/s)
0.00	20.000					0.381		
5.02	19.778	19.889	0.196	48.34	2.89E-04	0.366	2.21E-03	6.27E-08
17.56	19.467	19.623	0.250	16.00	8.50E-04	0.344	1.25E-03	1.05E-07
34.53	19.288	19.378	0.134	11.56	1.15E-03	0.332	5.42E-04	6.10E-08
68.48	19.077	19.183	0.146	4.84	2.69E-03	0.317	3.22E-04	8.49E-08
136.85	18.833	18.955	0.160	3.24	3.92E-03	0.300	1.87E-04	7.19E-08
273.20	18.539	18.686	0.180	2.25	5.48E-03	0.280	1.14E-04	6.16E-08
545.45	18.187	18.363	0.236	3.84	3.10E-03	0.256	6.97E-05	2.12E-08
1057.67	17.807	17.997	0.245	3.39	3.38E-03	0.230	4.08E-05	1.35E-08
545.50	17.855	17.831				0.233		
136.90	18.007	17.931				0.243		
545.50	17.877	17.942	0.101	0.49	2.32E-02	0.234	1.77E-05	4.02E-08
1057.70	17.748	17.813	0.068	0.30	3.71E-02	0.225	1.41E-05	5.12E-08
2080.10	17.364	17.556	0.230	2.25	4.84E-03	0.199	2.12E-05	1.00E-08
545.45	17.494	17.429				0.208		
136.85	17.680	17.587				0.221		
34.53	17.889	17.785				0.235		
5.02	18.229	18.059				0.259		

Coefficient of Consolidation vs Pressure



Notes: Cv and k calculated using  $t_{90}$  values





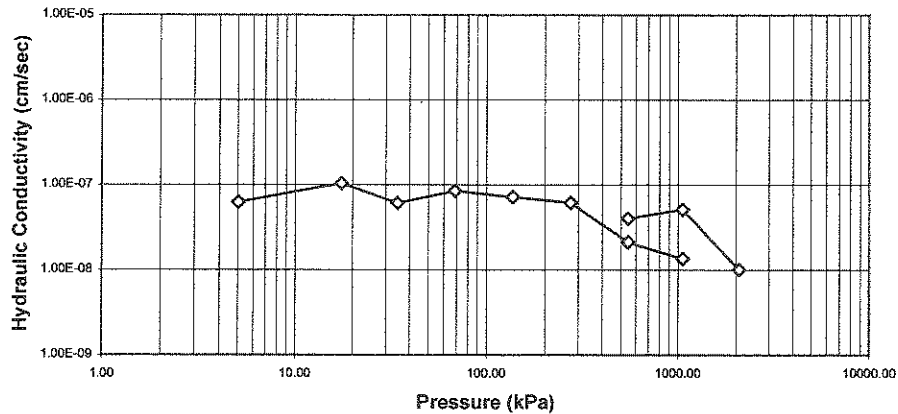
## Consolidation Test Report

HWY 401 Westbound Collectors, Jane to Kipling  
19-1351-122

BH08-3-TW4 (50'-52')

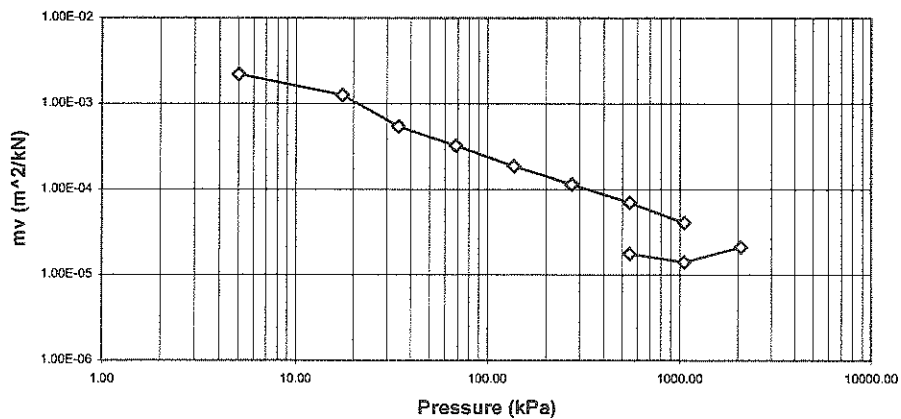
### Hydraulic Conductivity vs Pressure

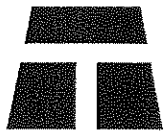
Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-3-TW4 (50'-52')  
Oedometer Consolidation Test



### mv vs Pressure

Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-3-TW4 (50'-52')  
Oedometer Consolidation Test





## Consolidation Test Report

CLIENT: **McCormick Rankin Corporation**

FILE NUMBER: 19-1351-122

PROJECT: HWY 401 Westbound Collectors, Jane to Kipling

REPORT DATE: 10-Dec-08

TEST DATES: November 12, 2008 - November 29, 2008

SAMPLE: BH08-4-TW11 (30'-32')

Clay, Silty, Sandy, trace Gravel, grey, (CL), 32% Clay, 43% Silt, 23% Sand & 2% Gravel

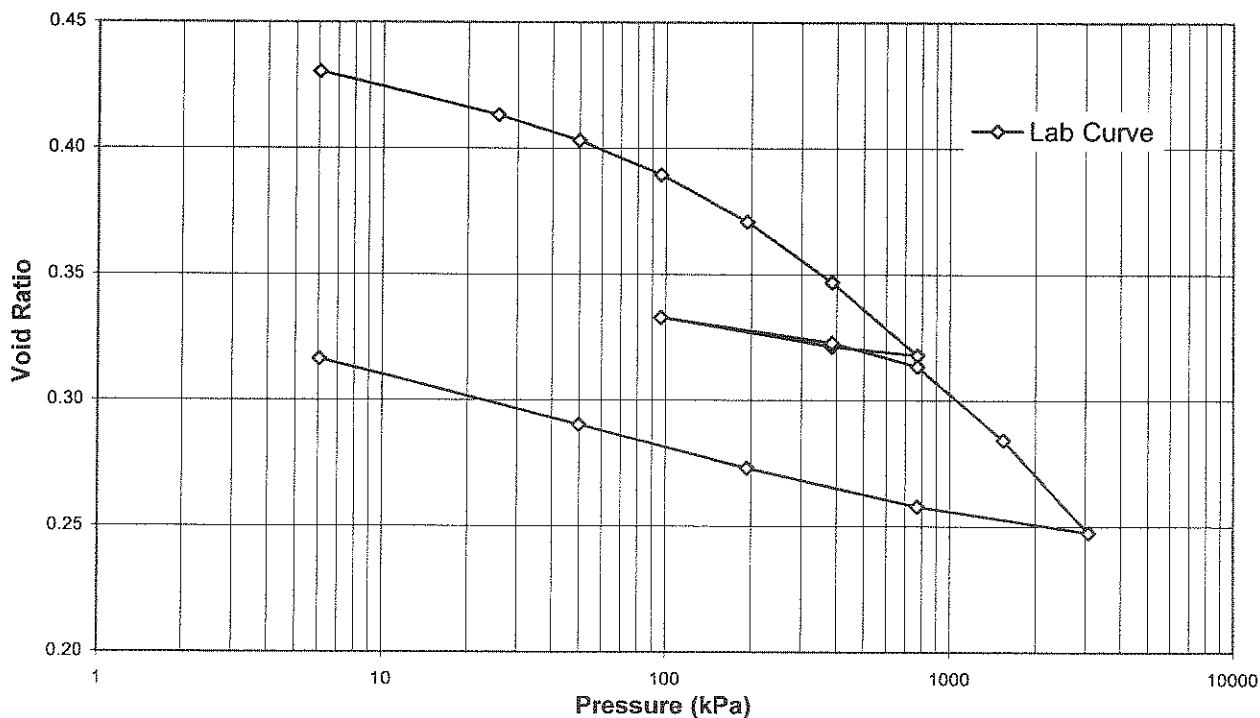
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

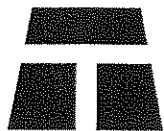
	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m <sup>3</sup> )	2195.1	2347.4
Dry Dens. (kg/m <sup>3</sup> )	1883.9	2058.5
Moisture Cont. (%)	16.2	14.0
Void Ratio	0.438	0.316
Saturation (%)	100.0	

Note: A Specific Gravity of 2.71 was measured for the void ratio and saturation calculations.

Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-4-TW11 (30'-32')  
Oedometer Consolidation Test

**Void Ratio vs Pressure**





## Consolidation Test Report

HWY 401 Westbound Collectors, Jane to Kipling  
19-1351-122

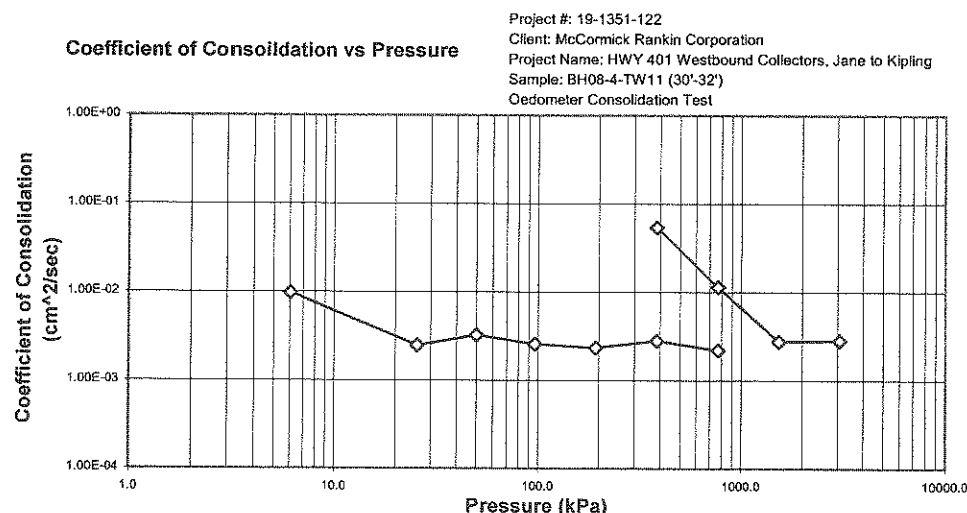
BH08-4-TW11 (30'-32')

**TRIMMING:** The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

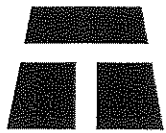
**LOADING:** A seating load of 6 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied and the duration of each load step was 24 hours.

**CALCULATIONS:** Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. Hgt (mm)	Avg. Hgt. (mm)	D90 (mm)	T90 (min)	Cv (cm <sup>2</sup> /s)	Void Ratio	mv (m <sup>2</sup> /kN)	k (cm/s)
0.00	25.500					0.438		
6.07	25.357	25.429	0.095	2.32	9.84E-03	0.430	9.24E-04	8.92E-07
25.67	25.052	25.205	0.246	9.00	2.49E-03	0.413	6.14E-04	1.50E-07
49.86	24.874	24.963	0.115	6.76	3.26E-03	0.403	2.94E-04	9.38E-08
96.64	24.633	24.754	0.166	8.41	2.57E-03	0.390	2.07E-04	5.23E-08
193.23	24.303	24.468	0.229	9.00	2.35E-03	0.371	1.39E-04	3.20E-08
385.74	23.876	24.090	0.275	7.29	2.81E-03	0.347	9.13E-05	2.52E-08
770.65	23.363	23.620	0.341	9.00	2.19E-03	0.318	5.58E-05	1.20E-08
385.70	23.424	23.394				0.321		
96.60	23.628	23.526				0.333		
385.70	23.451	23.540	0.116	0.36	5.44E-02	0.323	2.59E-05	1.38E-07
770.60	23.285	23.368	0.099	1.69	1.14E-02	0.314	1.84E-05	2.06E-08
1540.66	22.766	23.026	0.333	6.76	2.77E-03	0.284	2.89E-05	7.87E-09
3081.39	22.116	22.441	0.440	6.25	2.85E-03	0.248	1.85E-05	5.17E-09
770.65	22.299	22.208				0.258		
193.23	22.570	22.435				0.273		
49.86	22.875	22.723				0.290		
6.07	23.337	23.106				0.316		



Notes: Cv and k calculated using  $t_{90}$  values



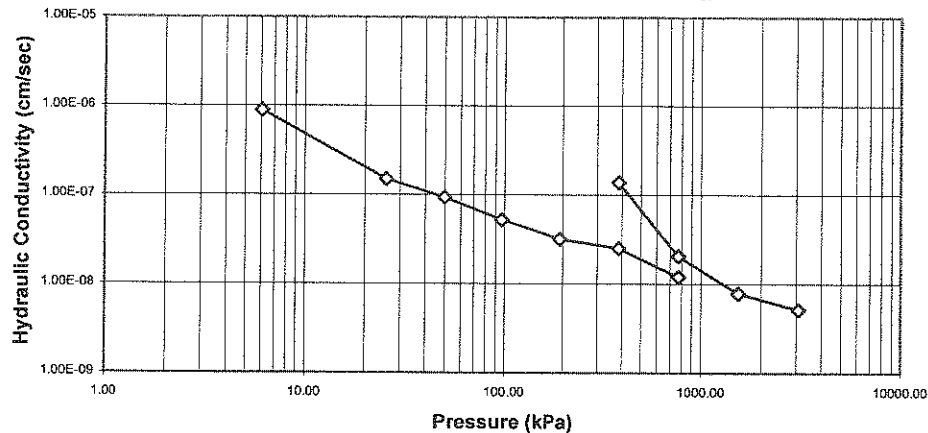
## Consolidation Test Report

HWY 401 Westbound Collectors, Jane to Kipling  
19-1351-122

BH08-4-TW11 (30'-32')

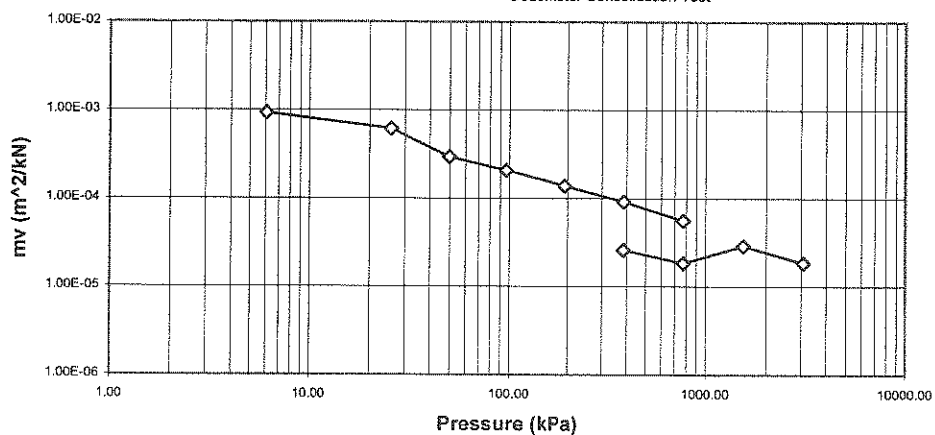
**Hydraulic Conductivity vs Pressure**

Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-4-TW11 (30'-32')  
Oedometer Consolidation Test



**mv vs Pressure**

Project #: 19-1351-122  
Client: McCormick Rankin Corporation  
Project Name: HWY 401 Westbound Collectors, Jane to Kipling  
Sample: BH08-4-TW11 (30'-32')  
Oedometer Consolidation Test



# OEDOMETER CONSOLIDATION SUMMARY

## SAMPLE IDENTIFICATION

Project Number	08-1116-0032	Sample Number	TW 10
Borehole Number	08-1	Sample Depth, m	15.2-15.8

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	11/17/2008		
Date Completed	12/11/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	21.67
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	18.77
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	80.16	Solids Height, cm	1.761
Water Content, %	15.44	Volume of Solids, cm <sup>3</sup>	55.59
Wet Mass, g	177.13	Volume of Voids, cm <sup>3</sup>	24.56
Dry Mass, g	153.44	Degree of Saturation, %	96.5

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv, cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	2.539	0.442	2.539				
4.87	2.533	0.438	2.536	6	2.27E-01	4.93E-04	1.10E-05
9.56	2.527	0.435	2.530	46	2.95E-02	5.04E-04	1.46E-06
19.31	2.518	0.430	2.522	34	3.97E-02	3.64E-04	1.41E-06
38.81	2.505	0.422	2.511	52	2.57E-02	2.63E-04	6.62E-07
77.67	2.487	0.412	2.496	46	2.87E-02	1.83E-04	5.16E-07
155.27	2.464	0.399	2.475	13	9.99E-02	1.17E-04	1.15E-06
310.85	2.433	0.381	2.448	38	3.34E-02	7.82E-05	2.56E-07
621.55	2.391	0.358	2.412	7	1.76E-01	5.34E-05	9.21E-07
155.30	2.403	0.364	2.397				
39.01	2.420	0.374	2.411				
9.64	2.436	0.383	2.428				
19.47	2.432	0.381	2.434	11	1.14E-01	1.56E-04	1.75E-06
38.80	2.428	0.379	2.430	12	1.04E-01	8.15E-05	8.33E-07
77.62	2.421	0.375	2.424	12	1.04E-01	7.20E-05	7.33E-07
155.30	2.412	0.370	2.416	41	3.02E-02	4.61E-05	1.36E-07
310.63	2.401	0.363	2.406	86	1.43E-02	2.76E-05	3.87E-08
620.89	2.383	0.353	2.392	22	5.51E-02	2.30E-05	1.24E-07
1240.67	2.341	0.329	2.362	59	2.00E-02	2.66E-05	5.23E-08
2481.32	2.286	0.298	2.313	15	7.56E-02	1.75E-05	1.29E-07
1240.67	2.290	0.300	2.288				
310.85	2.309	0.311	2.299				
77.67	2.331	0.324	2.320				
19.31	2.358	0.339	2.344				
4.87	2.373	0.348	2.366				

Note:

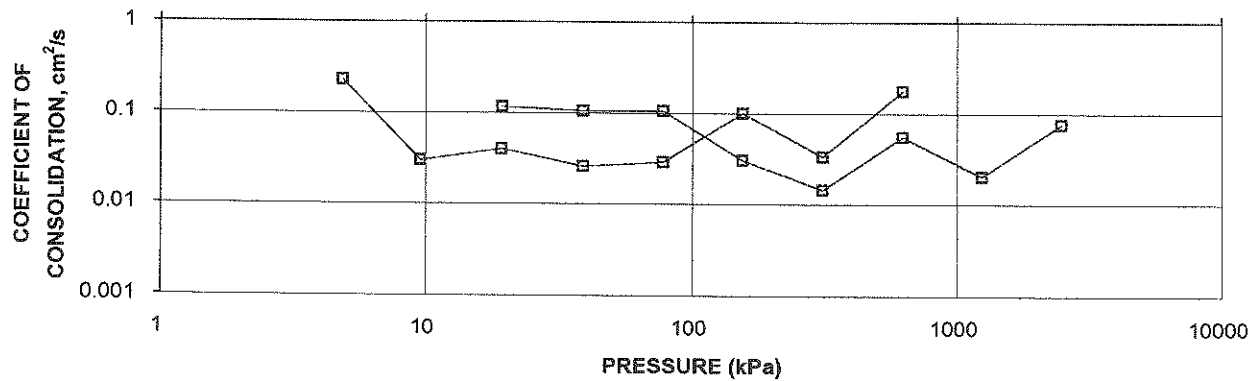
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

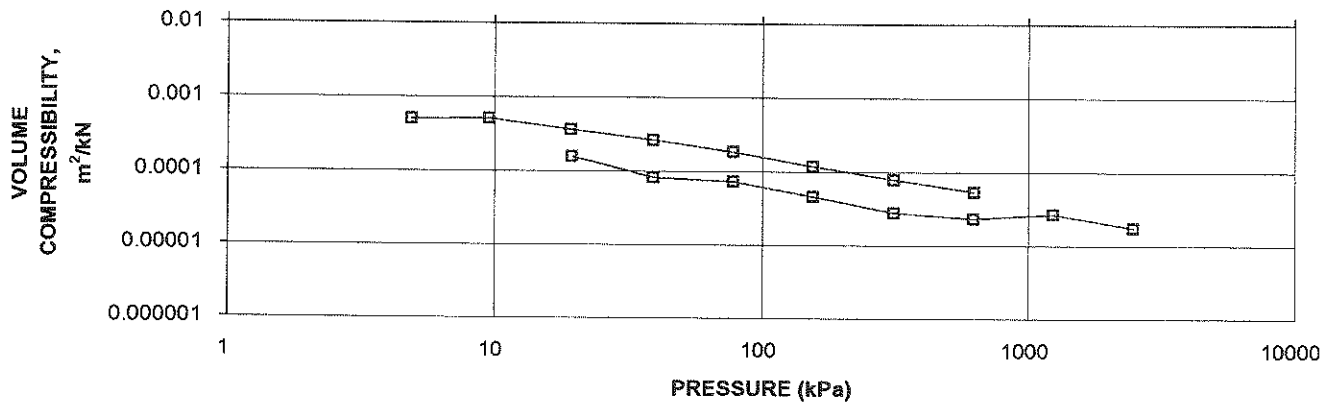
Sample Height, cm	2.37	Unit Weight, kN/m <sup>3</sup>	22.80
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	20.09
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	74.91	Solids Height, cm	1.761
Water Content, %	13.50	Volume of Solids, cm <sup>3</sup>	55.59
Wet Mass, g	174.16	Volume of Voids, cm <sup>3</sup>	19.32
Dry Mass, g	153.44		

# OEDOMETER CONSOLIDATION SUMMARY

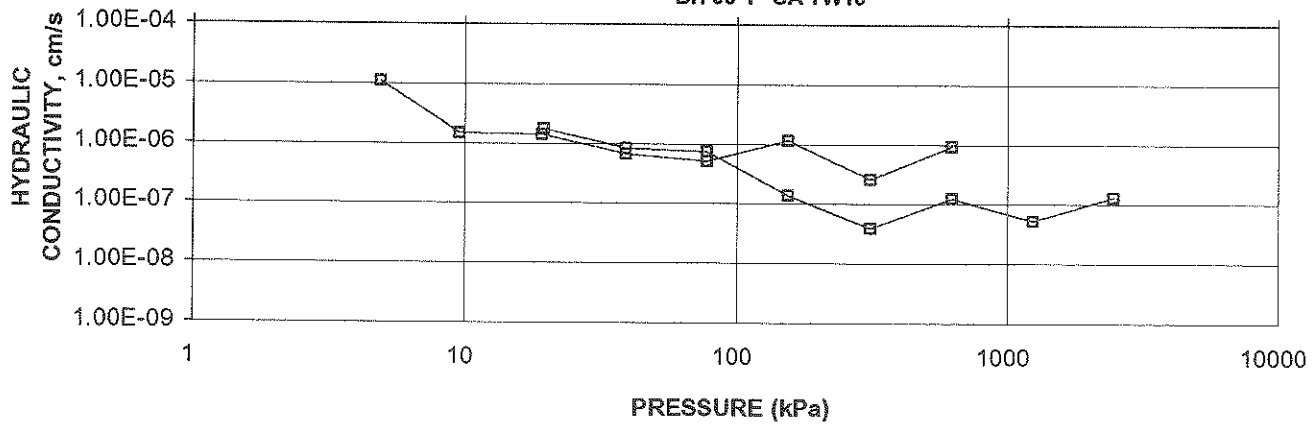
CONSOLIDATION TEST  
CV  $\text{cm}^2/\text{s}$  VS PRESSURE (kPa)  
BH 08-1 SA TW10



CONSOLIDATION TEST  
MV  $\text{m}^2/\text{kN}$  vs PRESSURE (kPa)  
BH 08-1 SA TW10



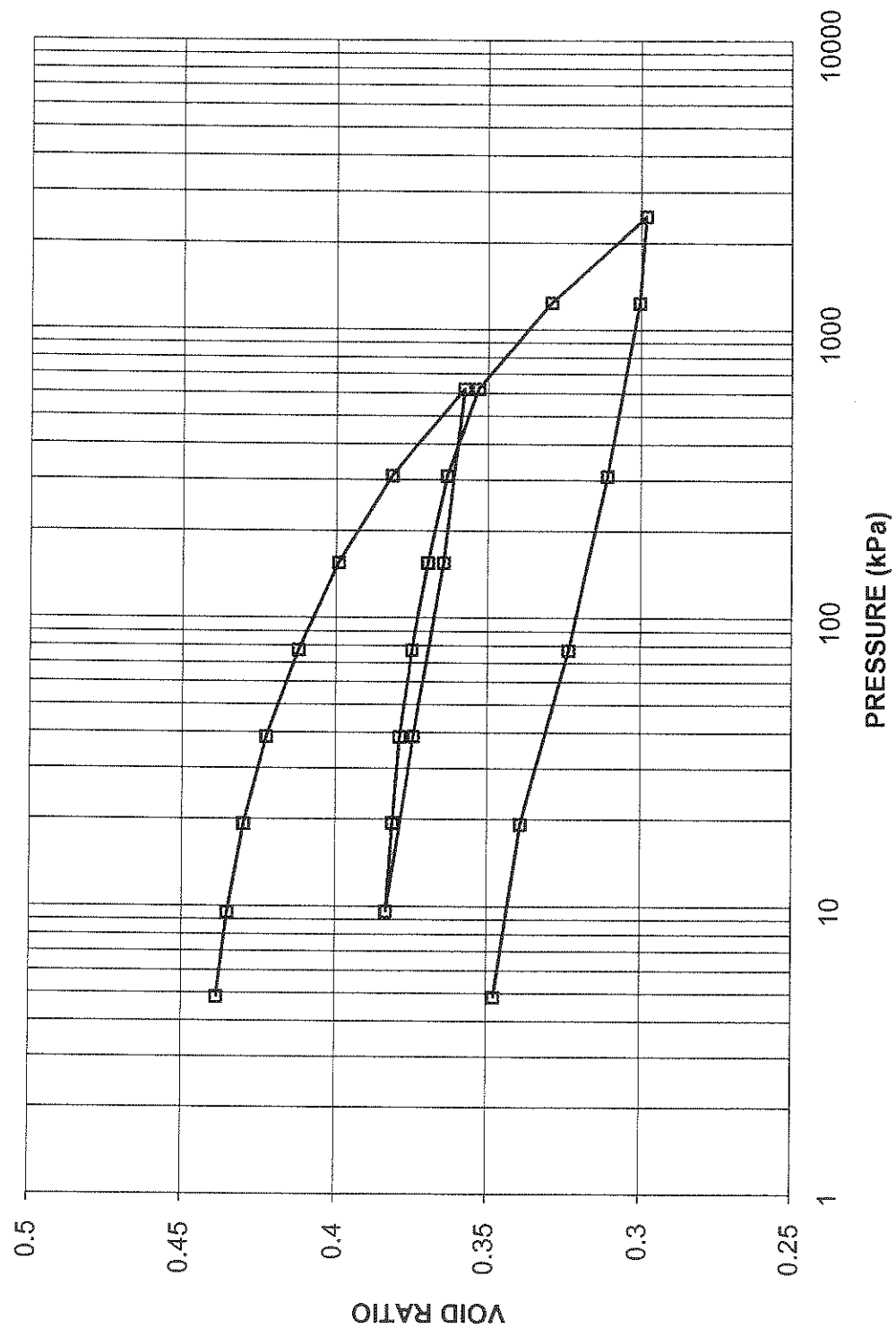
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 08-1 SA TW10



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH 08-1 SA TW10



# OEDOMETER CONSOLIDATION SUMMARY

## SAMPLE IDENTIFICATION

Project Number	08-1116-0032	Sample Number	TW2
Borehole Number	08-3	Sample Depth, m	6.1-6.7

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	11/18/2008		
Date Completed	12/09/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	20.97
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	18.06
Area, cm <sup>2</sup>	31.61	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	80.38	Solids Height, cm	1.697
Water Content, %	16.13	Volume of Solids, cm <sup>3</sup>	53.63
Wet Mass, g	171.89	Volume of Voids, cm <sup>3</sup>	26.76
Dry Mass, g	148.01	Degree of Saturation, %	89.3

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv, cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	2.543	0.499	2.543				
4.76	2.543	0.499	2.543	8	1.71E-01	8.26E-06	1.39E-07
9.47	2.539	0.496	2.541	2	6.84E-01	3.59E-04	2.41E-05
19.02	2.532	0.492	2.535	15	9.08E-02	2.84E-04	2.53E-06
38.71	2.516	0.483	2.524	118	1.14E-02	3.22E-04	3.61E-07
77.53	2.495	0.471	2.505	47	2.83E-02	2.11E-04	5.85E-07
155.07	2.466	0.454	2.480	10	1.30E-01	1.46E-04	1.87E-06
310.55	2.427	0.430	2.446	12	1.06E-01	9.91E-05	1.03E-06
620.03	2.381	0.403	2.404	14	8.75E-02	5.88E-05	5.04E-07
154.98	2.397	0.413	2.389				
38.57	2.420	0.426	2.408				
9.58	2.442	0.439	2.431				
19.53	2.436	0.436	2.439	12	1.05E-01	2.53E-04	2.60E-06
38.76	2.431	0.433	2.433	96	1.31E-02	1.04E-04	1.34E-07
77.66	2.420	0.426	2.425	34	3.67E-02	1.09E-04	3.92E-07
155.07	2.407	0.419	2.413	5	2.47E-01	6.50E-05	1.57E-06
309.76	2.390	0.409	2.398	20	6.10E-02	4.37E-05	2.61E-07
617.89	2.371	0.397	2.380	23	5.22E-02	2.46E-05	1.26E-07
1237.08	2.322	0.369	2.346	37	3.15E-02	3.08E-05	9.52E-08
2476.39	2.255	0.329	2.288	43	2.58E-02	2.14E-05	5.42E-08
1237.08	2.258	0.331	2.256				
309.76	2.287	0.348	2.273				
77.53	2.314	0.364	2.300				
19.02	2.348	0.384	2.331				
4.76	2.372	0.398	2.360				

Note:  
k calculated using cv based on t<sub>90</sub> values.

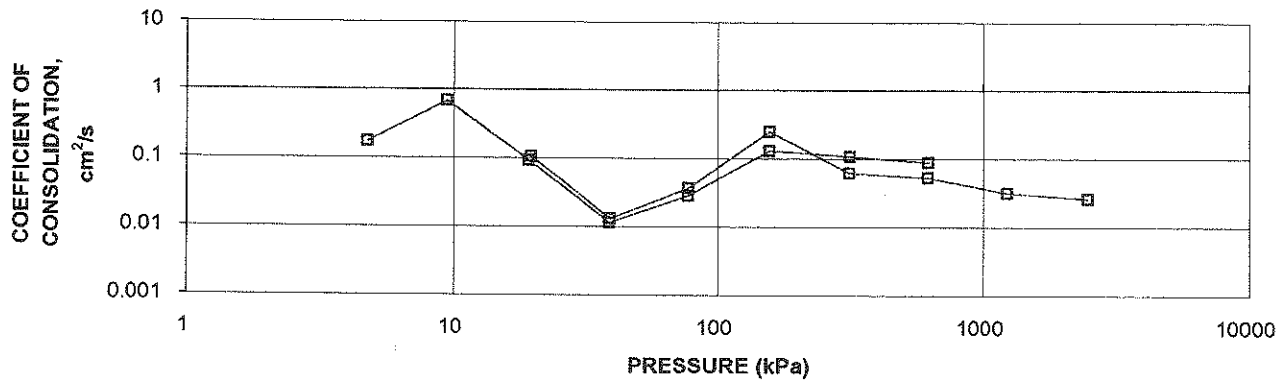
## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.37	Unit Weight, kN/m <sup>3</sup>	22.24
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	19.36
Area, cm <sup>2</sup>	31.61	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	74.99	Solids Height, cm	1.697
Water Content, %	14.90	Volume of Solids, cm <sup>3</sup>	53.63
Wet Mass, g	170.06	Volume of Voids, cm <sup>3</sup>	21.36
Dry Mass, g	148.01		

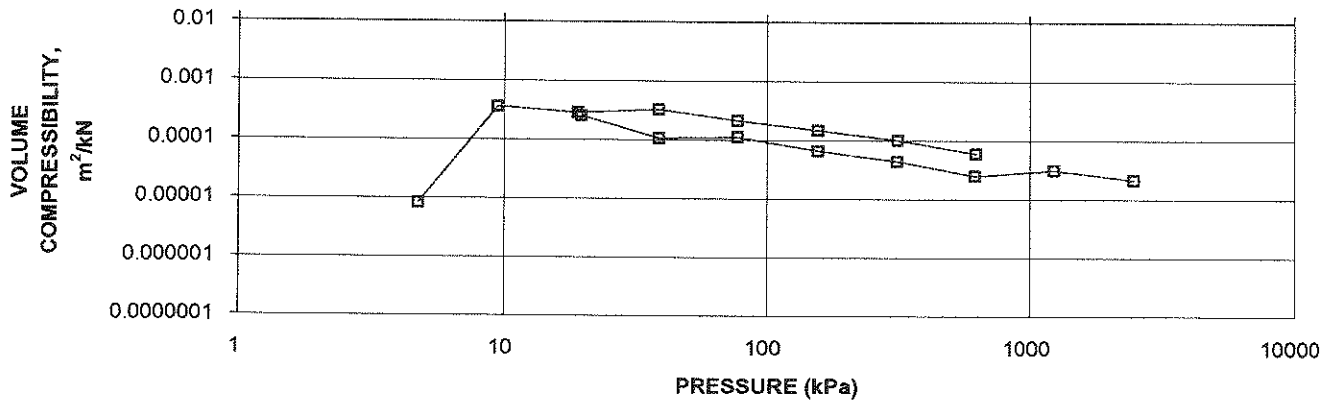


# OEDOMETER CONSOLIDATION SUMMARY

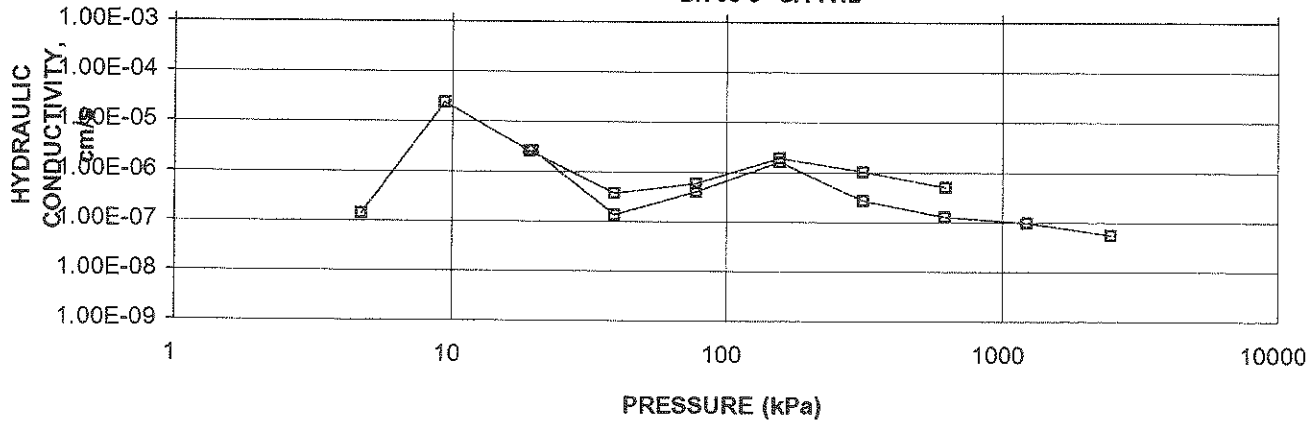
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH 08-3 SA TW2



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH 08-3 SA TW2



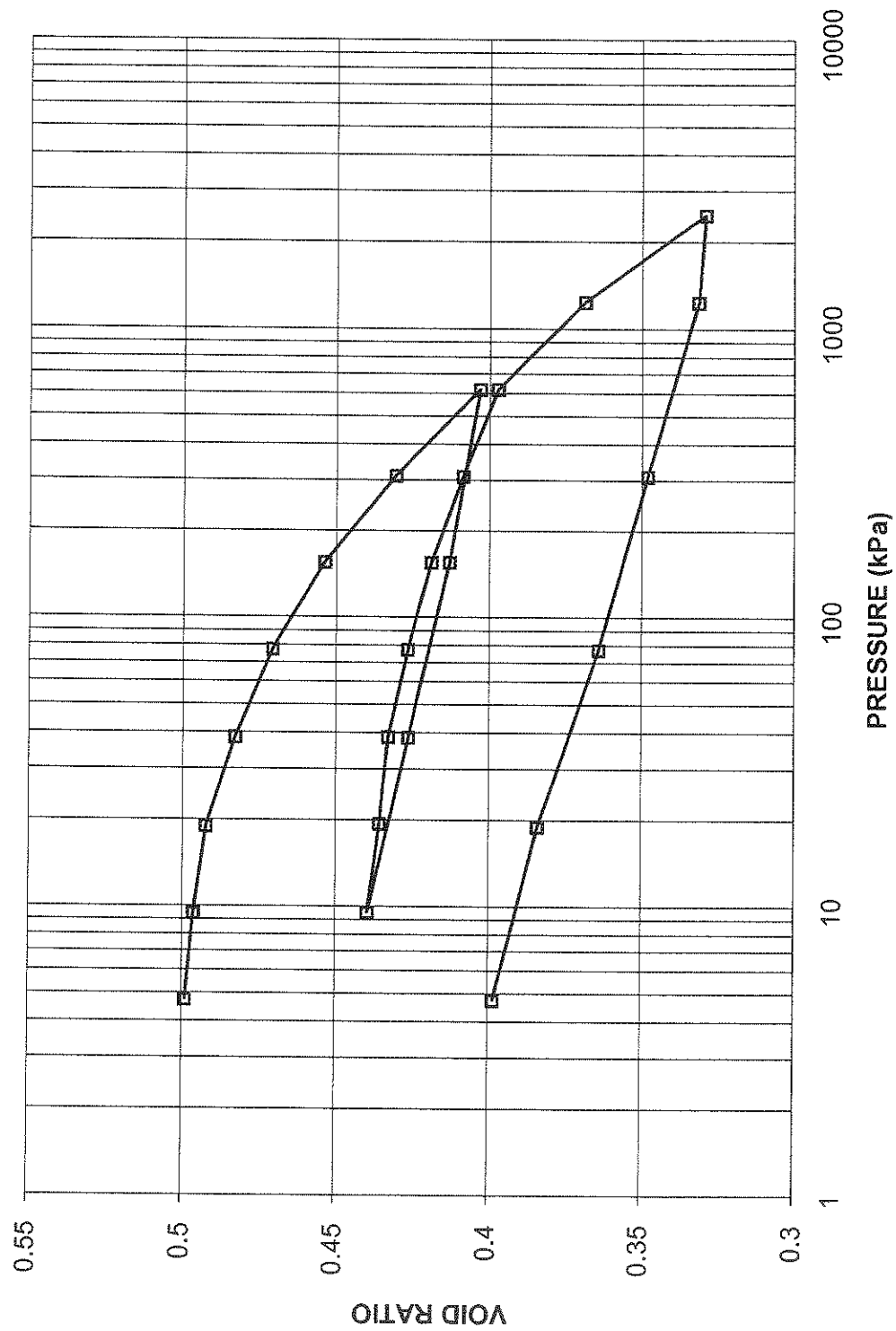
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 08-3 SA TW2



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE

CONSOLIDATION TEST  
VOID RATIO vs. PRESSURE  
BH 08-3 SA TW2



*hly*

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 1 OF 4**

**FIGURE**

TEST STAGE	A	B	C
BOREHOLE NUMBER	08-1		
SAMPLE	TW-10		
SPECIMEN DIAMETER, cm	5.02		
SPECIMEN HEIGHT, cm	10.15		
WATER CONTENT BEFORE CONSOLIDATION, %	17.0		
CELL PRESSURE, $\sigma_3$ , kPa	505.0		
BACK PRESSURE, kPa	205.0		
PORE PRESSURE PARAMETER "B"	0.96		
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	300.0		
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	4.9		
WATER CONTENT AFTER CONSOLIDATION, %	14.3		
AVERAGE RATE OF STRAIN, %/hr	0.5		
TIME TO FAILURE, DAYS	1		
WATER CONTENT AFTER TEST, %	15.5		
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	262.6		
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	5.9		
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, $(\sigma_1 / \sigma_3)$ MAXIMUM	3.4		
DEVIATOR STRESS AT $(\sigma_1 / \sigma_3)$ MAXIMUM, kPa	243.0		
AXIAL STRAIN AT $(\sigma_1 / \sigma_3)$ MAXIMUM, %	11.2		
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.71		
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 / \sigma_3)$ MAXIMUM	0.82		
NATURAL WATER CONTENT, %	16.2		
DRY DENSITY, Mg/m <sup>3</sup>	1.86		
FILTER DRAINS USED, y/n	y		
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-		
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-		
FAILURE PLANE NUMBER	1.0		
ANGLE OF FAILURE, DEGREES	55.0		

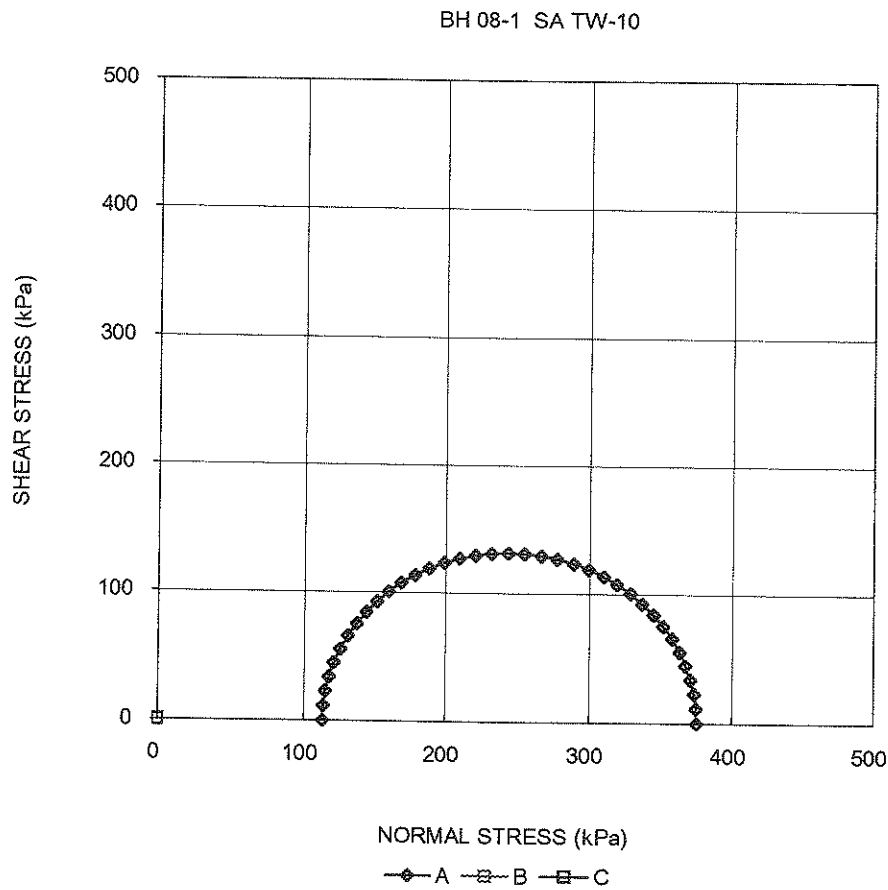
Date: 12/23/2008  
Project No. 08-1116-0032

**Golder Associates**

Prepared By: MM  
Checked By: *ll*

CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 2 OF 4

FIGURE



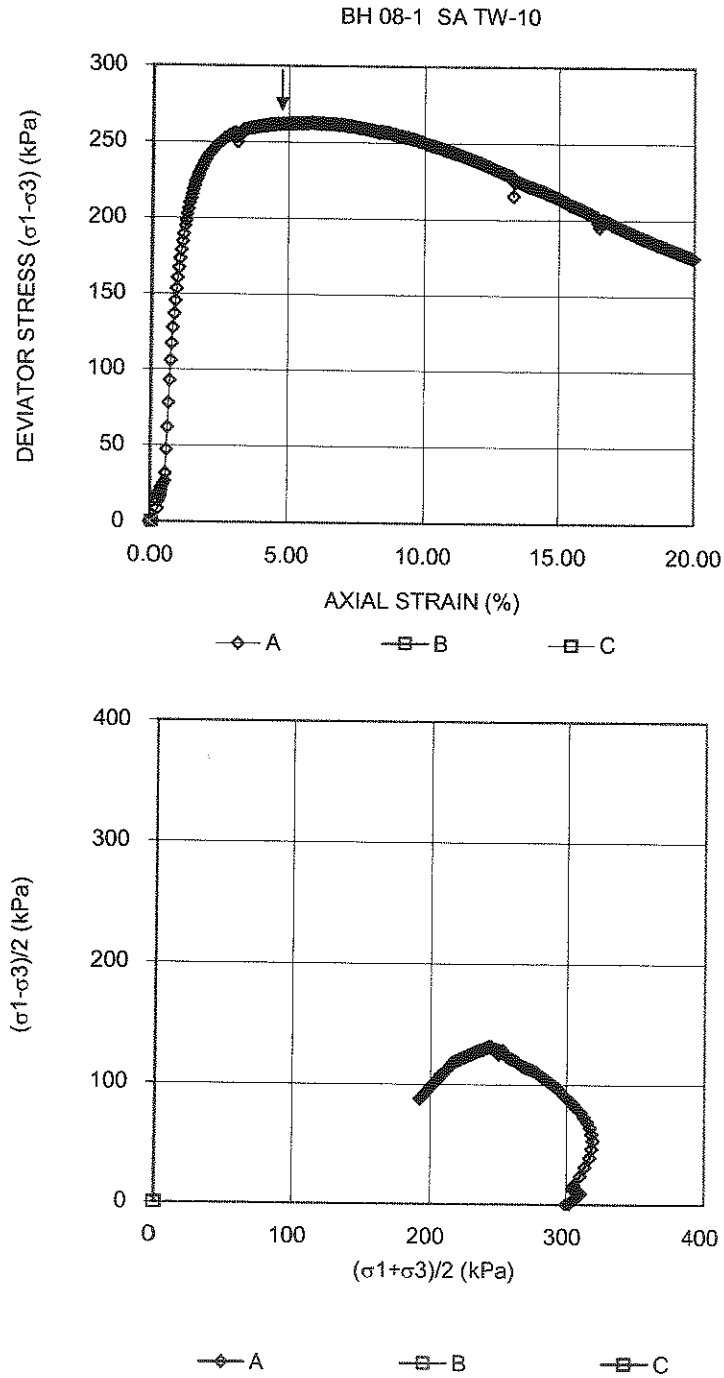
Date: 12/23/2008  
Project No. 08-1116-0032

Golder Associates

Prepared By: MM  
Checked By: RO

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 3 OF 4**

**FIGURE**



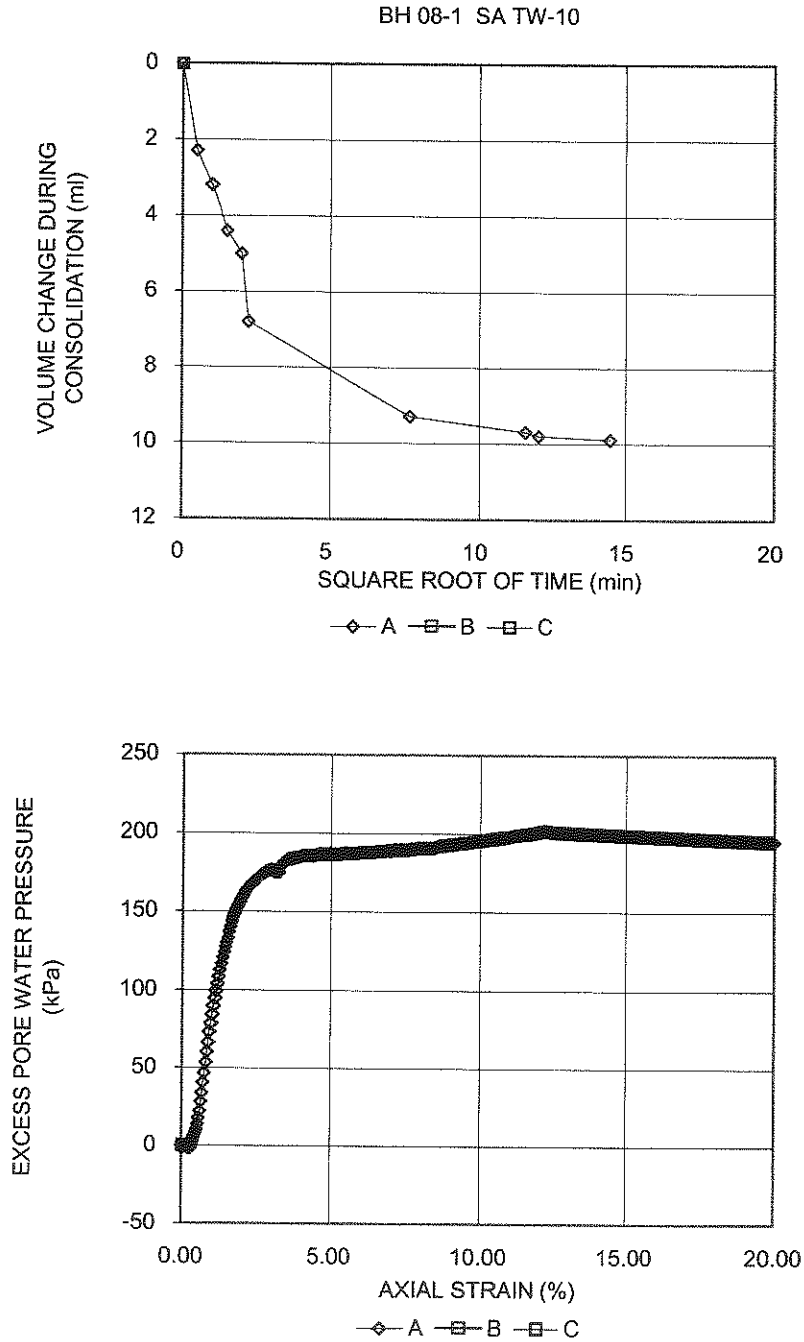
Date: 12/23/2008  
Project No. 08-1116-0032

**Golder Associates**

Prepared By: MM  
Checked By: *PO*

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 4 OF 4**

**FIGURE**



Date: 12/23/2008  
Project No. 08-1116-0032

**Golder Associates**

Prepared By: MM  
Checked By: Ro

<b>CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS</b> <b>SHEET 1 OF 4</b>		<b>FIGURE</b>	
TEST STAGE	A	B	C
BOREHOLE NUMBER	08-3		
SAMPLE	TW-2		
SPECIMEN DIAMETER, cm	5.00		
SPECIMEN HEIGHT, cm	10.19		
WATER CONTENT BEFORE CONSOLIDATION, %	15.3		
CELL PRESSURE, $\sigma_3$ , kPa	255.0		
BACK PRESSURE, kPa	135.0		
PORE PRESSURE PARAMETER "B"	0.97		
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	120.0		
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	4.7		
WATER CONTENT AFTER CONSOLIDATION, %	12.8		
AVERAGE RATE OF STRAIN, %/hr	0.5		
TIME TO FAILURE, DAYS	1		
WATER CONTENT AFTER TEST, %	13.8		
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	297.7		
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	12.5		
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, $(\sigma_1 / \sigma_3)$ MAXIMUM	3.5		
DEVIATOR STRESS AT $(\sigma_1 / \sigma_3)$ MAXIMUM, kPa	264.9		
AXIAL STRAIN AT $(\sigma_1 / \sigma_3)$ MAXIMUM, %	6.2		
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	-0.02		
PORE PRESSURE PARAMETER, $A_f$ , AT $(\sigma_1 / \sigma_3)$ MAXIMUM	0.05		
NATURAL WATER CONTENT, %	14.5		
DRY DENSITY, Mg/m <sup>3</sup>	1.93		
FILTER DRAINS USED, y/n	y		
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-		
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-		
FAILURE PLANE NUMBER	1.0		
ANGLE OF FAILURE, DEGREES	60.0		

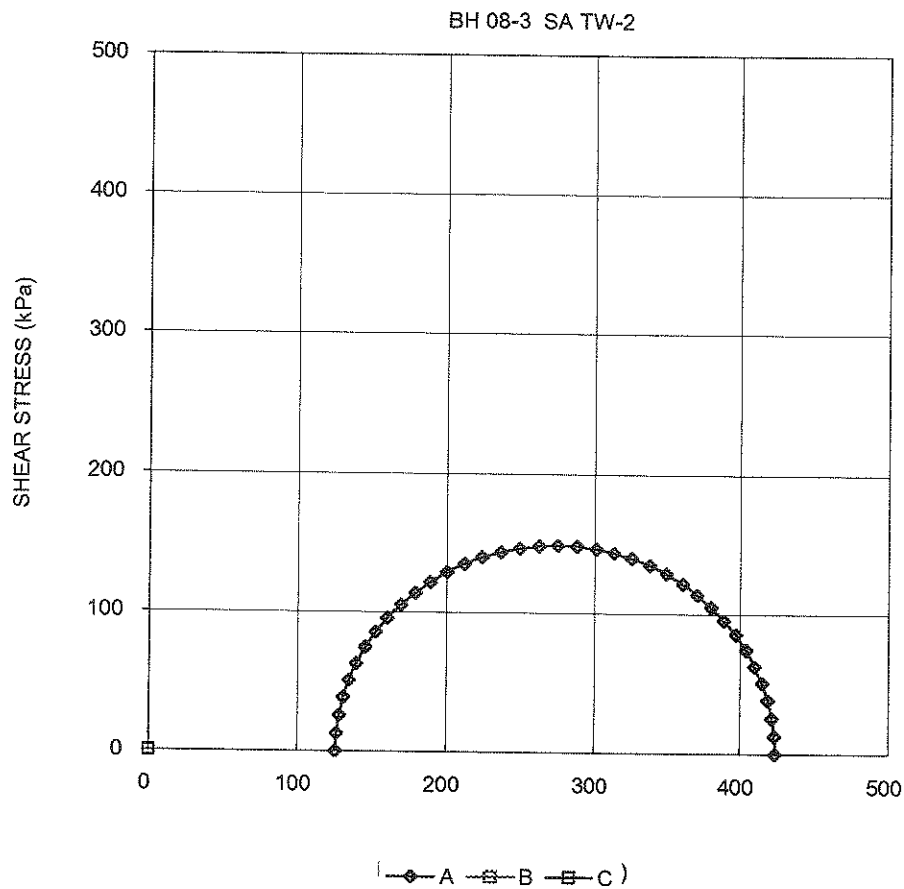
Date: 12/23/2008  
 Project No. 08-1116-0032

**Golder Associates**

Prepared By: MM  
 Checked By: Ro

CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 2 OF 4

FIGURE



Date: 12/23/2008  
Project No. 08-1116-0032

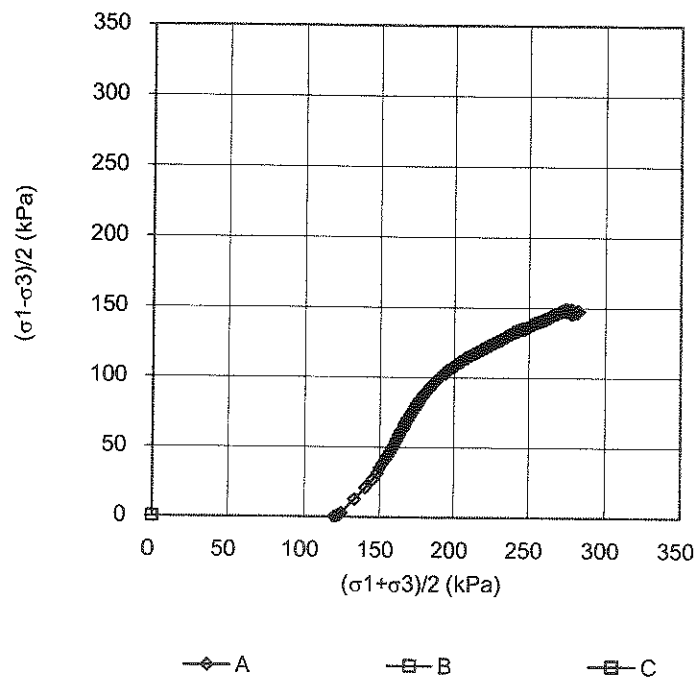
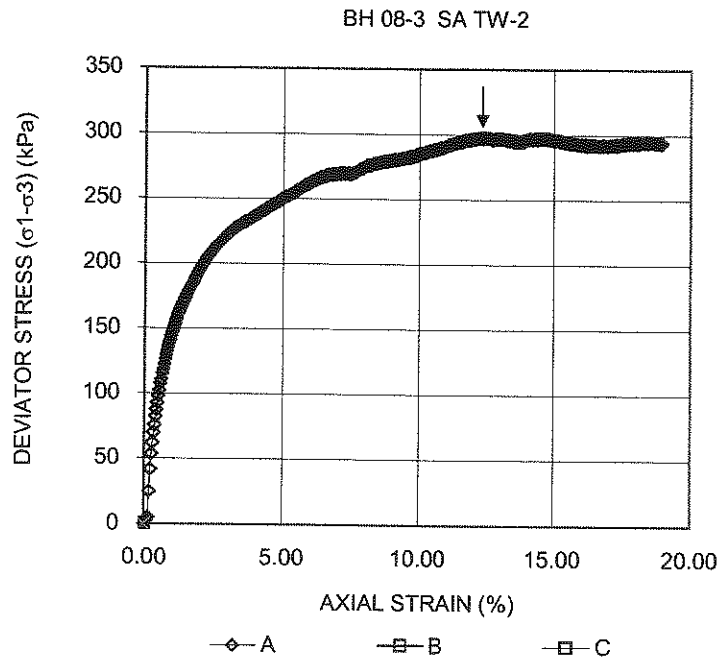
**Golder Associates**

Prepared By: MM  
Checked By: Ro



**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 3 OF 4**

**FIGURE**



Date: 12/23/2008  
Project No. 08-1116-0032

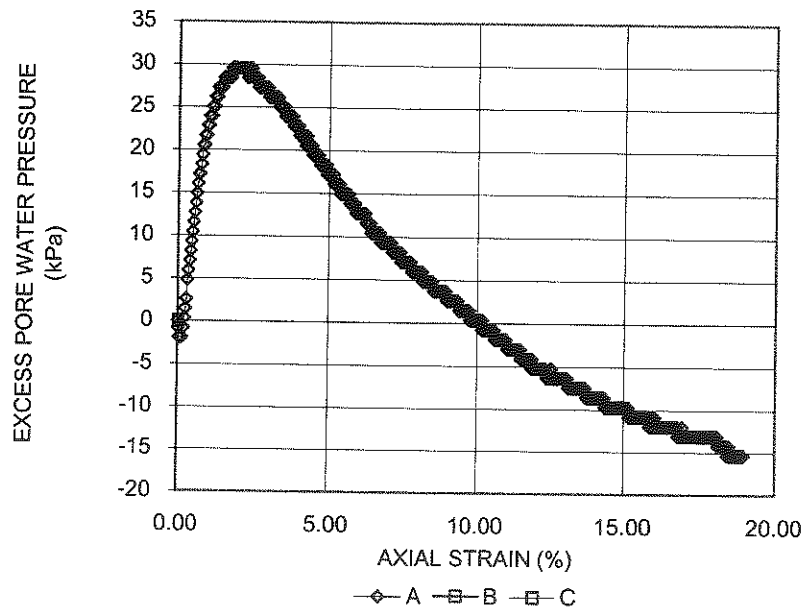
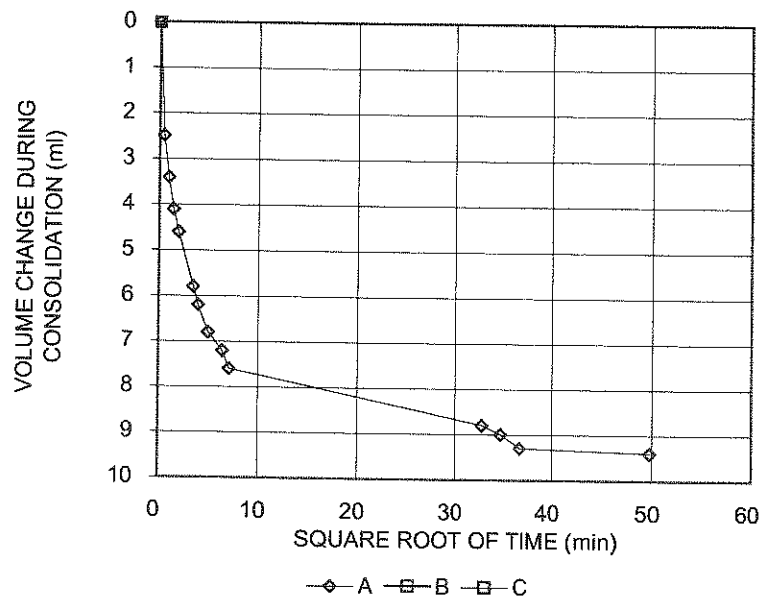
**Golder Associates**

Prepared By: MM  
Checked By: RO

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 4 OF 4**

**FIGURE**

BH 08-3 SA TW-2



Date: 12/23/2008  
Project No. 08-1116-0032

**Golder Associates**

Prepared By: MM  
Checked By: *LO*

## SPECIFIC GRAVITY TEST RESULTS

### ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	08-1116-0032	
PROJECT NAME	Thurber / Lab Testing / 19-1351-122	
DATE TESTED	December, 2008	
Borehole	Sample	Specific
No.	No.	Gravity
08-1	TW10	2.76
08-3	TW2	2.76

Note: Test carried out on soil particles <4.75mm using distilled water.

Checked By:



**Golder Associates**

## **Appendix C**

### **Photographs, Figures and Tables**

## Highway 401 Westbound Collectors – CPR Overhead

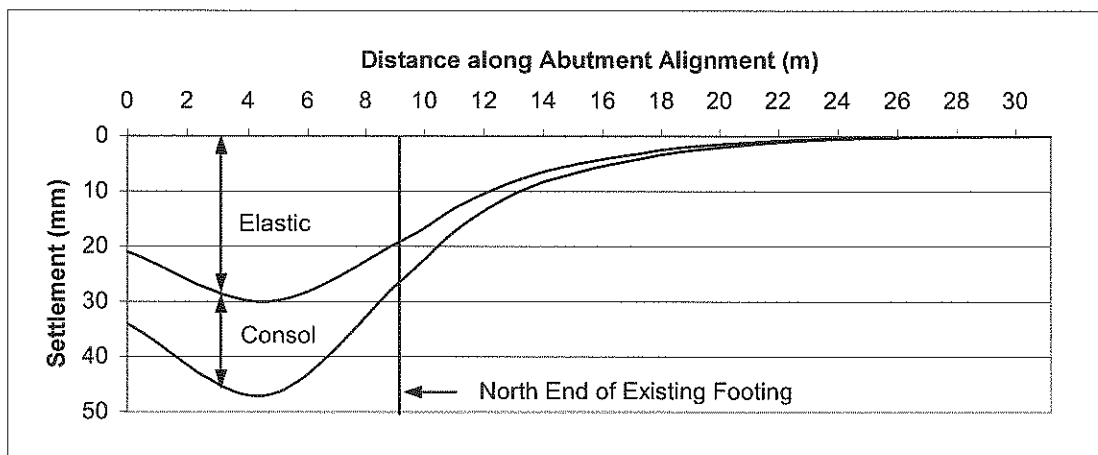
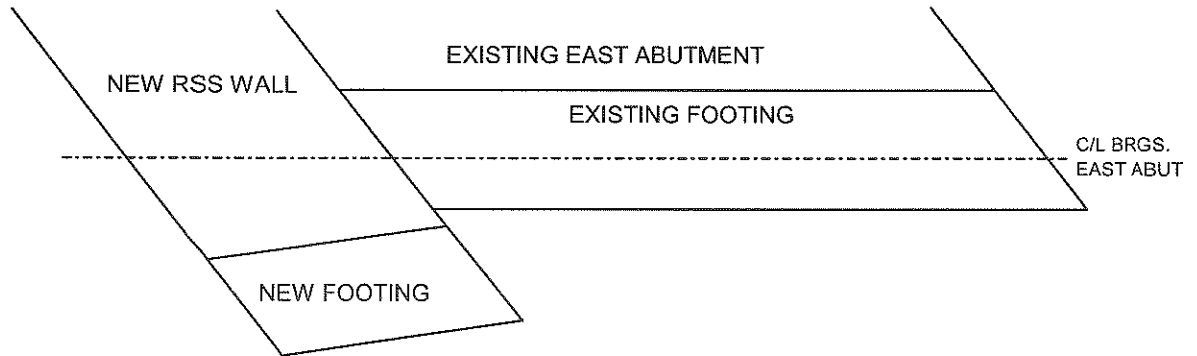


Photograph 1: View of CPR overhead opening looking east.

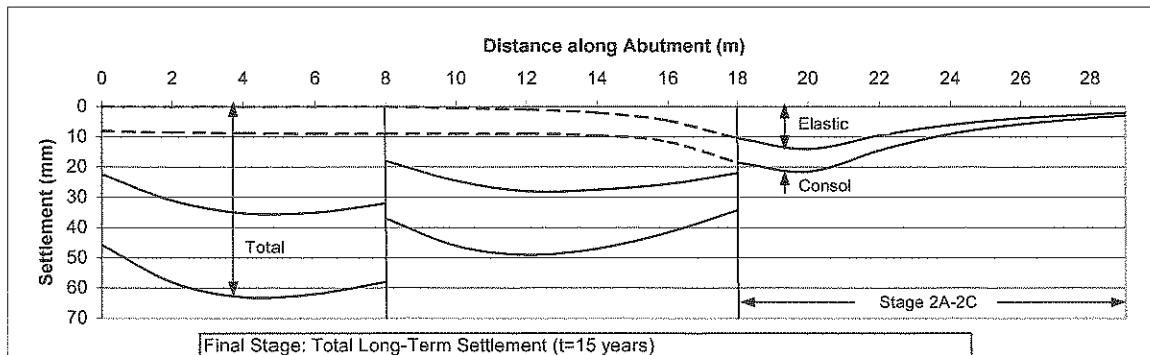
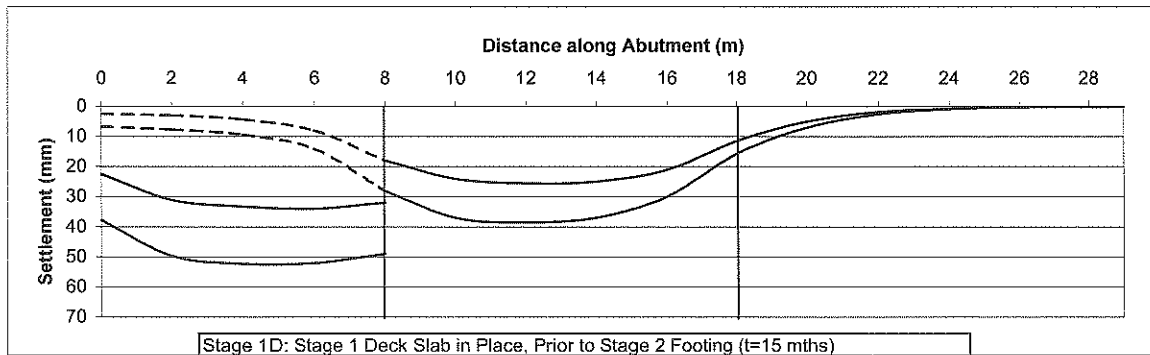
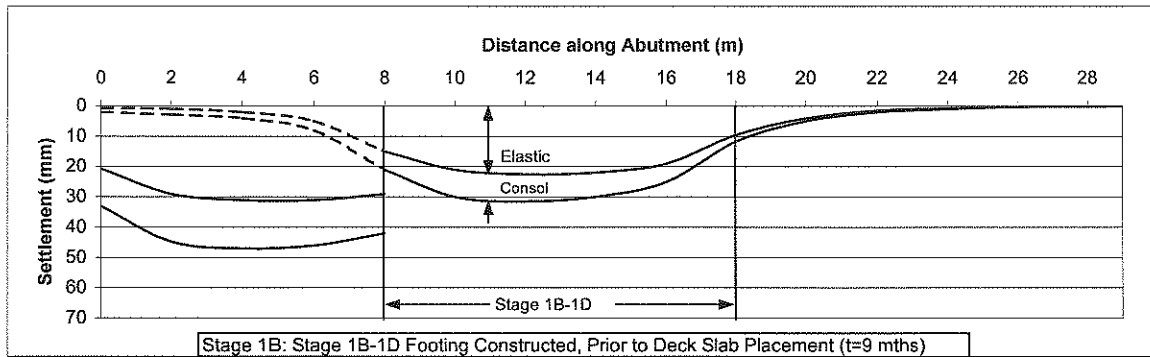
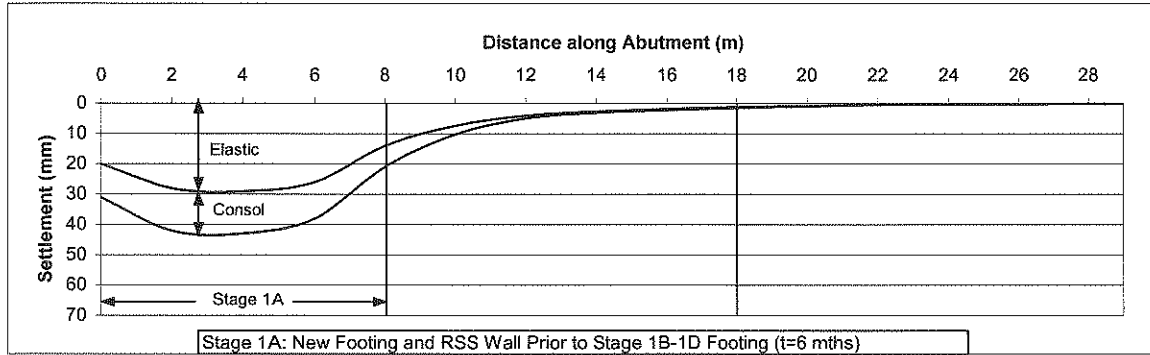
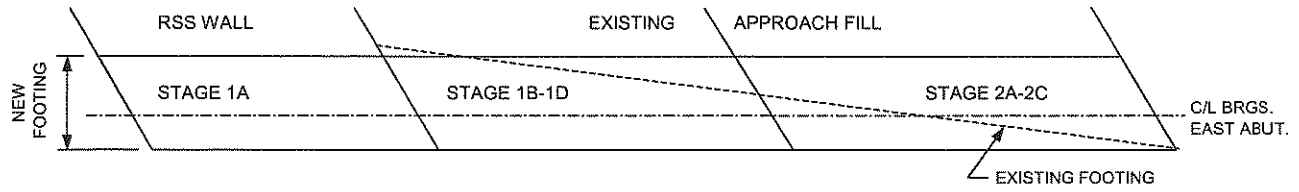


Photograph 2: Rail level inside of CPR overhead

**Hwy 401 - Westbound Collectors at CPR Structure**  
**Figure C1: Estimated Settlement Along Existing East Abutment**



Hwy 401 - Westbound Collectors at CPR Structure  
Figure C2: Anticipated Settlement Along East Abutment





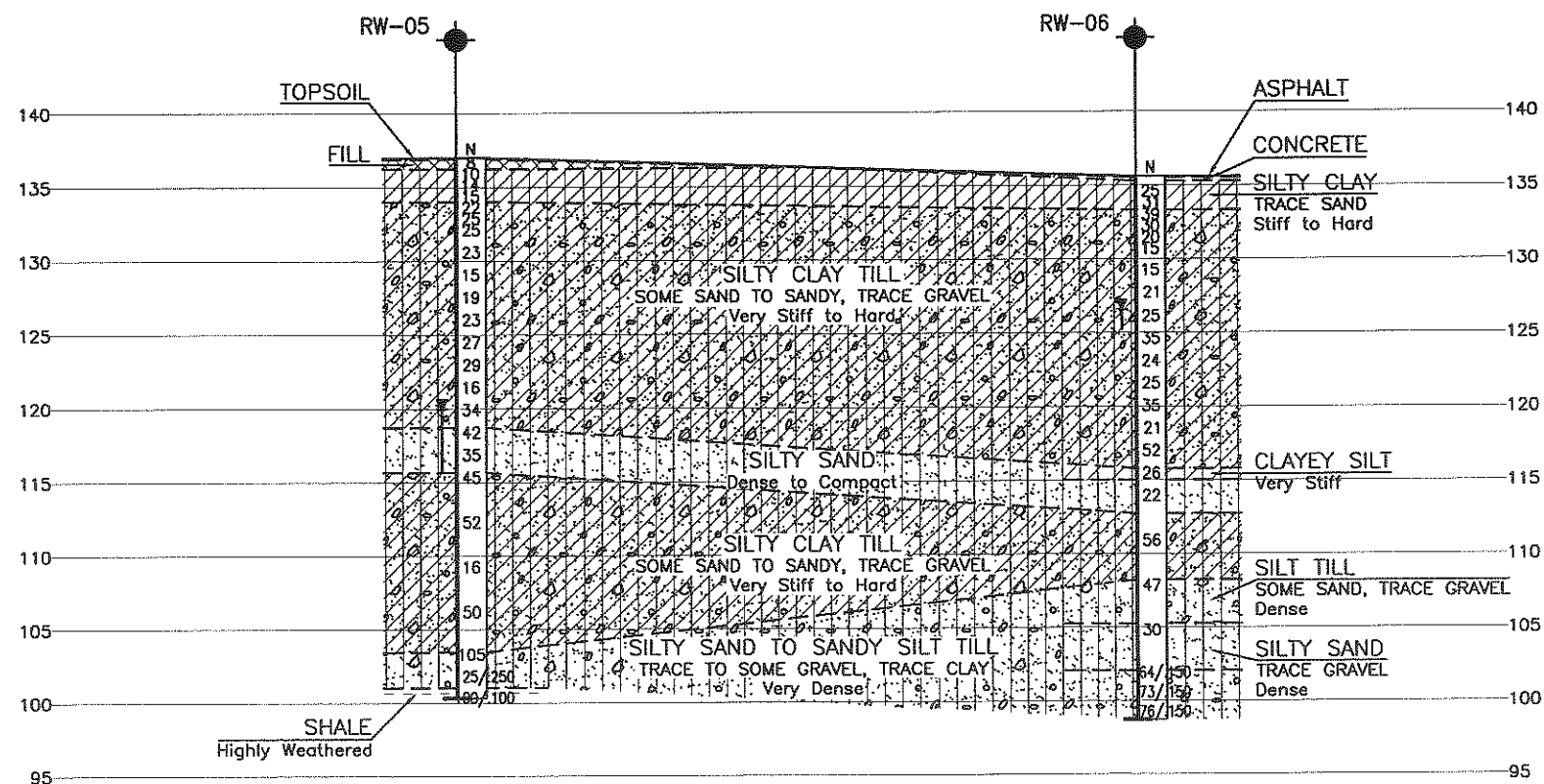
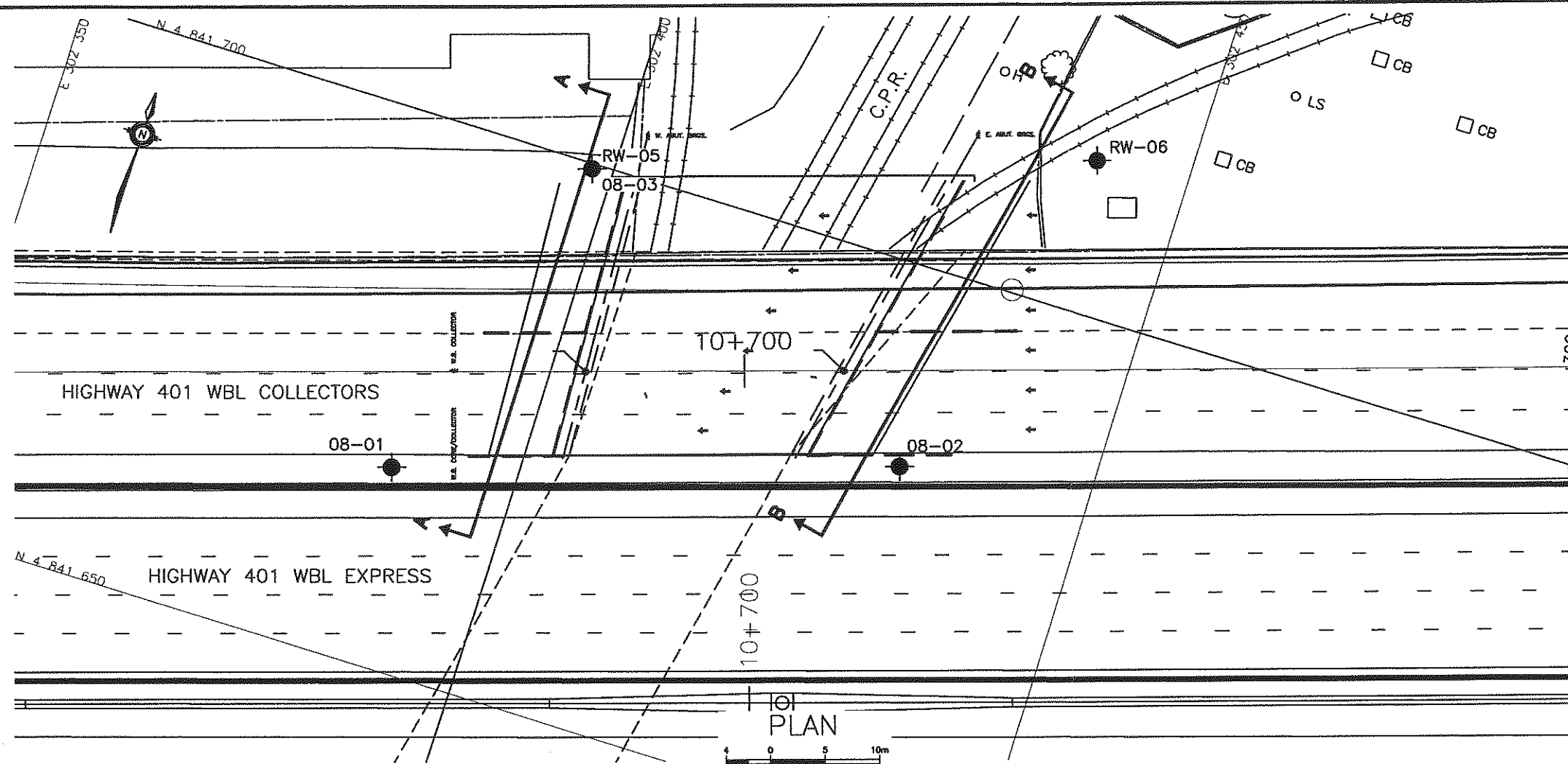
## COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Driven Piles	Caissons
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Compatible with existing foundations.</li> <li>ii. Ease of construction.</li> <li>iii. Lower cost than deep foundations.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. May need to step footings down locally to penetrate fill or softer material.</li> <li>ii. Potential for excessive post-construction settlement.</li> </ul> <p><b>RECOMMENDED</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Would permit use of higher geotechnical resistance than is available on the native soil.</li> <li>ii. Lower cost than deep foundations.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Cost of constructing engineered fill.</li> <li>ii. Potential for undermining of existing footings.</li> <li>iii. Potential for post-construction settlement.</li> <li>iv. Excavation and disposal of silty clay required.</li> </ul> <p><b>NOT RECOMMENDED</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Piles will develop high geotechnical resistance if driven to refusal.</li> <li>ii. Construction of piles could continue in freezing weather.</li> <li>iii. Readily installed.</li> <li>iv. Minimizes settlement.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Inconsistent with existing foundations.</li> <li>ii. Higher unit costs than footings.</li> <li>iii. Relatively long pile length.</li> <li>iv. Confined space during staged construction may restrict the size of pile driving equipment.</li> <li>v. Flat slopes will be required for access of large pile driving equipment to the foundation area.</li> </ul> <p><b>NOT RECOMMENDED</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High resistance is available for caissons founded on very dense material or bedrock at depth.</li> <li>ii. Construction of caissons could continue in freezing weather.</li> <li>iii. Minimizes settlement.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Incompatible with existing foundations.</li> <li>ii. Significant depth to bearing stratum.</li> <li>iii. Higher unit costs than footings.</li> <li>iv. Possibility of boulders being encountered during augering.</li> <li>v. Difficulty excluding seepage and flow of soil under rim of liner.</li> </ul> <p><b>NOT RECOMMENDED</b></p>



## **Appendix D**

### **Borehole Locations and Soil Strata Drawings**



# PROFILE ALONG HIGHWAY 401

## METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 2147-01-00

HIGHWAY 401 W.B. COLLECTORS  
C.P.R. OVERHEAD  
REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA



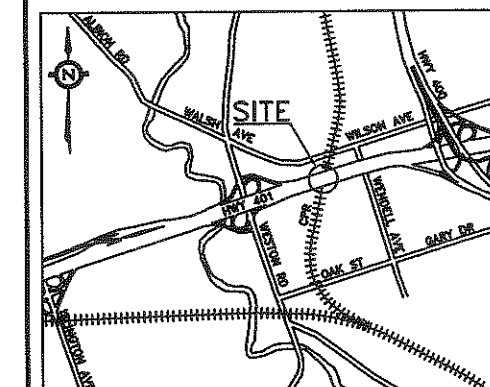
SHEET



**McCORMICK RANKIN  
CORPORATION**








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## KEYPLAN

## LEGEND

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

[illegible]

-NOTES-

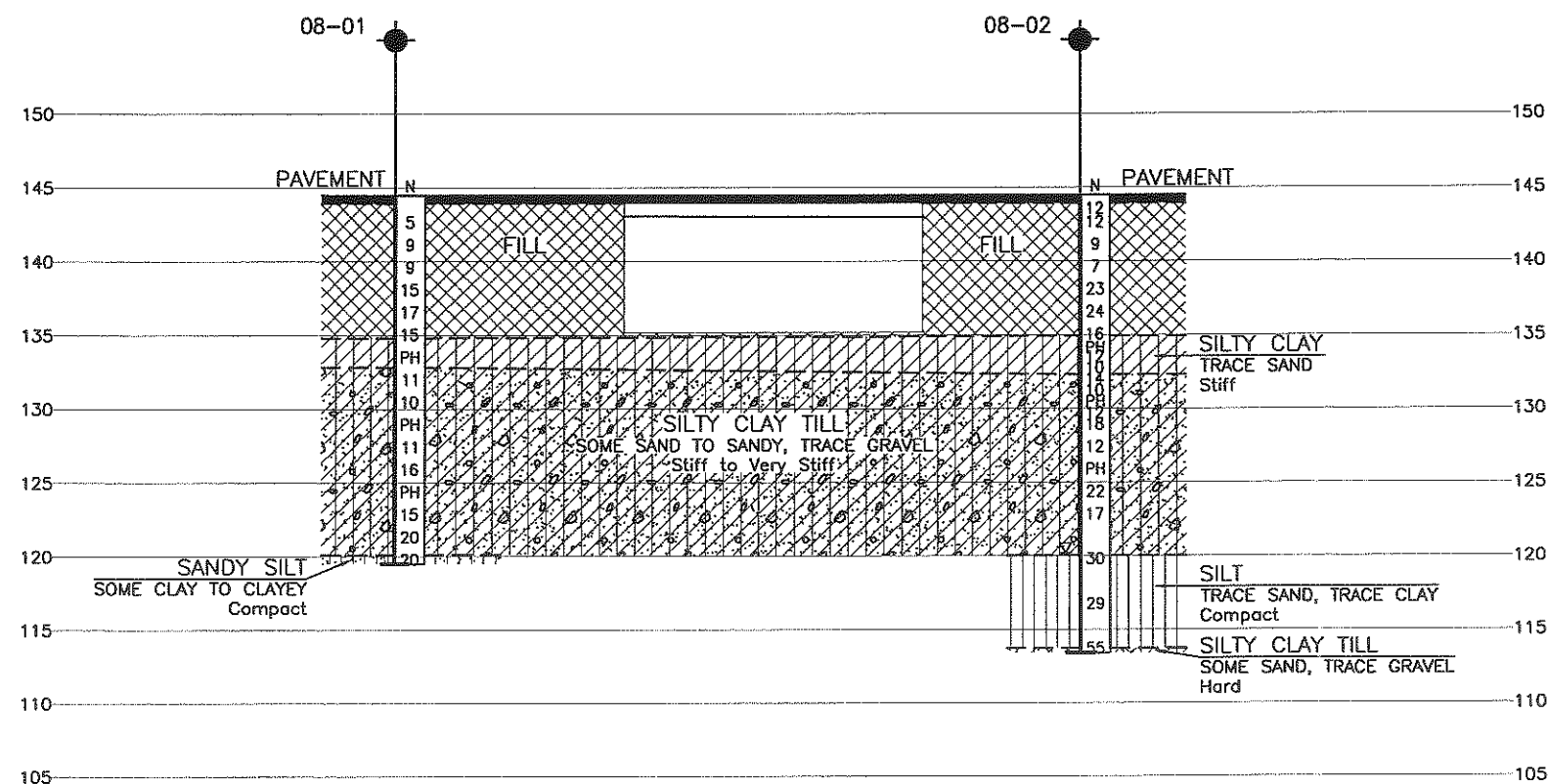
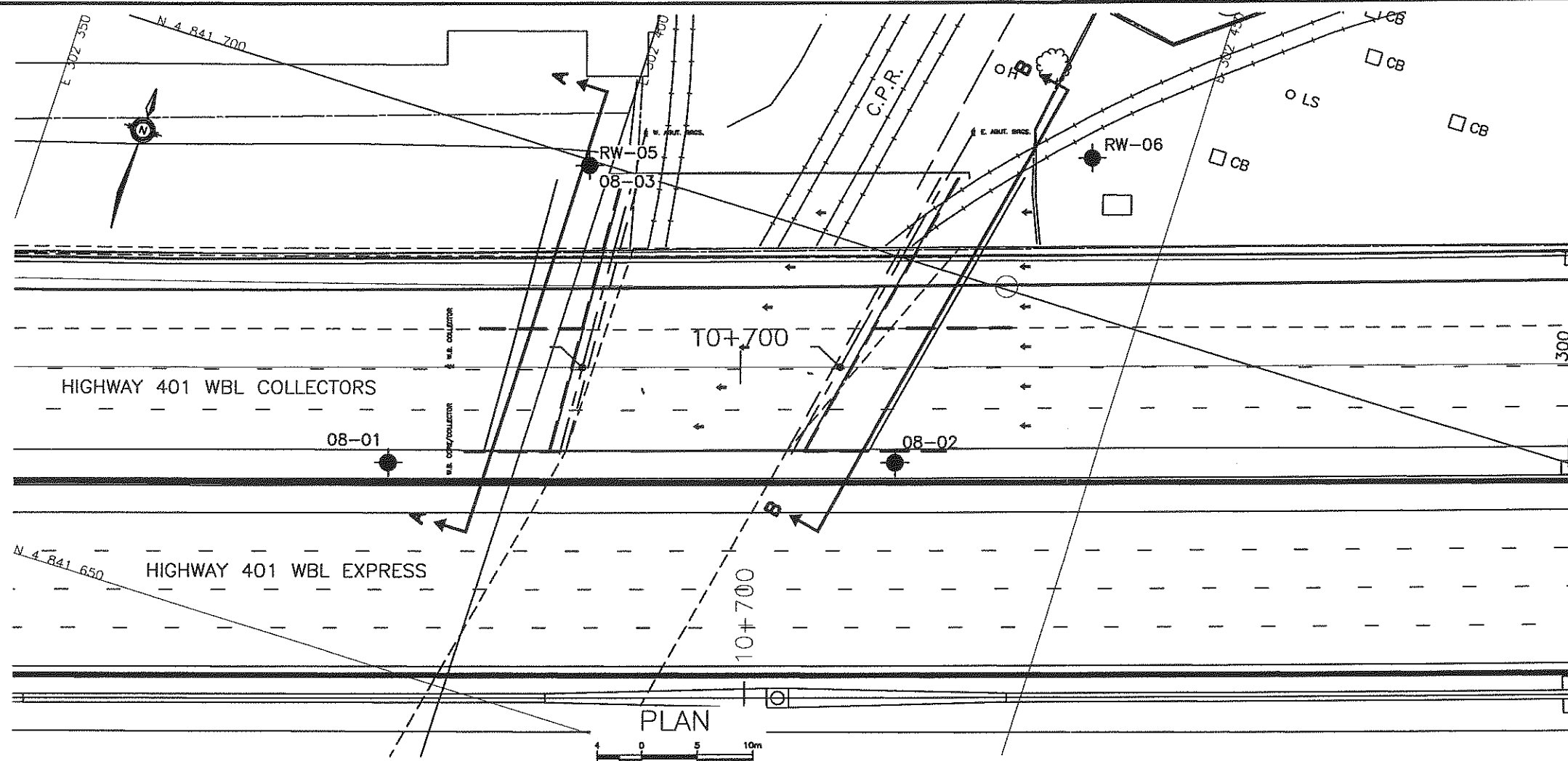
- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

**GEOCRES No. 30M11-231**



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE		BY		DESCRIPTION				DATE	DEC. 200
	DATE	BY	CHK	PKC	CODE	LOAD	STRUCT	DWG	1	
DESIGN	MRA	CHK	PKC	CODE	LOAD	STRUCT	DWG	1		
DRAWN	MFA	CHK	MRA	SITE	STRUCT	DWG	1			



# PROFILE ALONG HIGHWAY 401

A horizontal number line with tick marks at 0, 5, and 10. The segment between 0 and 5 is shaded with diagonal lines.

METRIC

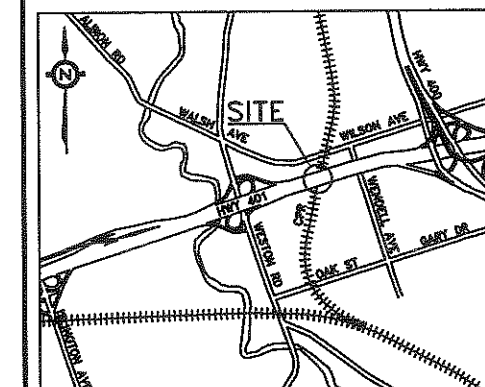
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 2147-01-00  
HIGHWAY 401 W.B. COLLECTORS  
C.P.R. OVERHEAD  
REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

**McCORMICK RANKIN CORPORATION**








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## KEYPLAN

## LEGEND

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

NO	ELEVATION	NORTHING	EASTING
RW-05	137.0	4 841 699.5	302 398.1
RW-06	135.6	4 841 713.9	302 441.6
08-01	144.5	4 841 668.1	302 389.0
08-02	144.5	4 841 681.9	302 432.8
08-03	137.0	4 841 699.5	302 398.1

-NOTES-

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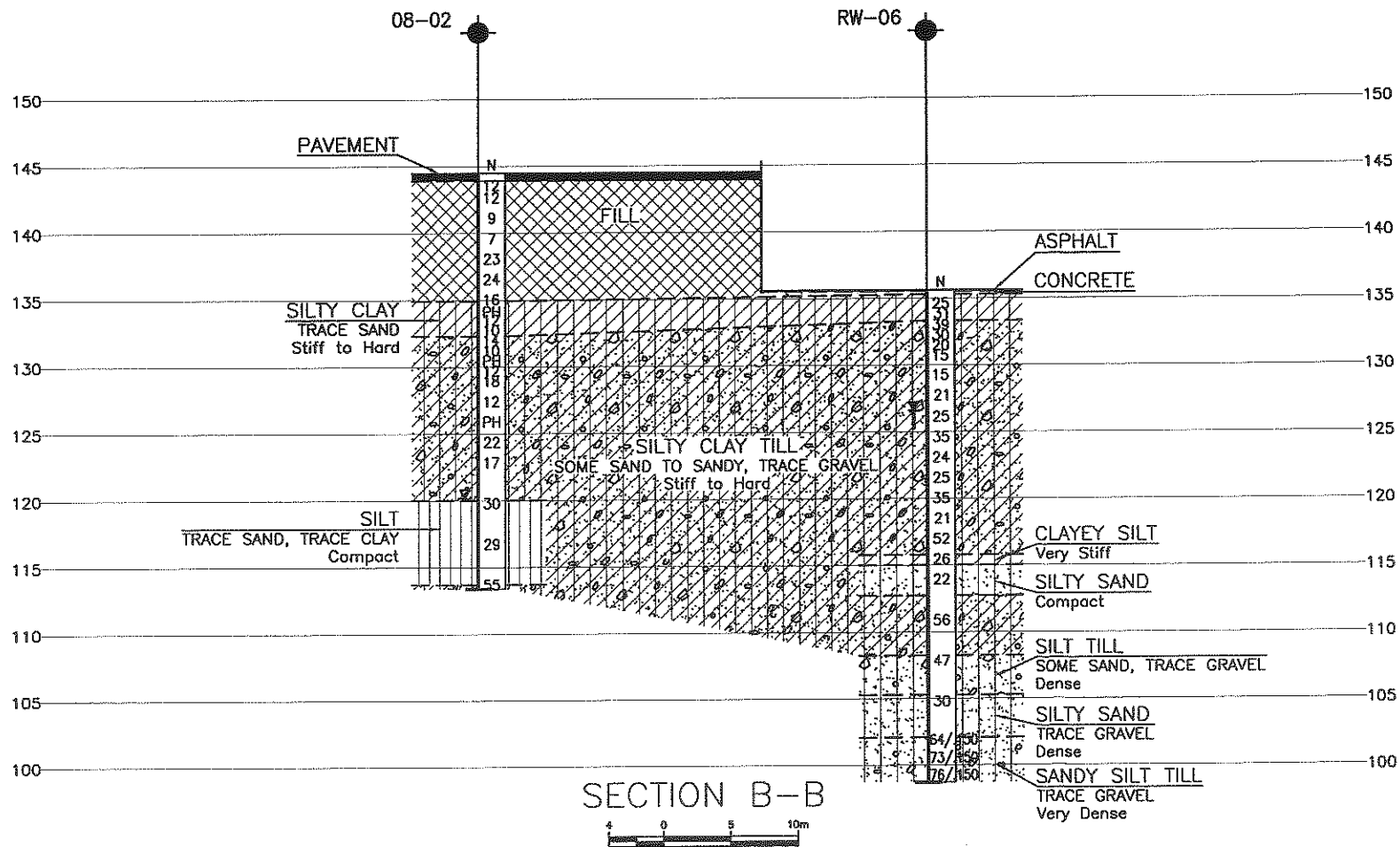
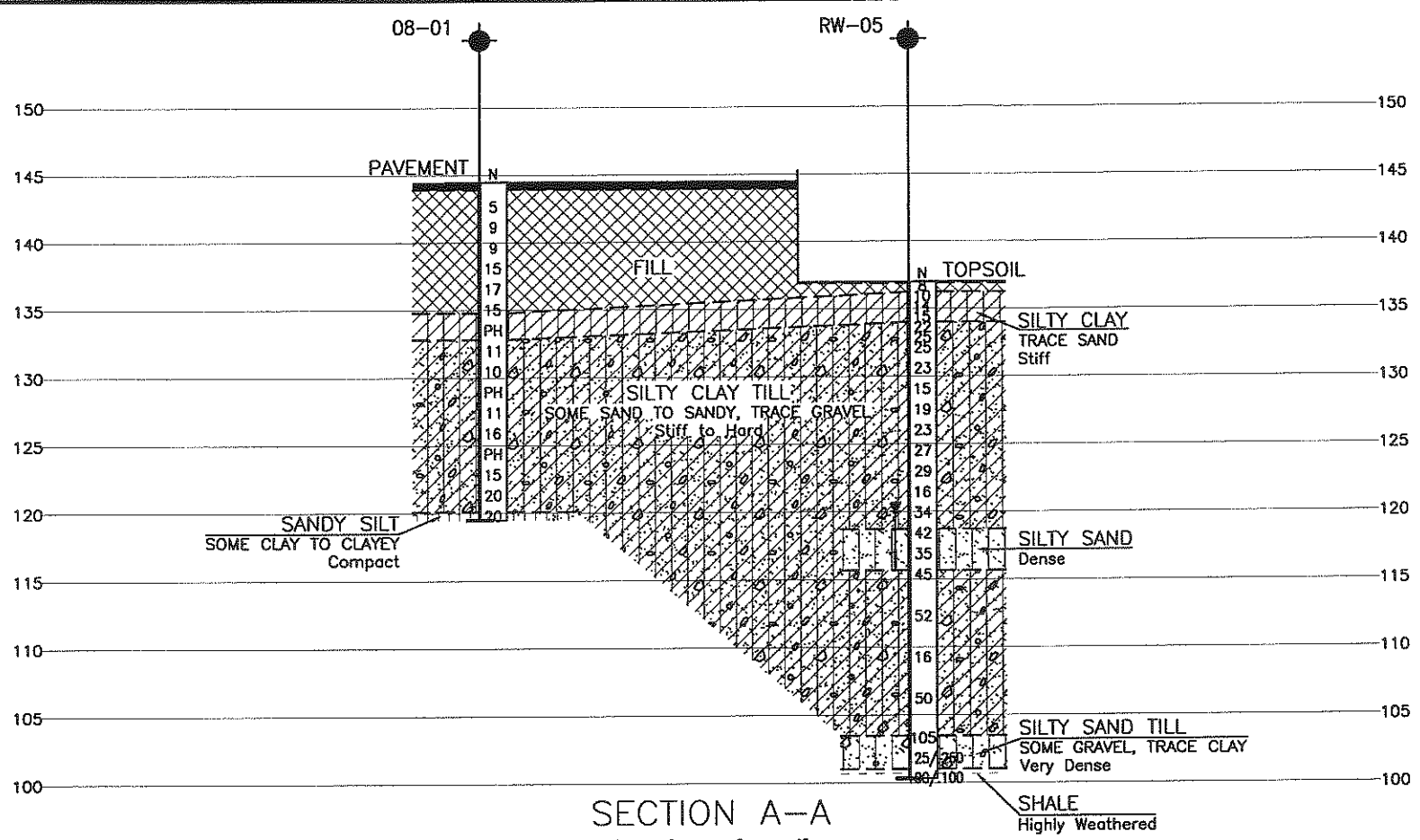
**GEOCRES No. 30M11-231**

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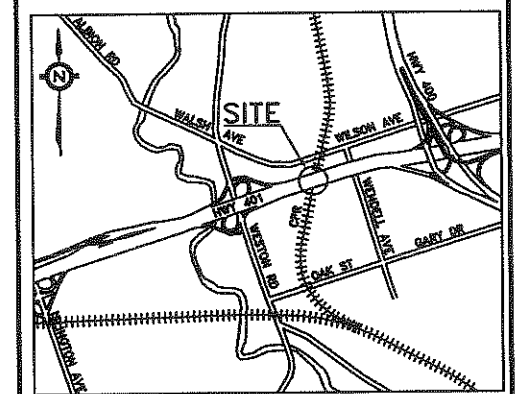
**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 2147-01-00  
HIGHWAY 401 W.B. COLLECTORS  
C.P.R. OVERHEAD  
REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**McCORMICK RANKIN CORPORATION**

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**KEYPLAN  
LEGEND**

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

NO	ELEVATION	NORTHING	EASTING
RW-05	137.0	4 841 699.5	302 398.1
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08-01	144.5	4 841 668.1	302 389.0
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**GEOCRES No. 30M11-231**



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

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