

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
5th LINE UNDERPASS AT HIGHWAY 400
TOWN OF BRADFORD WEST GWILLIMBURY
ONTARIO

Geocres Number: 31D-504

Report to

McCormick Rankin Corporation

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TABLE OF CONTENTS

PART 1 FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Topsoil and Fill	4
5.2	Clayey Silt.....	4
5.3	Clayey Silt Till.....	5
5.4	Sand	6
5.5	Groundwater	6
6	MISCELLANEOUS.....	7

PART 2 ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	INTRODUCTION	8
8	STRUCTURE FOUNDATIONS.....	8
8.1	Spread Footings	9
8.1.1	Spread Footings on Native Soil.....	9
8.1.2	Spread Footings on Engineered Fill	9
8.2	Driven Steel Piles.....	10
8.3	Drilled Shafts (Caissons)	11
8.4	Recommended Foundation	12
8.5	Abutment Type	12
8.6	Frost Protection.....	12
9	EARTH PRESSURE COEFFICIENTS (ABUTMENTS)	13
10	APPROACH EMBANKMENTS	14
11	CONSTRUCTION CONCERNS	15
12	SCOPE OF DETAILED INVESTIGATION	15
13	CLOSURE.....	16

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Foundation Comparison
Appendix D	Preliminary Stability Analyses
Appendix E	Borehole Locations and Soil Strata Drawing

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the site of a proposed new underpass structure at Highway 400 and 5th Line in Bradford West Gwillimbury. The proposed structure will replace the existing 5th Line flyover located to the south, and will be constructed as part of a new interchange proposed at the site.

A preliminary foundation investigation consisting of two boreholes drilled at the location of the existing structure was carried out in December 2000 for replacement of the structure along the existing alignment (Golder Associates report dated December 2001, G.W.P. 40-00-00). One of the boreholes was drilled close to the revised structure location and is referenced in the current report.

The purpose of the current investigation was to conduct additional exploration along the revised alignment of the structure and, based on the data obtained, to provide a borehole location plan, record of borehole sheets, a preliminary stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the present and previous investigation.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation, ultimately for the Town of Bradford West Gwillimbury.

2 SITE DESCRIPTION

The site is located approximately 45 m north of the existing flyover structure carrying 5th Line over Highway 400 and approximately 2.5 km south of the interchange of Highway 400 and Simcoe Road 88 (former Highway 88) in the Town of Bradford West Gwillimbury, Ontario. The new structure site is along the original alignment of 5th Line prior to construction of the existing structure.

Existing road grade on Highway 400 is near Elevation 225 m. The ground surface adjacent to the highway is slightly above the highway grade on the east side and slightly below the highway grade on the west side. A treed area with a small watercourse is present on the west side of the highway.

The surrounding land is gently undulating and used primarily for agricultural purposes.

Physiographically, the site lies within the Schomberg Clay Plains which consists of deep deposits of stratified clay and silt overlying a drumlinized till plain. The drumlins are completely or partially buried by the clay and silt deposits, depending on the size of the drumlin. The silt and clay deposits are typically 5 m thick, although thicker deposits of about 15 m have been identified at nearby locations.

3 SITE INVESTIGATION AND FIELD TESTING

A preliminary foundation investigation was carried out at the location of the existing structure in December 2000 and the results were documented in a Foundation Investigation Report dated December 2001 (Golder Associates). The investigation consisted of two boreholes (designated B2-1 and B2-2) advanced to depths of 33.6 and 30.9 m. The Record of Borehole sheets for these boreholes are reproduced in Appendix A. Borehole B2-1 was located near the east abutment of the new bridge alignment.

The current site investigation consisted of two boreholes drilled and sampled to 32.4 m depth. Borehole 10-01 was located near the proposed east abutment and drilled between March 24 and 26, 2010. Borehole 10-02 was located near the west abutment and drilled between April 5 and 8, 2010. The approximate locations of the boreholes drilled during the previous and current investigations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix E.

Prior to commencing the site investigation, clearance was obtained from utility companies having plant in the area.

A CME-55 track-mounted drill rig was used to drill the boreholes. Hollow stem augers were used to advance the boreholes to approximately 19 m depth, followed by tricone wash-boring equipment to complete the borehole. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In addition to the SPT samples, two thin wall Shelby tube samples were collected from each borehole at selected depths. The in situ shear strength of the cohesive soils was also assessed using the MTO shear vane.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer consisting of 25 mm diameter Schedule 40 PVC pipe with a slotted screen was installed in Borehole 10-01 to permit longer term groundwater level monitoring. Borehole 10-02 was grouted with bentonite upon completion, and a shallow piezometer was installed in a

separate adjacent borehole augered to 9.1 m depth. The completion details of the piezometers are shown in Table 3.1. Following the final water level reading, the piezometers were decommissioned in accordance with MOE Regulation 903.

Table 3.1 – Borehole Completion Details

Borehole	Piezometer Tip		Completion Details
	Depth (m)	Elevation (m)	
10-01	31.1	194.4	Piezometer with 3.0 m slotted screen installed with sand filter to 27.3 m, bentonite seal from 27.3 m to 2.6 m, cuttings from 2.6 m to ground surface.
10-02	-	-	Borehole backfilled with bentonite to ground surface
10-02A	9.1	215.5	Piezometer with 2.1 m slotted screen installed with sand filter to 6.2 m, bentonite seal from 6.2 m to 1.7 m, cuttings from 1.7 m to ground

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) 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 also subjected to gradation analysis and Atterberg Limits testing, and the results are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Two thin wall tube samples, one from each borehole, were selected for one-dimensional consolidation tests. The results are shown on the charts in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

A detailed description of the soil stratigraphy encountered in the boreholes is presented in the Record of Borehole sheets included in Appendix A and on the “Borehole Locations and Soil Strata” drawing in Appendix E. A general description of the stratigraphy encountered in boreholes 10-01, 10-02 and B2-1 drilled at the proposed alignment is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general, the stratigraphy encountered in the boreholes consists of a thin layer of topsoil or fill overlying a thick deposit of firm to very stiff clayey silt. A layer of hard clayey silt till, and an underlying sand deposit at the west abutment, was encountered below the clayey silt.

5.1 Topsoil and Fill

A 200 mm thick layer of topsoil was encountered surficially in boreholes 10-01 and 10-02 drilled during the current investigation. In previous borehole B2-1, a 600 mm thick layer of clayey silt fill was encountered surficially.

The topsoil/fill thickness may vary between and beyond the borehole locations, and the limited data presented in this report should not be used for quantity estimation purposes.

5.2 Clayey Silt

Clayey silt was encountered below the topsoil layer in boreholes 10-01 and 10-02. This deposit was also encountered below the fill in borehole B2-1, and was previously identified as till. The clayey silt generally contains trace to no sand and gravel, but incorporates zones of till-like material with a greater percentage (some) of sand. It is described as brown becoming grey near 3.7 m depth. The deposit of clayey silt is 25.9 to 28.2 m thick, with a lower boundary at depths of 26.1 to 28.4 m (elevation 197.1 to 198.5 m).

The results of laboratory grain size distribution tests carried out on samples of the clayey silt during the current investigation are illustrated in Figures B1 and B2, Appendix B. The Atterberg Limits test results are plotted on Figures B5 and B6, Appendix B. The results were as follows:

Gravel (%)	0 to 2	
Sand (%)	0 to 15	
Silt (%)	45 to 79	
Clay (%)	21 to 54	
Liquid Limit	21 to 37	(one value of 49)
Plastic Limit	12 to 19	(one value of 29)

The results indicate that the clayey silt typically has low plasticity (CL), with zones of slight to medium plasticity (CL-ML to CI). One sample from 12.5 m depth in borehole 10-01 indicated a borderline CI to MI classification.

SPT N-values recorded in the clayey silt ranged from 3 to 28 blows for 0.3 m of penetration, locally 30 to 37 blows/0.3 m below 18 m depth in borehole B2-1. In situ vane testing indicated that the undrained shear strength typically exceeds 100 kPa, with only one result (56 kPa) below this value. Based on the N-values and vane test results, the clayey silt is generally firm to very stiff, and becomes hard below about 18 m depth in borehole B2-1.

The moisture content of samples from this deposit ranged from about 12 to 31%, typically 17 to 26%.

The results of consolidation testing conducted on two samples of the clayey silt are included in Appendix B and are summarized in Table 5.1.

Table 5.1 – Consolidation Test Parameters

Borehole	Sample Depth (m)	Soil Type	w _o (%)	γ (kN/m ³)	e _o	p _o ' (kPa)	p _c ' (kPa)	OCR	C _c	C _r
10-01	12.2-12.8	CI-MI	24	20.0	0.65	170	170	1.0	0.18	0.021
10-02	6.1-6.7	CL	20	20.7	0.55	80	145	1.8	0.15	0.017

Comparison of the existing and preconsolidation pressures (p_o' and p_c') derived from the test results indicate that the natural clayey silt is normally consolidated to lightly preconsolidated. The coefficient of consolidation, c_v, recorded during the test was generally in the order of 2 to 4 x 10⁻² cm²/s for 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.

5.3 Clayey Silt Till

A deposit of grey clayey silt till was encountered below the clayey silt in all boreholes. The till contains some sand, trace gravel, and sand seams. Boreholes 10-01 and B2-1 were terminated in the clayey silt till at depths of 32.4 and 33.8 m (elevation 193.1 and 190.9 m), indicating a thickness of at least 4.0 and 6.2 m. In borehole 10-02, the till deposit was 1.5 m thick, with a lower boundary at 27.6 m depth (elevation 197.0 m).

The results of a grain size analysis and Atterberg Limits testing carried out on a single sample of the clayey silt till are presented in Figures B3 and B7, respectively (Appendix B). The results were as follows:

Gravel (%)	3
Sand (%)	18
Silt (%)	49
Clay (%)	30
Liquid Limit	21
Plastic Limit	13

The results indicate that the clayey silt till has low plasticity (CL).

SPT N-values recorded in the clayey silt till ranged from 37 blows/0.3 m to 100 blows/0.15 m, indicating a hard consistency. Glacial tills inherently contain cobbles and boulders. Moisture contents ranged from 12 to 19%.

5.4 Sand

Grey sand was encountered below the clayey silt till in borehole 10-02 at a depth of 27.6 m (elevation 197.0 m). The borehole was terminated in the sand at 32.4 m depth (elevation 192.2 m).

The results of a laboratory grain size distribution test carried out on a sample of the sand are shown in Figure B4, Appendix B and were as follows:

Gravel (%)	0
Sand (%)	89
Silt and Clay (%)	11

Standard Penetration tests in the sand gave 'N' values of 100 blows for 0.20 to 0.28 m of penetration, indicating a very dense condition.

The moisture content of samples from this unit varied between 17 and 19%.

5.5 Groundwater

Standpipe piezometers were installed in borehole 10-01, in a shallow borehole augered adjacent to borehole 10-02, and in borehole B2-1 drilled during the previous investigation. The groundwater depths and elevations measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Groundwater Depths and Elevations

Location	Borehole	Completion Date	Water Levels in Piezometers		
			Date	Depth (m)	Elevation (m)
East Abutment	10-01	26-Mar-10	31-Mar-10	2.6	222.9
			09-Apr-10	2.8	222.7
			20-Apr-10	2.7	222.8
			03-May-10	2.4	223.1
	B2-1	07-Dec-00	19-Jun-01	0.8 ags*	225.5
West Abutment	10-02	08-Apr-10	09-Apr-10	9.1	215.5
			20-Apr-10	1.5	223.1
			03-May-10	1.0	223.6

* above ground surface (artesian condition)

The above water levels reflect 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.

Based on the water levels measured in the piezometers, the groundwater level is expected to be near elevation 223.1 m at the east abutment, and elevation 223.6 m at the west abutment. The artesian condition reported during the earlier investigation was not confirmed during the current study and should be investigated further during detail design.

6 MISCELLANEOUS

The co-ordinates and ground elevations at the boreholes were established by Thurber using a GPS unit with an accuracy of 0.2 m.

DBW Drilling Limited of Ajax, Ontario supplied the drill rig and conducted the drilling, sampling and in-situ testing operations. The operations in the field were supervised on a full time basis by Mr. Stephane Loranger, C.E.T. of Thurber.

Supervision of the field program was performed by Mrs. Lindsey Blaine, B.A.Sc. Interpretation of the field data and preparation of the report was carried out by Mrs. Lindsey Blaine, B.A.Sc. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents preliminary geotechnical design recommendations to assist planning of the foundation system and approach fills for the proposed underpass structure replacement.

It is understood that a new two-span underpass structure will be constructed to carry 5th Line over Highway 400 at an alignment approximately 45 m north of the existing flyover structure. The new structure will carry four lanes plus potential additional width to accommodate Highway 400 on-ramps.

Structure approaches will consist of fill embankments in the order of 6 to 9 m high. Grades on Highway 400 will not be revised.

The discussion and preliminary recommendations presented in this report are based on the information provided by McCormick Rankin Corporation and on the factual data obtained in the course of the investigation.

8 STRUCTURE FOUNDATIONS

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).

Preliminary design parameters for viable alternatives are presented in the following sections. A preliminary recommendation regarding the foundation scheme preferred from a foundations perspective is presented based on the subsurface conditions identified at the site.

The subsurface stratigraphy encountered in the exploratory boreholes drilled at the site consists of a thick deposit of firm to very stiff clayey silt extending to depths of 26.1 to 28.4 m, underlain by hard clayey silt till and very dense sand. The clayey silt deposit could undergo significant settlement under foundation and embankment loads.

The most recent groundwater levels measured in piezometers installed during the current investigation were 1.0 and 2.4 m below the ground surface. An artesian condition was noted in a previous borehole at the east abutment.

8.1 Spread Footings

8.1.1 Spread Footings on Native Soil

Consideration was given to supporting the new structure on spread footings founded on the native clayey silt. However, the geotechnical resistance available in the native soil is relatively low, and the design must also consider the potential for long-term consolidation settlement of the foundation soils under the new approach embankment loading. For these reasons, the use of spread footings is not recommended.

If spread footings are considered, the footings should be founded on stiff to very stiff, undisturbed native soil at least 1.2 m below the ground surface (at or below elevation 224.1 m at the east abutment, and elevation 223.4 m at the west abutment).

Footings bearing on stiff to very stiff native clayey silt at these levels may be designed using the following resistance values:

Factored Geotechnical Resistance at ULS	300 kPa
Geotechnical Resistance at SLS	200 kPa

The SLS value is based on a maximum footing width of 2.5 to 4.0 m and a total settlement not exceeding 25 mm due to the applied footing load. Additional settlement of the footings may result from compression of the clayey silt under the load applied by the new approach fill. Discussion regarding settlement of the structure and approaches is presented in Section 10.

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 clayey silt may be computed using an unfactored friction coefficient of 0.35. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.1.2 Spread Footings on Engineered Fill

Construction of an engineered fill pad over the native soils to support spread footings was also considered as a measure to increase the geotechnical resistance available for design of the footings. However, similar to footings on native soil, the design must consider the potential for long-term consolidation settlement of the foundation soils under the new approach embankment loading. Therefore, spread footings constructed on engineered fill are not recommended and this option has not been developed further.

8.2 Driven Steel Piles

The geotechnical conditions encountered at this site are considered suitable for driven steel H-pile foundations. The piles must be driven into the hard clayey silt till or very dense sand encountered below approximate elevation 197 m. Preliminary design of the piles should be carried out on the basis of the axial geotechnical resistances given in Table 8.1.

Table 8.1 – Pile Geotechnical Resistance

Pile Section	ULS (Factored)	SLS	Estimated Pile Tip Elevation	
			W. Abutment	E. Abutment
HP 310 X 110	1,600 kN	1,400 kN	195	194
HP 360 X 132	1,800 kN	1,600 kN		

The pile tip elevations are presented for estimating purposes only. Additional drilling will be required during the detailed design stage to confirm the anticipated pile lengths at each foundation unit.

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

As the piles will be driven into hard till or very dense sand, the tips of all piles should be fitted with driving shoes as per OPSD 3000.100.

Downdrag forces will develop along the length of pile embedded in the clayey silt deposit if the piles are installed prior to completion of consolidation settlements in the foundation soils under the new approach embankments. For preliminary purposes, an unfactored downdrag force of 500 kN per pile is recommended. The downdrag loads must be confirmed during detail design after additional boreholes are drilled. An alternative is to construct surcharged approach embankments under an advance contract and wait for completion of the consolidation settlement prior to installing the piles.

The ground conditions at this site are considered suitable for an integral abutment design. To provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures. After the pile is driven, the space between the pile and the CSP should be filled with sand.

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:

$$k_s = 67 \cdot S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 \cdot S_u \text{ (kPa) at and below a depth of } 3 \cdot D \text{ (m) reduced to zero at the ground surface}$$

where D = pile width in metres

$$S_u = \text{undrained shear strength (kPa)}$$

$$= 125 \text{ kPa}$$

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, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), 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 on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 160 kN at ULS and 65 kN at SLS.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing. Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

For conventional abutments, the lateral resistance may be provided by battered piles.

8.3 Drilled Shafts (Caissons)

Drilled shaft foundations founded on the hard clayey silt till or very dense sand below elevation 197 m may be considered for the support of structural loads at this site. Based on the preliminary subsurface information collected however, the use of caissons may present some design and constructability issues, and is not a favourable option. Potential issues to be addressed include the following:

- To achieve the high resistances required to make caisson foundations practical, the caissons must be extended to the hard clayey silt till or very dense sand below elevation 197 m and designed on the basis of end-bearing resistance. However, entry into the caisson shaft to hand-clean and inspect the caisson base is typically avoided due to safety concerns, and therefore the competency of the founding surface cannot be verified.
- The groundwater level information indicates that an unbalanced hydraulic head and possibly an artesian condition may exist within the deep sand deposit encountered at the west abutment. Significant groundwater inflow and instability of the base and sidewalls of the excavation may be experienced when the caisson is extended into this deposit.
- Construction of caissons with a base terminated above the sand deposit and basing the design on skin friction developed along the caisson shaft is not recommended in view of potential consolidation settlements that may occur in the clayey silt under the new approach embankment loads.
- Downdrag forces will develop along the length of caisson embedded in the clayey silt deposit if the caissons are installed prior to completion of consolidation settlements in the foundation soils under the new approach embankments.

Based on these considerations, the use of drilled shaft (caisson) foundations is not recommended at this site, and this option has not been developed further.

8.4 Recommended Foundation

From a geotechnical perspective, the recommended foundation system for both abutments and the pier at this site is steel H-piles driven into hard clayey silt till and very dense sand near elevation 194 to 195 m.

8.5 Abutment Type

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments. The recommended foundation system of H-piles makes integral abutments a feasible option.

8.6 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.4 m. It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation.

9 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

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

γ = 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 abutment wall are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficient (K)

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

* For wing walls.

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

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.

10 APPROACH EMBANKMENTS

It is anticipated that the approach embankments will be in the order of 6 to 9 m in height. The foundation soils governing stability of the approach embankments consist of native firm to very stiff clayey silt. All topsoil, organic soils and poor quality existing fill should be stripped from the footprint of the approach fills. Mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments exceeding 8 m in height. The bench should maintain a 2% slope to shed surface run-off.

Preliminary stability analyses were carried out for earth fill embankments under static loading conditions. The stability analyses were carried out using the commercially available slope stability program GSLOPE developed by Mitre Software Inc. Bishop's modified method of slices was used for the limit equilibrium analyses. Based on consideration of the risk involved and past experience with highway embankment design/monitoring, the minimum factors of safety considered appropriate to achieve stability are 1.3 for short-term (undrained) stability and 1.5 for long-term (effective stress) stability of embankments founded on cohesive foundation soils.

Two slope geometries were assessed: a slope with a maximum height of 8 m inclined at 2H:1V, and a slope with a height of 9 m inclined at 2H:1V and incorporating a 2 m wide mid-height bench. The results are shown in Figures 1 to 4 in Appendix D. The results indicate that the factor of safety for a 6 to 9 m high embankment will be above 1.3 for short-term conditions, and approximately 1.5 for long-term conditions. Therefore, measures to improve the stability of the approach embankments should not be necessary.

Further investigation and stability analyses during detailed design, including seismic analysis, is required to more closely define the shear strength profile of the clayey silt deposit and provide detailed recommendations for embankment design.

Construction of the approaches is expected to result in long-term consolidation settlement of the thick clayey silt deposit. Based on the preliminary data, consolidation settlement is anticipated to be in the order of 225 mm under a new 6 m high embankment and 350 mm under a 9 m high embankment. Consolidation will occur over an extended period of 40 to 45 months before substantially (98%) completed.

Methods of increasing the rate and/or reducing the magnitude of post-construction settlement, such as advance embankment construction, surcharging, wick drains or EPS, will be required. These options should be further investigated during detailed design to select an appropriate method based on design tolerances, schedule and constructability.

11 CONSTRUCTION CONCERNS

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

- The surficial soils at the site are susceptible to disturbance by construction traffic and particular attention/measures will be required to provide a stable trafficable base for movement of heavy equipment.
- Piles driven to the hard clayey silt and very dense sand may achieve the required geotechnical resistance at varying elevations.
- The cohesionless sand at depth would be susceptible to disturbance under conditions of unbalanced hydrostatic head. If caissons are employed, temporary liners should be installed to support caisson sidewalls and provide seepage cut-off where required.
- The thick clayey silt deposit at this site may undergo significant long-term consolidation settlements under the loading of the new foundations and approach fill. The construction schedule and procedures must be specifically adapted to limit post-construction settlement of the structure and approaches.

12 SCOPE OF DETAILED INVESTIGATION

As design progresses, additional geotechnical investigation and analysis will be required to prepare geotechnical design recommendations for the detail design phase of the project. The recommended scope of work for detail design should include the following:

1. The subsurface conditions must be confirmed at the locations of all proposed foundation units, including the pier where no boreholes were drilled during the preliminary investigation. For deep foundations, this is typically accomplished by drilling two sampled boreholes within each new abutment and pier footprint. The boreholes should be extended at least 3 m below refusal defined by SPT test values exceeding 100 blows/0.3 penetration.
2. Additional boreholes should be drilled under the footprint of the proposed 6 to 9 m high approach embankments. Additional shear strength testing should be conducted in the clayey silt to confirm the embankment stability and assess embankment construction procedures. This should include in situ vane tests using an MTO vane.
3. Additional laboratory consolidation testing should be completed on undisturbed samples of the clayey silt from the approach boreholes to provide a profile of the compression characteristics of the clayey silt. Detailed settlement analysis must be carried out during detailed design to further evaluate the magnitude and duration of consolidation settlements to be expected under the embankment loads.
4. Methods of increasing the rate and/or reducing the magnitude of post-construction settlement, such as advance embankment construction, surcharging, wick drains or EPS,

should be further investigated during detailed design to select an appropriate method based on design tolerances, schedule and constructability.

5. A detailed monitoring program should be developed to measure actual settlement of the approach embankments during construction, to confirm the rate and magnitude of settlement, verify the performance of the preloading, surcharging and/or wick drain works, and ascertain when settlement has progressed sufficiently prior to further construction. Details of the monitoring program should include specifications for installation of settlement rods and monitoring points, as well as establishment of settlement criteria and response instructions.

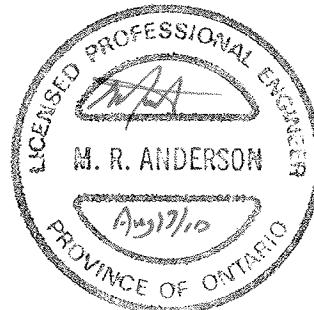
The geotechnical design recommendations must address the issues normally included in an MTO Foundation Investigation and Design Report for Detail Design.

13 CLOSURE

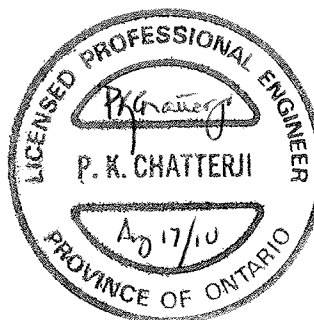
Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer



P.K. Chatterji, P.Eng., Ph.D.
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 ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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



Water Level

C_{pen}


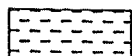



Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. $(W_L < 30\%)$.
		CI	Inorganic clays of medium plasticity, silty clays. $(30\% < W_L < 50\%)$.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
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

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No BH10-01

1 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 710.7 E 294 841.6 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.03.24 - 2010.03.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
225.5								20 40 60 80 100								
0.0	TOPSOIL, with roots and rootlets															
0.2	Clayey SILT, trace to some sand, trace gravel Firm to very stiff Brown Moist (CL-CI)		1	SS	4		225									
			2	SS	15		224									0 1 56 43
			3	SS	21											
			4	SS	19		223									
			5	SS	19		222									0 1 45 54
			6	SS	16											
			7	SS	9		221									
			1	TW			220									
			8	SS	10		219									
			9	SS	6		217									
			10	SS	6		216									
	Grey															
			</													

Continued Next Page

+³, x³: Numbers refer to Sensitivity 20 15 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-01

2 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 710.7 E 294 841.6 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.03.24 - 2010.03.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE			● QUICK TRIAXIAL × LAB VANE				
								20	40	60	80	100	20		
Continued From Previous Page															
	Clayey SILT , trace to some sand, trace gravel Firm to very stiff Grey Moist (CL)		11	SS	9		215								
							214								
			2	TW			213								
							212								
			12	SS	10		211								
							210								
			13	SS	9		209								
							208								
			14	SS	20		207								
							206								
			15	SS	27										

Continued Next Page

+³ . X³ : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-01

3 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 710.7 E 294 841.6 ORIGINATED BY SL
HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
DATUM Geodetic DATE 2010.03.24 - 2010.03.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
Continued From Previous Page													
	Clayey SILT, trace to some sand, trace gravel Firm to very stiff Grey Moist to wet (CL)		16	SS	14		205						
			17	SS	28		204						
			18	SS	11		203						
	Clayey SILT, some sand seams, trace gravel, occasional cobbles Hard Grey Moist (TILL)(CL)						202						
							201						
							200						
197.1							199						0 1 56 43
28.4							198						
							197						
							196						3 18 49 30

Continued Next Page

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

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RECORD OF BOREHOLE No BH10-01

4 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 710.7 E 294 841.6 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.03.24 - 2010.03.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L —○—							
							WATER CONTENT (%) 20 40 60							
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							
							20 40 60 80 100							
193.1	Continued From Previous Page Clayey SILT, some sand seams, trace gravel, occasional cobbles Hard Grey Moist (TILL)(CL)		20	SS	100/25		195 							

+³ X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

1 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 654.2 E 294 774.3 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.04.05 - 2010.04.08 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W P W W L	WATER CONTENT (%)	GR SA SI CL		
224.6													
0.0	TOPSOIL, some roots and rootlets												
0.2	Clayey SILT, trace sand, trace gravel Firm to very stiff Brown Moist (CL)		1	SS	9		224						
			2	SS	15		223						
			3	SS	10		222						0 0 60 40
			4	SS	11		221						
	Grey		5	SS	10		220						
			6	SS	6		219						
			7	SS	8		218						1 3 52 44
			1	TW			217						
	Soft and wet at 6.9 m		8	SS	3		216						
			9	SS	10		215						0 13 47 40
			2	TW									
			10	SS	10								

Continued Next Page

+³ X³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

2 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 654.2 E 294 774.3 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.04.05 - 2010.04.08 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE		WATER CONTENT (%) w _p w w _L				
Continued From Previous Page														
	Clayey SILT, trace sand, trace gravel Firm to very stiff Grey Moist (CL)													
			11	SS	12		214							
							213							
			12	SS	4		212							
							211							
			13	SS	5		210							
							209							
							208							
			14	SS	19		207							
							206							
			15	SS	16		205							
			16	SS	17									

Continued Next Page

+³, X³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

3 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 654.2 E 294 774.3 ORIGINATED BY SL
HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
DATUM Geodetic DATE 2010.04.05 - 2010.04.08 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE						● QUICK TRIAXIAL	×	LAB VANE
	Continued From Previous Page		17	SS	23		204											
	Clayey SILT, trace sand, trace gravel Firm to very stiff Grey Moist (CL)						203											
							202											
			18	SS	8		201											
							200											
							199											
198.5																		
26.1	Clayey SILT, some sand, trace gravel Hard Grey Moist (TILL)		19	SS	37		198											
197.0							197											
27.6	SAND, trace to some silt Very Dense Grey Moist						196											
			20	SS	100 / 0.23		195											

Continued Next Page


+³ . X³ : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

4 OF 4

METRIC

G.W.P. 19-1351-166 LOCATION N 4 881 654.2 E 294 774.3 ORIGINATED BY SL
 HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB
 DATUM Geodetic DATE 2010.04.05 - 2010.04.08 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20 40 60 80 100										
	Continued From Previous Page																	
	SAND, trace to some silt Very Dense Grey Moist		21	SS	100 / 0.20		194										0 89 11 (SI+CL)	
								193										
192.2				22	SS	100 / 0.28												
32.4	END OF BOREHOLE AT 32.4 m Piezometer installation consists of 25 mm diameter Schedule 40 PVC pipe with a 2.1 m slotted screen WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2010.04.09 9.06 215.5 2010.04.20 1.46 223.1 2010.05.03 1.00 223.6																	

ONTMT4S 1166.GPJ 6/17/10

ON MOT D01-1151.GPJ ON MOT.GDT 7/12/01

+ 3, X 3. Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

PROJECT 001-1151		RECORD OF BOREHOLE No B2-1		2 OF 2	METRIC
W.P. 40-00-00	LOCATION N 4891670.2 E 254821.5	ORIGINATED BY FKS			
DIST SW HWY 400	BOREHOLE TYPE 106mm Diameter Solid Stem Augers	COMPILED BY JCL			
DATUM Geodetic	DATE December 6-7, 2000	CHECKED BY ASP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIMIT MOISTURE CONTENT		UNIT WEIGHT γ	REMARKS S GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100		12 20 30			
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOLDED		WATER CONTENT (%)			
	— CONTINUED FROM PREVIOUS PAGE —												GR SA SI CL
	Clayey Sil. trace to some sand trace gravel (T46) Stiff to hard Grey Moist						204						
			16	SS	30		203						
							202						
							201						
			17	SS	37		200						
							199						
							198						
197.1													
27.6	Clayey Sil. trace sand and gravel (T46) Hard Grey Dry to moist		18	SS	85		197						
							196						
							195						
			19	SS	100/15		194						
							193						
192.7													
52.0	Clayey Sil. with sand, trace gravel (T46) Hard Grey Moist						192						
			20	SS	75/15		191						
190.9													
33.8	END OF BOREHOLE												
<p>Notes:</p> <p>1. The water level in the open borehole during drilling operations was at about 15-m depth (Elev 209.7m)</p> <p>2. The water in the piezometer was frozen at ground surface (Elev 224.7m) on January 18, 2001 and March 20, 2001. The water level in the piezometer was measured at 0.8m above ground surface (Elev 225.5m) on June 19, 2001</p>													

ON MOT 001-1151 GPJ ON MOT GDT 7/12/01

Continued Next Page

✱ ✱ ✱ ✱

Numbers refer to
Sensitivity

Q^{3*} STRAIN AT FAILURE

ON MOT 001-1151.GPJ ON MOT GOT 1/12/01

PROJECT 001-1151			RECORD OF BOREHOLE No B2-2			2 OF 2		METRIC				
W.P. 40-00-00			LOCATION N 4681613.9; E 294800.0			ORIGINATED BY PKS						
DIST SW HWY 400			BOREHOLE TYPE 100mm Diameter Solid Stem Augers			COMPILED BY LCC						
DATUM Geodetic			DATE December 11-14, 2000			CHECKED BY ASP						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	W _p	W _n			W _L
— CONTINUED FROM PREVIOUS PAGE —												
197.7	Clayey Silt, trace sand, trace gravel (Till) Silt to hard Brown to grey Moist		16	SS	30		205					
							204					
							203					
							202					
							201					
196.1	Clayey Silt with sand and gravel (Till) Hard Grey Moist		17	SS	28		200					
							199					
27.4	Silty Sand Very dense Grey Moist		18	SS	200-15		198					
							197					
194.2	END OF BOREHOLE		19	SS	185-23		196					
30.9							195					
<p>Note: The groundwater level in the piezometer was measured at 1.1m depth (Elev. 224.0m) on January 18, 2001, at 0.5m depth (Elev. 224.8m) on March 20, 2001, and at 0.8m depth (Elev. 224.2m) on June 19, 2001.</p>												

ON MOT 001-1151.GPJ ON MOT.GDT 7/12/01

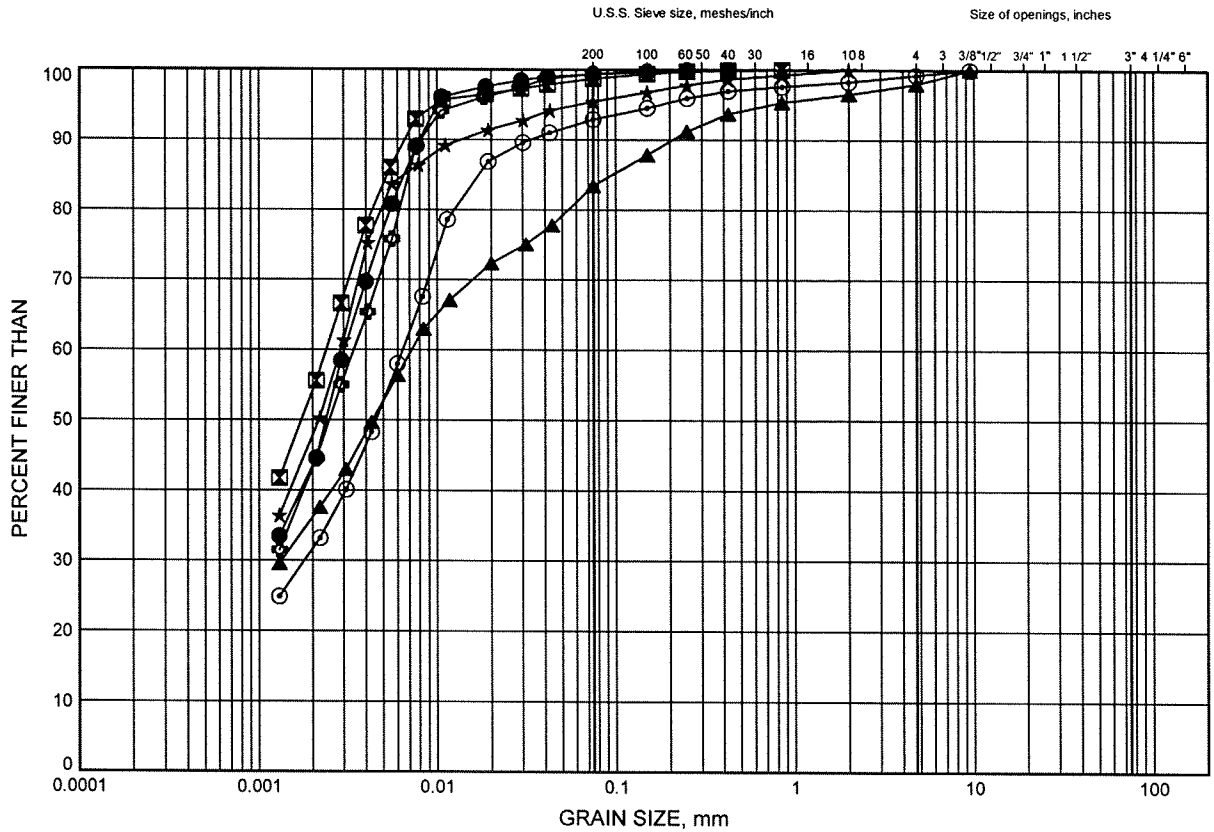
Appendix B

Laboratory Test Results

Highway 400 and 5th Line Underpass GRAIN SIZE DISTRIBUTION

FIGURE B1

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-01	1.07	224.43
⊠	BH10-01	3.35	222.15
▲	BH10-01	7.16	218.34
★	BH10-01	12.50	213.00
⊙	BH10-01	17.07	208.43
⊛	BH10-01	26.21	199.29

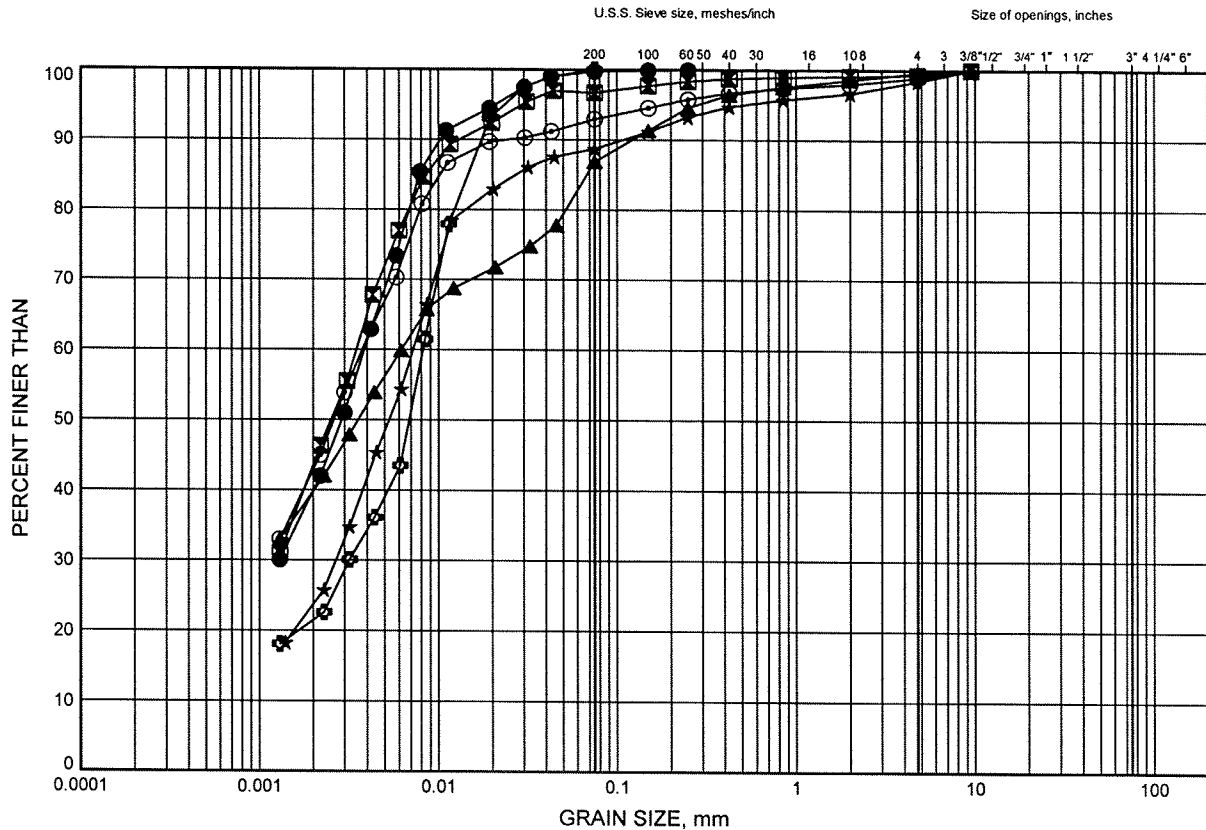


W.P.# 19-1351-166
Prepared By AN
Checked By AEG

Highway 400 and 5th Line Underpass GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-02	1.83	222.77
⊠	BH10-02	6.40	218.20
▲	BH10-02	9.45	215.15
★	BH10-02	14.02	210.58
⊙	BH10-02	18.59	206.01
⊛	BH10-02	23.16	201.44

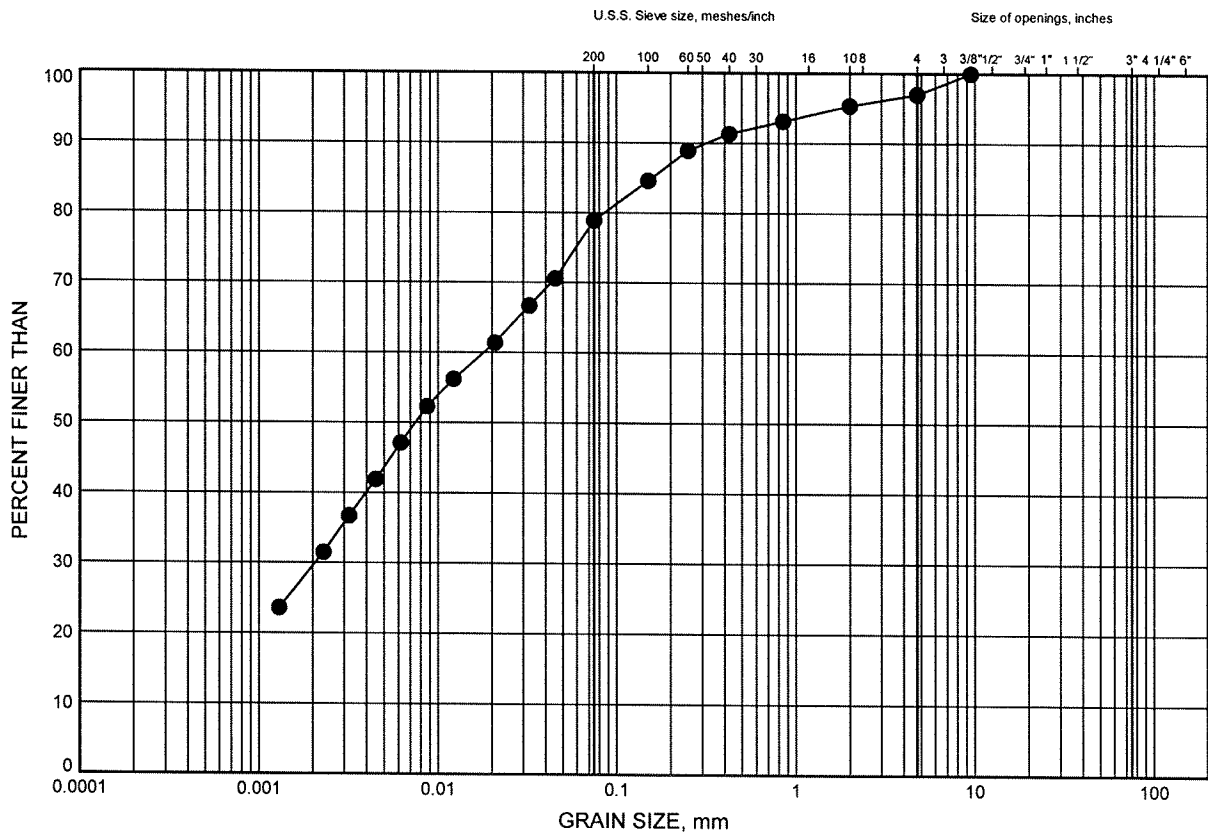


W.P.# 19-1351-166
Prepared By AN
Checked By AEG

Highway 400 and 5th Line Underpass GRAIN SIZE DISTRIBUTION

FIGURE B3

CLAYEY SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-01	29.18	196.32

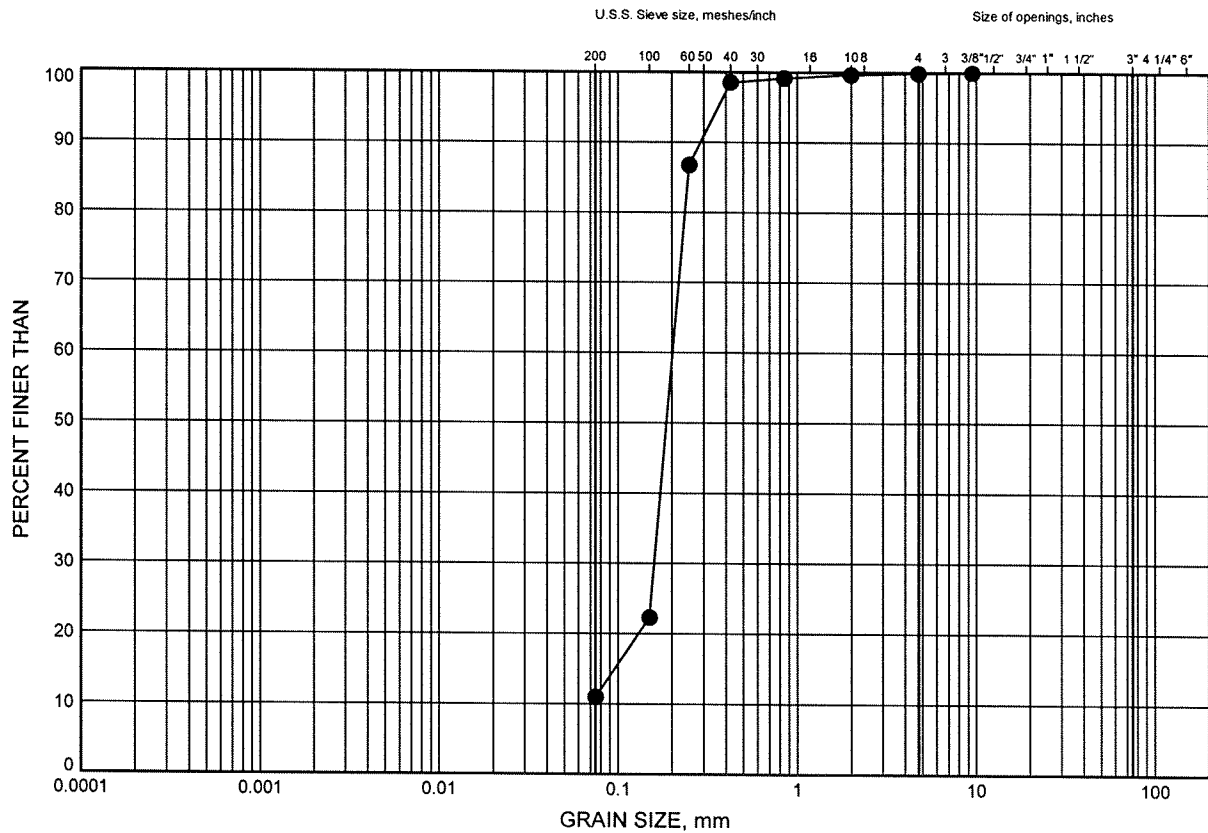


W.P.# 19-1351-166
Prepared By AN
Checked By AEG

Highway 400 and 5th Line Underpass GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-02	30.66	193.94

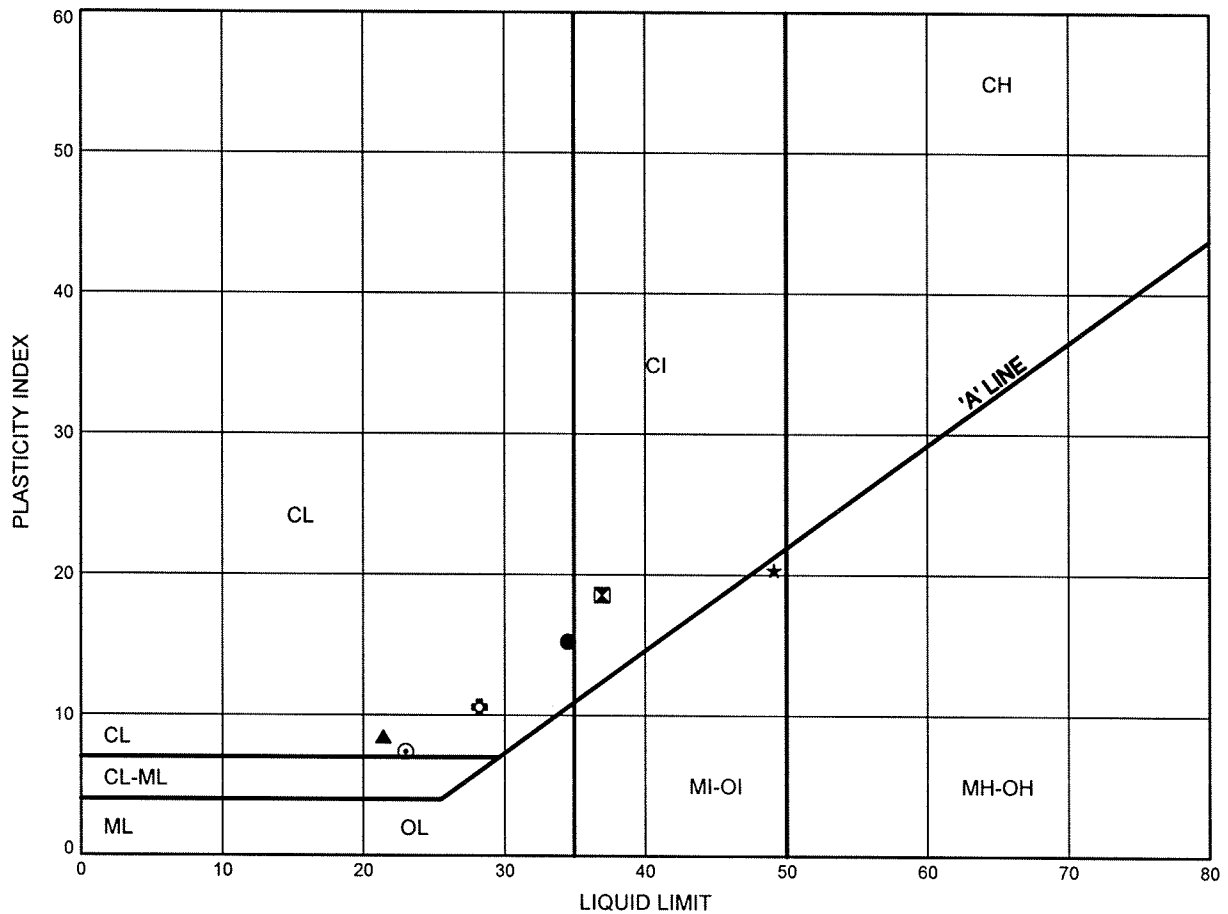


W.P.# 19-1351-166
Prepared By AN
Checked By AEG

Highway 400 and 5th Line Underpass ATTERBERG LIMITS TEST RESULTS

FIGURE B5

CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH10-01	1.07	224.43
⊠	BH10-01	3.35	222.15
▲	BH10-01	7.16	218.34
★	BH10-01	12.50	213.00
⊙	BH10-01	17.07	208.43
⊕	BH10-01	26.21	199.29

Date June 2010
Project 19-1351-166

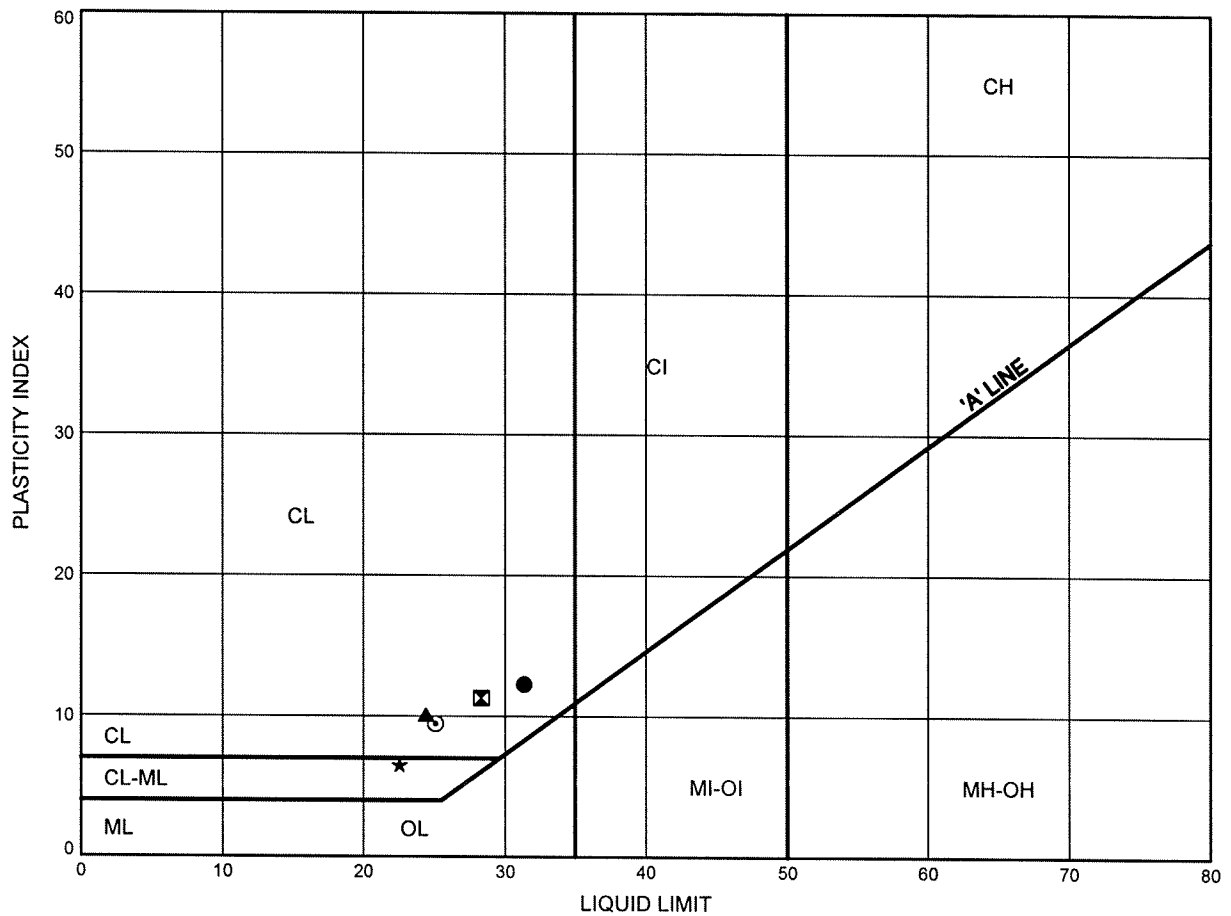


Prep'd AN
Chkd. AEG

Highway 400 and 5th Line Underpass
ATTERBERG LIMITS TEST RESULTS

FIGURE B6

CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH10-02	1.83	222.77
⊠	BH10-02	6.40	218.20
▲	BH10-02	9.45	215.15
★	BH10-02	14.02	210.58
⊙	BH10-02	18.59	206.01

Date June 2010
 Project 19-1351-166

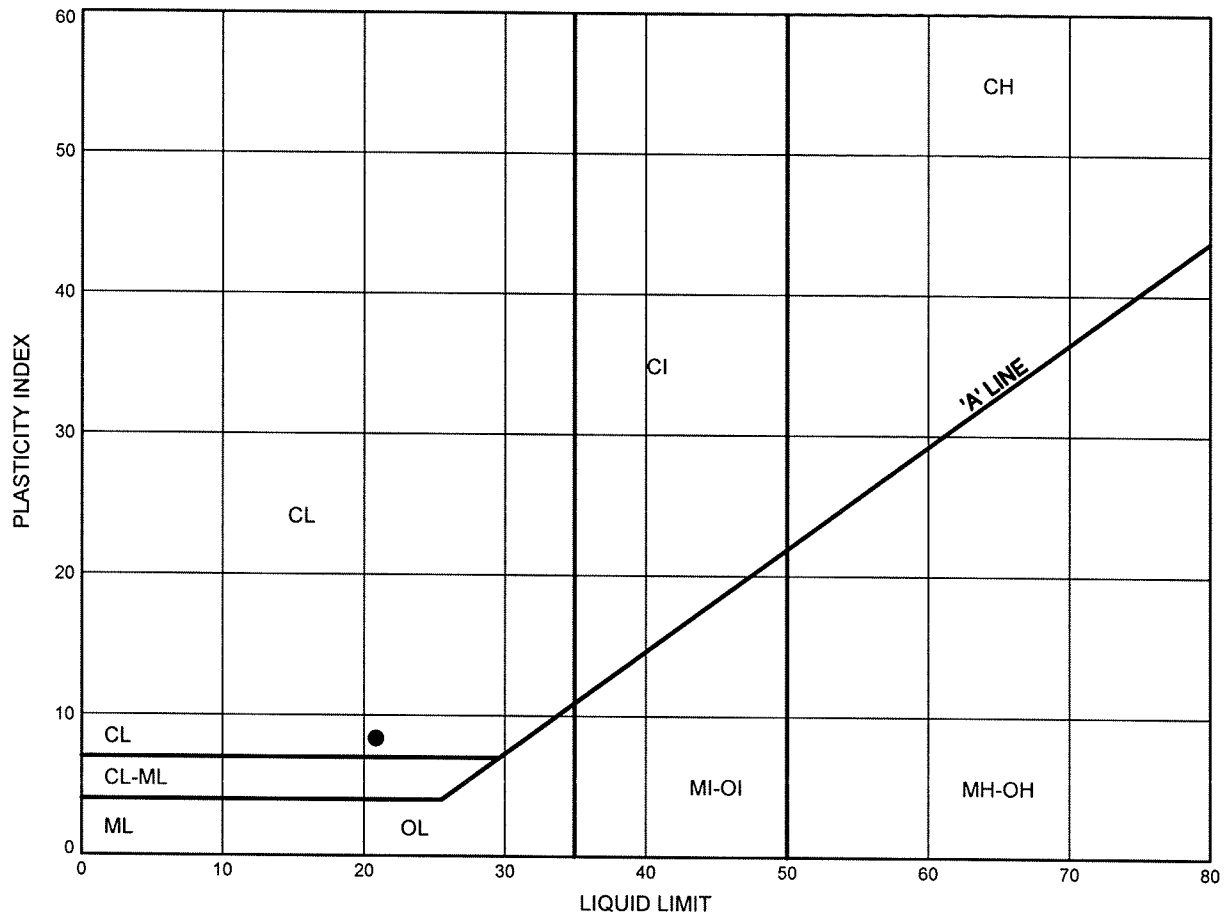


Prep'd AN
 Chkd. AEG

Highway 400 and 5th Line Underpass
ATTERBERG LIMITS TEST RESULTS

FIGURE B7

CLAYEY SILT TILL

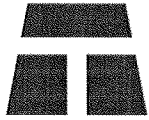


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH10-01	29.18	196.32

Date June 2010
 Project 19-1351-166



Prep'd AN
 Chkd. AEG



Consolidation Test Report

CLIENT: McCormick Rankin Corporation

FILE NUMBER: 19-1351-166

PROJECT: Highway 400 & 5th Line EAS

REPORT DATE: 13-Apr-10

TEST DATES: March 30, 2010 - April 11, 2010

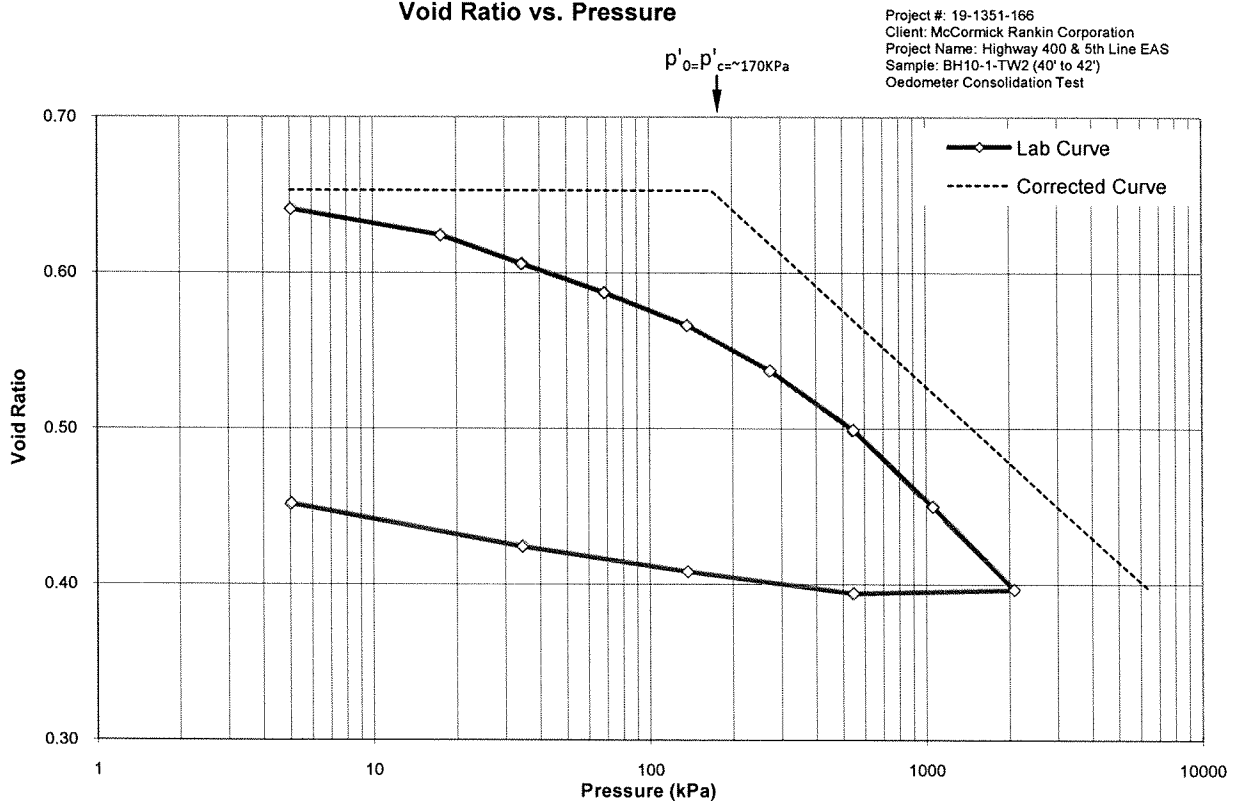
SAMPLE: BH10-1-TW2 (40' to 42')
Silty Clay, trace Gravel, grey, Grain Size: 47% Clay, 48 % Silt, 5% Sand
Atterberg Limits: LL=49.2%, PI=20.4%

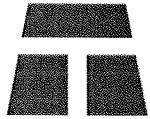
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

	Start of Test	End of Test
Wet Dens. (kg/m ³)	2035.1	2238.1
Dry Dens. (kg/m ³)	1639.6	1867.3
Moisture Cont. (%)	24.1	19.9
Void Ratio	0.653	0.451
Saturation (%)	100.0	

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

Void Ratio vs. Pressure





Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

BH10-1-TW2 (40' to 42')

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

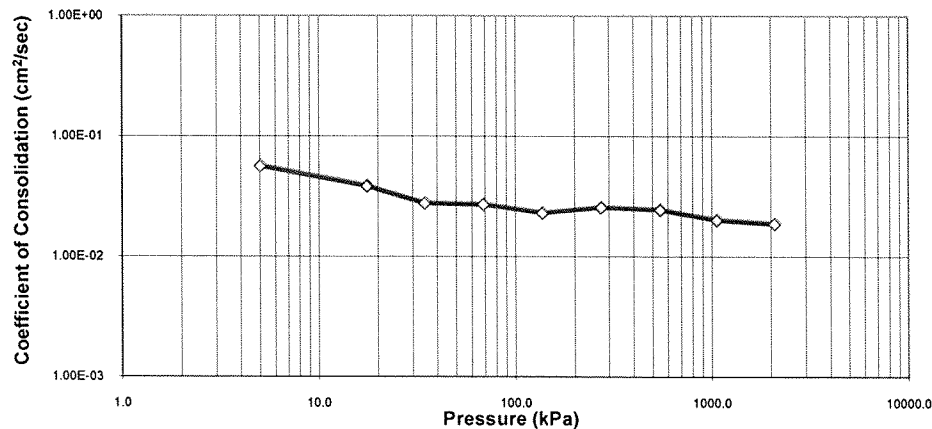
LOADING: A seating load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

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

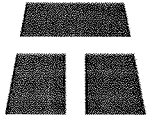
Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d ₉₀ (mm)	t ₉₀ (min)	c _v (cm ² /s)	Void Ratio	m _v (m ² /kN)	k (cm/s)
0.0	20.000					0.653		
5.0	19.851	19.926	0.143	0.25	5.61E-02	0.641	1.48E-03	8.16E-06
17.6	19.648	19.750	0.142	0.36	3.83E-02	0.624	8.16E-04	3.06E-06
34.5	19.426	19.537	0.076	0.49	2.75E-02	0.606	6.66E-04	1.80E-06
68.5	19.202	19.314	0.097	0.49	2.69E-02	0.587	3.40E-04	8.96E-07
136.9	18.948	19.075	0.120	0.56	2.29E-02	0.566	1.93E-04	4.34E-07
273.2	18.597	18.773	0.125	0.49	2.54E-02	0.537	1.36E-04	3.39E-07
545.5	18.134	18.366	0.138	0.49	2.43E-02	0.499	9.14E-05	2.18E-07
1057.7	17.539	17.837	0.148	0.56	2.00E-02	0.450	6.40E-05	1.26E-07
2080.1	16.892	17.216	0.178	0.56	1.86E-02	0.396	3.61E-05	6.59E-08
545.5	16.865	16.879				0.394		
136.9	17.032	16.949				0.408		
34.5	17.228	17.130				0.424		
5.0	17.561	17.395				0.451		

Coefficient of Consolidation vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



Notes: C_v and k calculated using t₉₀ values



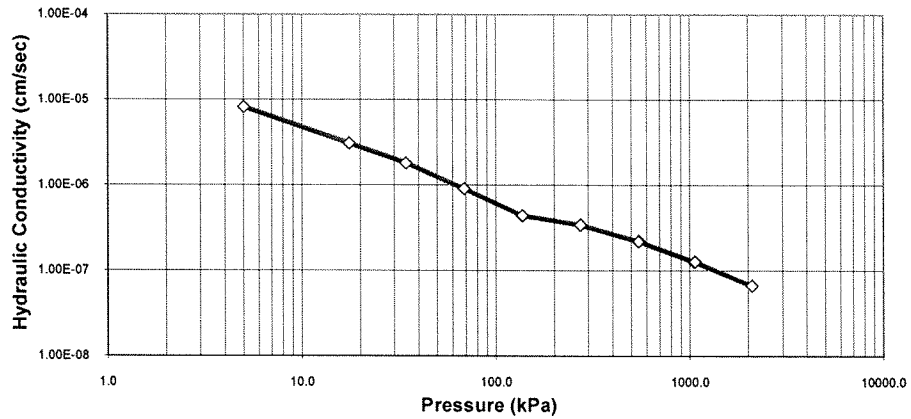
Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

BH10-1-TW2 (40' to 42')

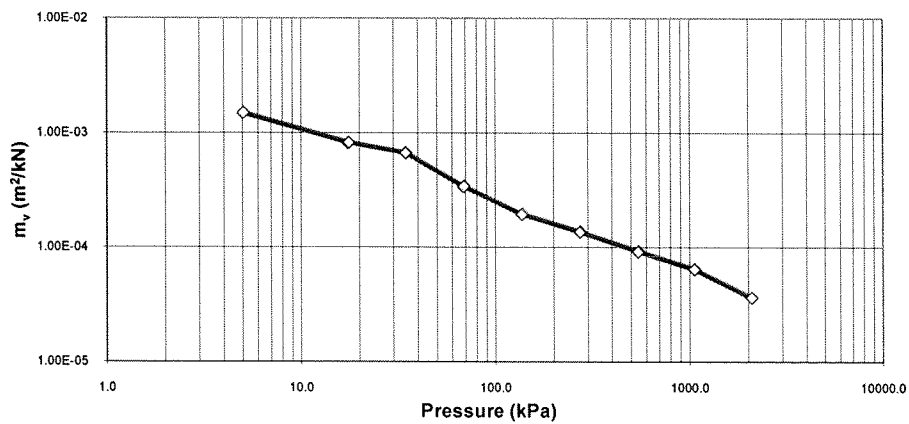
Hydraulic Conductivity vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



m_v vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



Consolidation Test Report

CLIENT: McCormick Rankin Corporation

FILE NUMBER: 19-1351-166

PROJECT: Highway 400 & 5th Line EAS

REPORT DATE: 30-Apr-10

TEST DATES: April 13, 2010 - April 25, 2010

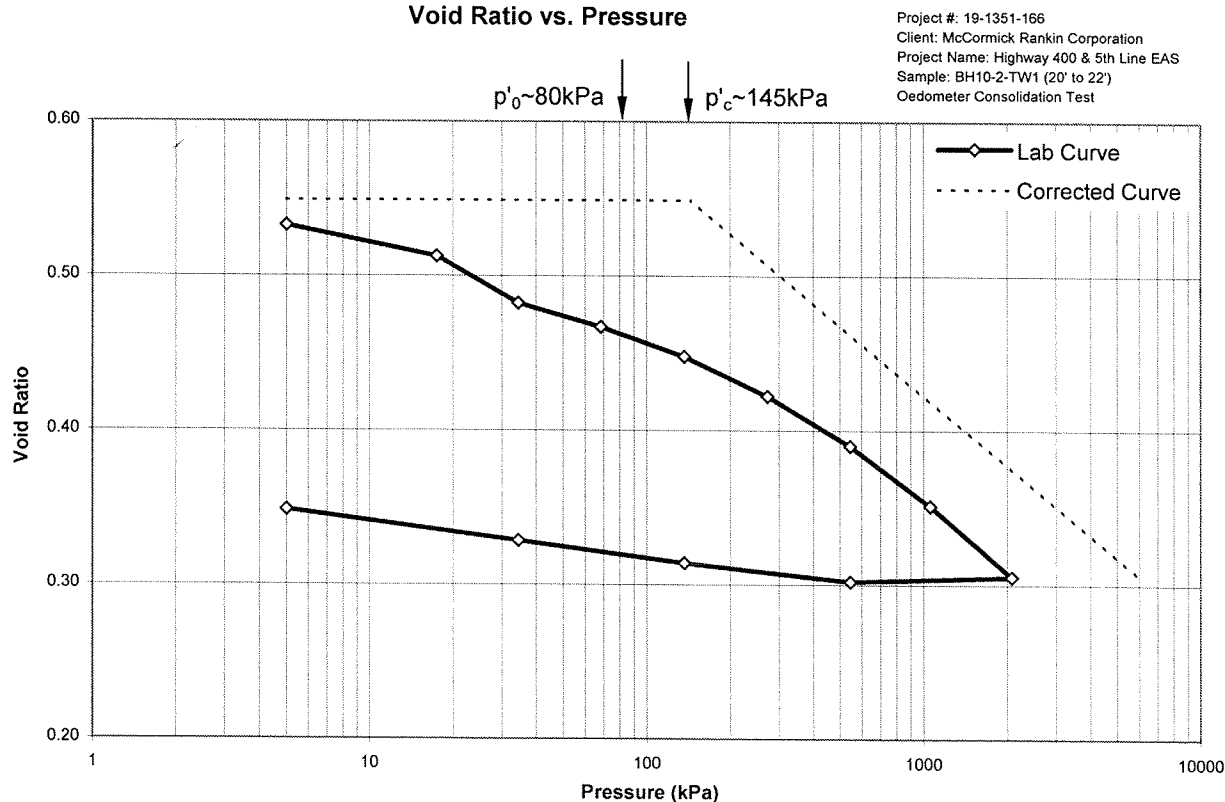
SAMPLE: BH10-2-TW1 (20' to 22')
Clay, Silty, trace Sand and Gravel, grey, (CL), Grain Size: 44 % Clay, 53% Silt, 3% Sand, 1% Gravel

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 ³)	2113.1	2393.3
Dry Dens. (kg/m ³)	1758.8	2020.2
Moisture Cont. (%)	20.1	18.5
Void Ratio	0.549	0.349
Saturation (%)	99.8%	

Note: A Specific Gravity of 2.73 was measured for the void ratio and saturation calculations.

Void Ratio vs. Pressure



Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

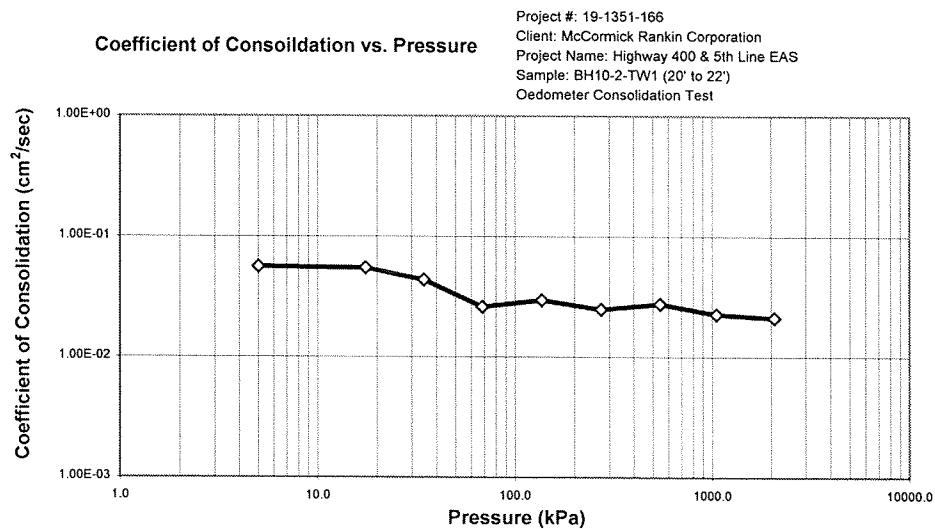
BH10-2-TW1 (20' to 22')

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

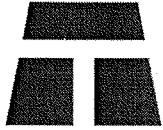
LOADING: A seating load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

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

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d_{90} (mm)	t_{90} (min)	c_v (cm ² /s)	Void Ratio	m_v (m ² /kN)	k (cm/s)
0.0	20.000					0.549		
5.0	19.790	19.895	0.255	0.25	5.59E-02	0.533	2.09E-03	1.15E-05
17.6	19.530	19.660	0.272	0.25	5.46E-02	0.513	1.05E-03	5.62E-06
34.5	19.140	19.335	0.274	0.30	4.37E-02	0.482	1.18E-03	5.04E-06
68.5	18.940	19.040	0.084	0.49	2.61E-02	0.467	3.08E-04	7.89E-07
136.9	18.695	18.818	0.123	0.42	2.96E-02	0.448	1.89E-04	5.50E-07
273.2	18.364	18.530	0.154	0.49	2.48E-02	0.422	1.30E-04	3.15E-07
545.5	17.949	18.157	0.153	0.42	2.76E-02	0.390	8.30E-05	2.24E-07
1057.7	17.446	17.698	0.177	0.49	2.26E-02	0.351	5.47E-05	1.21E-07
2080.1	16.852	17.149	0.161	0.49	2.12E-02	0.305	3.33E-05	6.93E-08
545.5	16.813	16.833				0.302		
136.9	16.965	16.889				0.314		
34.5	17.153	17.059				0.328		
5.0	17.412	17.283				0.349		



Notes: c_v and k calculated using t_{90} values



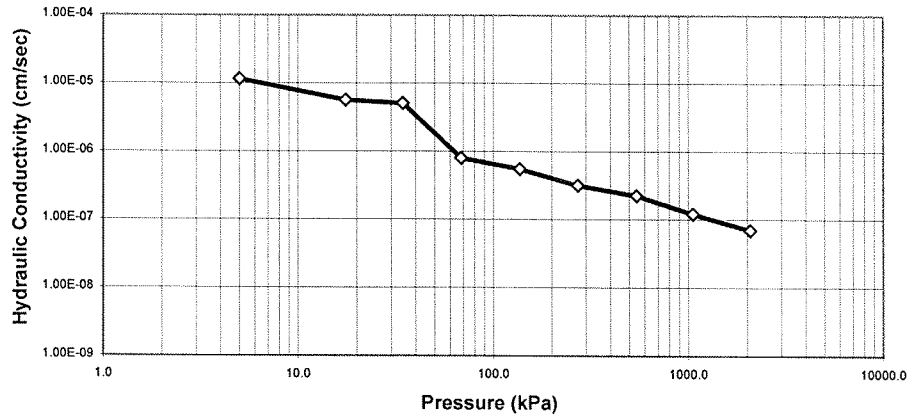
Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

BH10-2-TW1 (20' to 22')

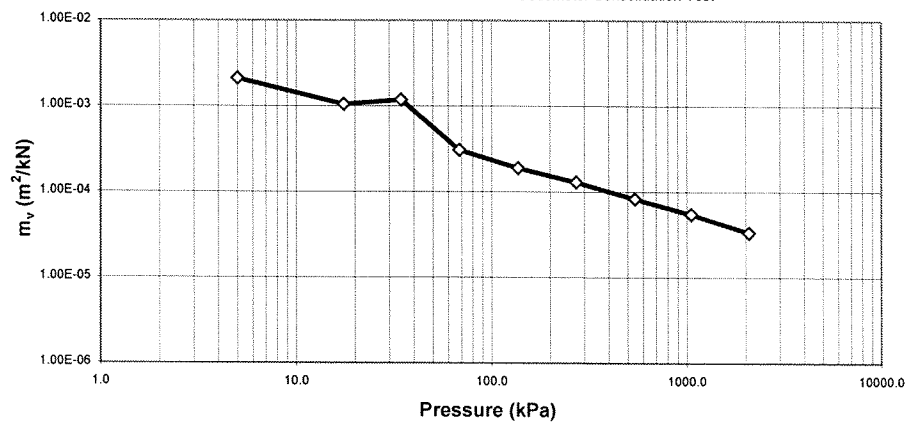
Hydraulic Conductivity vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-2-TW1 (20' to 22')
Oedometer Consolidation Test



m_v vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-2-TW1 (20' to 22')
Oedometer Consolidation Test



Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil	Footings on Engineered Fill	Driven Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Relatively low geotechnical resistance is available. ii. Potential for long-term settlement of foundation soils due to consolidation under approach fill loads. iii. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Provides higher geotechnical resistance than footings on native soil. ii. Generally less costly construction than deep foundation elements. iii. Allows use of perched abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement. ii. Potential for long-term settlement of foundation soils due to consolidation under approach fill loads. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance in hard or very dense soils. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Foundation construction may require less volume of excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Possibility of variable pile length. iii. Downdrag forces will develop on the length of pile embedded in the clayey silt deposit. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded in hard till or very dense sand. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Potential for high unbalanced hydraulic head or artesian condition in deep sand at west abutment. iii. More likely to encounter groundwater. Temporary liners may be required to install caissons. iv. Potential difficulty in cleaning and inspecting bases. v. Downdrag forces will develop on the length of pile embedded in the clayey silt deposit. <p>NOT RECOMMENDED</p>

Appendix D

Preliminary Stability Analyses

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Approach Fill	21	32	0
Clayey Silt	20	0	1

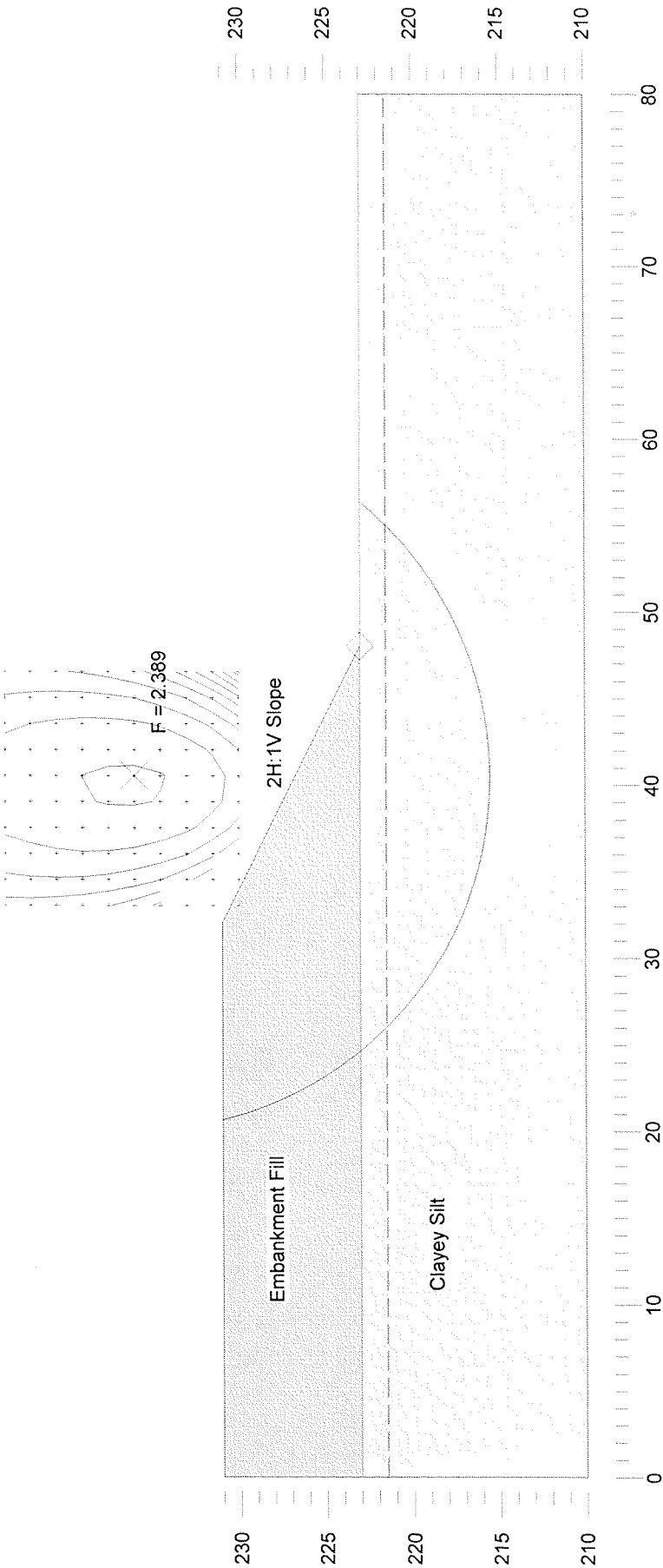


Fig. 1

Thurber Engineering Ltd. - Toronto
19-1351-166
Highway 400 & 5th Line
August 2010
8m High Approach Embankment
Long-term (Drained) Case

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Approach Fill	21	32	0
Clayey Silt	20	29	1

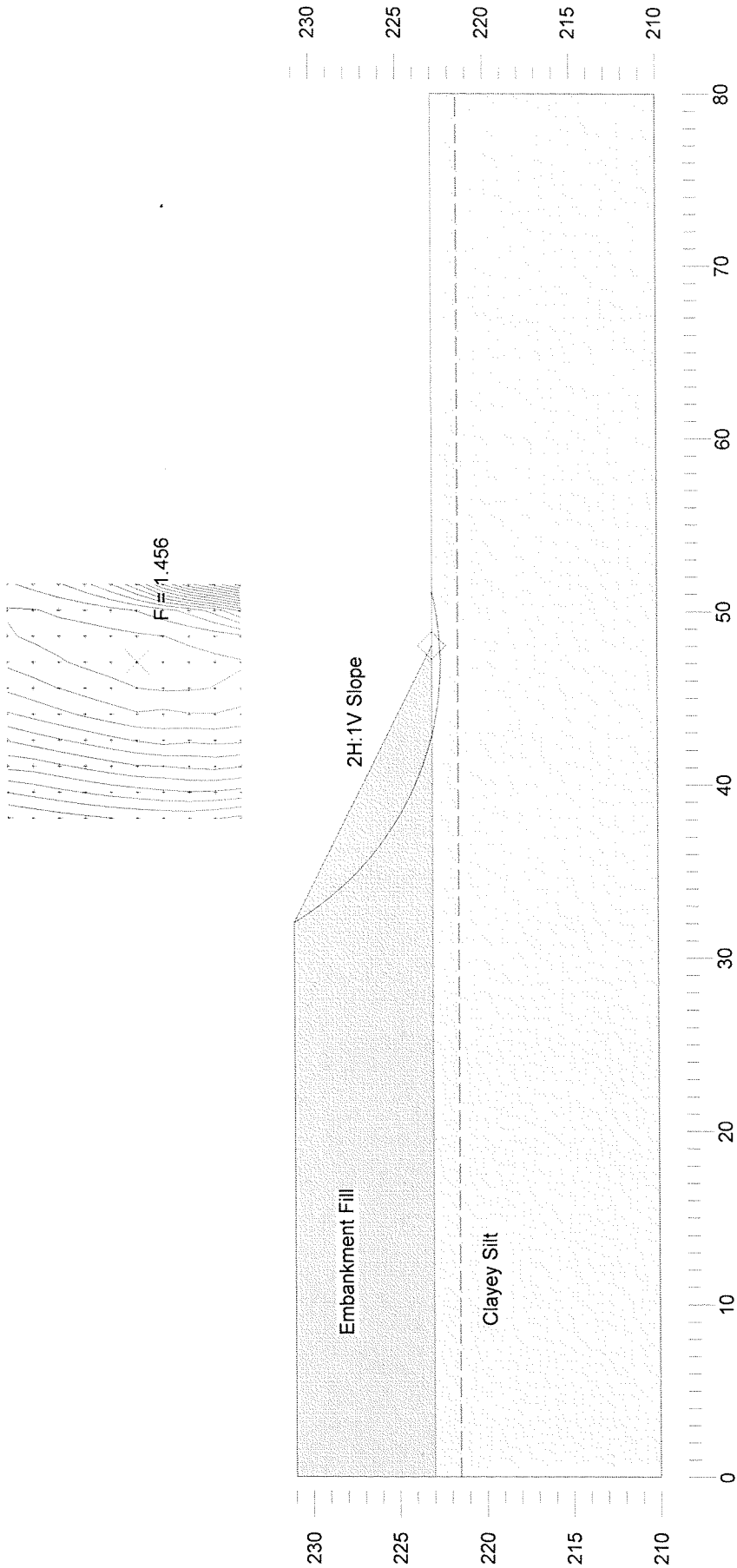


Fig. 2

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Approach Fill	21	0	32	0
Clayey Silt	20	75	0	1

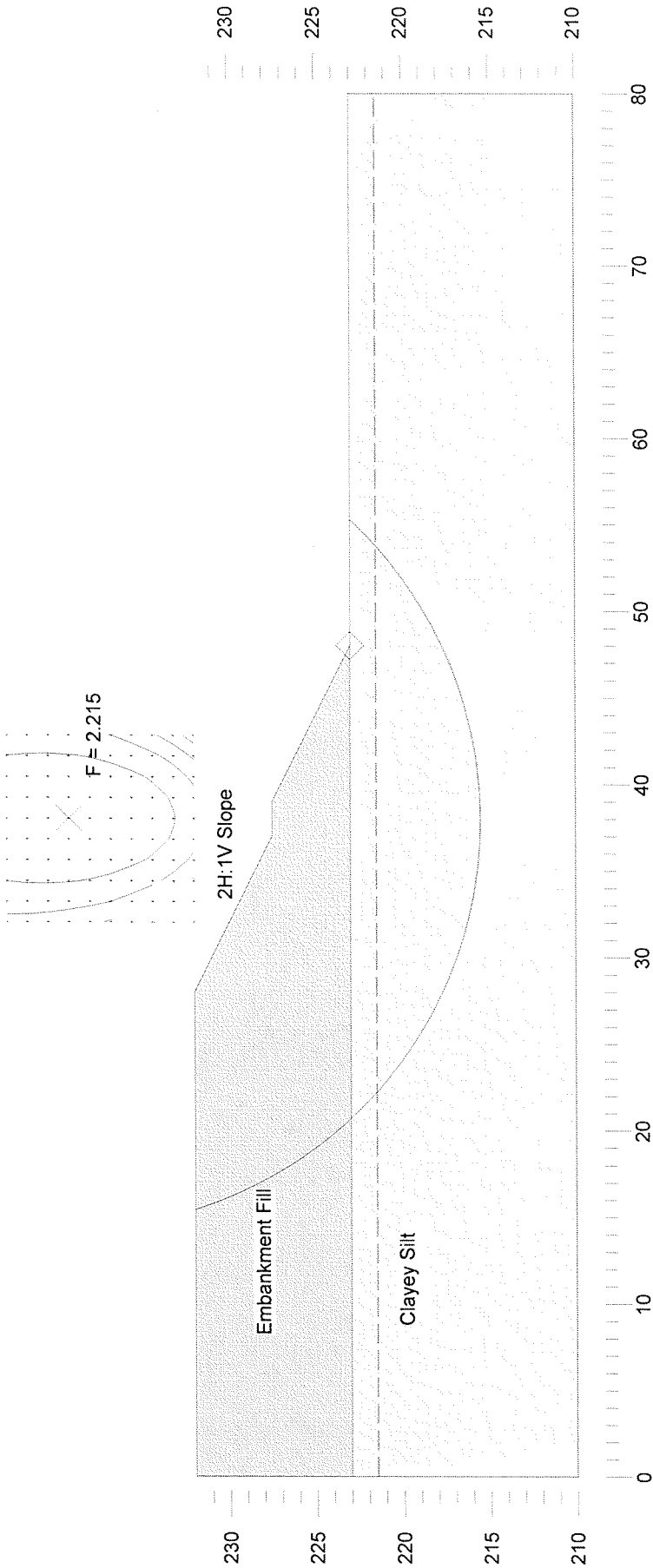


Fig. 3

Thurber Engineering Ltd. - Toronto
19-1351-166
Highway 400 & 5th Line
August 2010
9m High Approach Embankment
Long-term (Drained) Case

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Approach Fill	21	0	32	0
Clayey Silt	20	0	29	1

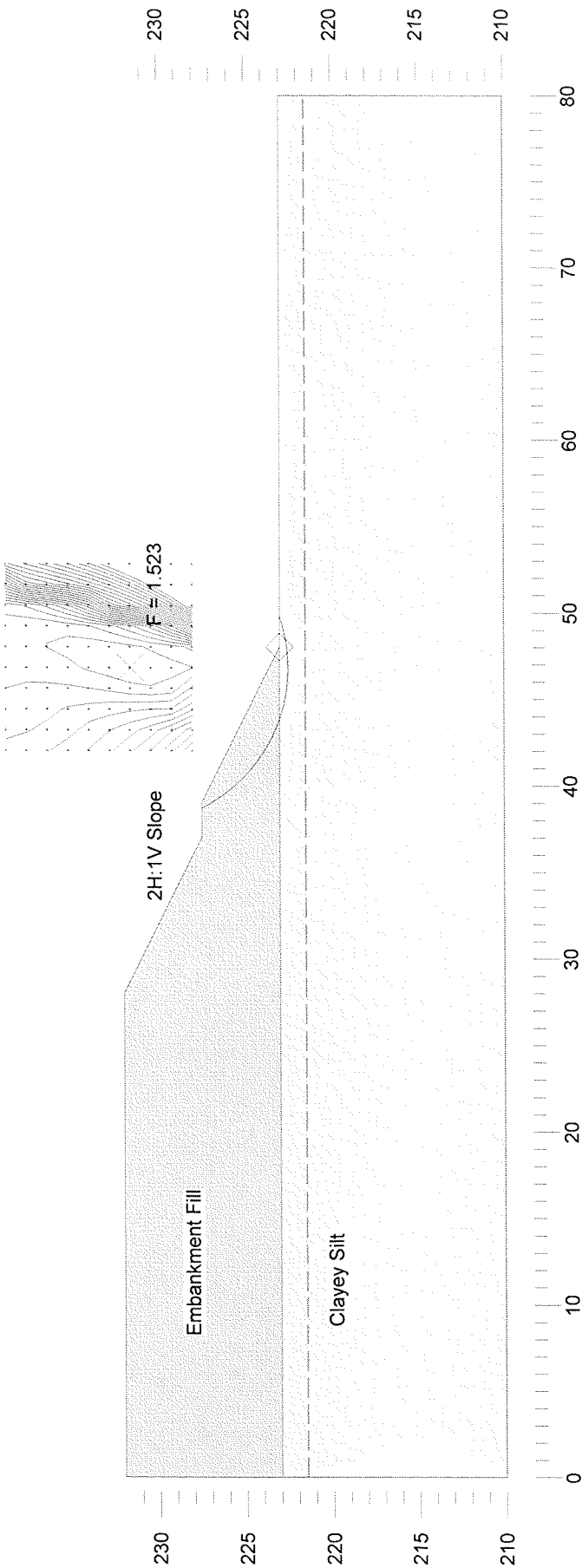


Fig. 4

Appendix E

Borehole Locations and Soil Strata Drawing

