

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 11 NORTHBOUND LANES OVER
TRANSCANADA PIPELINE
HIGHWAY 11, HIGHWAY 518 WEST to HIGHWAY 520
G.W.P. 480-93-00, W.P. 472-93-01, SITE 44-411**

Geocres Number: 31E-231

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed bridge to carry the Northbound Lanes of the widened Highway 11 over TransCanada Pipeline south of the village of Katrine, Ontario. A previous, preliminary investigation had been carried out at the site by AGRA Earth and Environmental Ltd. and the factual data from that investigation has been used in the preparation of this report.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed using the data from the previous AGRA investigation, along with a single borehole drilled during the current study. This model describes the geotechnical conditions influencing design and construction of the foundations and approach embankments for the bridge.

Thurber carried out the report preparation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000285.

2 SITE DESCRIPTION

The site lies at the location where the proposed northbound lanes of Highway 11 will cross the TransCanada Pipeline south of the Village of Katrine, Armour Township. The site lies approximately 250 m north of the existing Highway 11 intersection with Highway 592/Sunset Pass Drive.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. The area is generally wooded but there are cleared areas on both sides of the existing highway immediately south of the site.

There is a commercial development, including a store and a gas bar, on the west side of the highway approximately 250 m south of the site.

3 SITE INVESTIGATION AND FIELD TESTING

The initial site investigation and field testing for this project was carried out by AGRA between June 4 and June 13, 1999. Subsequently on February 10, 2005, Thurber drilled an additional borehole (BH 411-2) to obtain samples for supplementary laboratory testing.

The original site investigation consisted of drilling and sampling a total of four boreholes. At the approach fills, boreholes were advanced to depths of 6.6 m at the south and 14.2 m at the north. Boreholes were advanced to depths of 50.3 m at the south abutment and 46.9 m at the north abutment. For the current investigation, one borehole was drilled immediately adjacent to previous borehole TCPL1 at the north abutment, and terminated at 9.8 m depth.

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

A combination of solid stem auger, hollow stem auger and rotary drilling techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Tests (SPT). Thin wall Shelby tube samples were taken in the cohesive soils and the insitu strength was assessed using the MTO shear vane.

Where difficulty in penetrating cobbles and boulders was encountered, diamond coring techniques were employed to penetrate the obstructions.

The positions of the boreholes considered in the preparation of this report, relative to the structure site are as shown in Table 3.1.

Table 3.1 – Borehole Locations Relative to Structure

Location on Structure	Boreholes Considered in Design
South Approach	TCPL4
South Abutment	TCPL3
North Abutment	TCPL1, 411-2
North Approach	TCPL2

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.

Two standpipe piezometers were installed in each of the two abutment boreholes and one piezometer was installed in the north approach borehole to monitor the groundwater levels. The completion details of the piezometers are presented in Table 3.2.

Table 3.2 – Piezometer Details

Piezometer Location	Tip Position (m)		Completion Details
	Depth	Elevation	
TCPL1	3.0	298.5	Sand filter and screen from 3.0 to 1.0 m, bentonite seal to 0.8 m, grout/cuttings to surface.
	45.5	256.0	Sand filter and screen from 45.5 to 42.5 m, bentonite seals at 19.8 to 19.1, 16.7 to 16.1, and 3.6 to 3.0, grout/cuttings between seals and to surface.
TCPL2	13.4	286.7	Sand filter and screen from 13.4 to 10.4 m, grout/cuttings to 6.1, bentonite seal to 5.5 m, grout/cuttings to surface.
TCPL3	4.7	302.0	Sand filter and screen from 4.7 to 2.7 m, grout/cuttings to surface.
	43.2	263.5	Sand filter and screen from 43.2 to 40.2 m, grout/cuttings to surface.

4 LABORATORY TESTING

The results of laboratory testing are shown on the Record of Borehole sheets and are plotted in the figures in Appendix B. Moisture content, gradation and Atterberg limit tests were carried out on selected samples.

Samples were also selected for consolidation testing, unconsolidated, undrained triaxial and consolidated, undrained triaxial testing with pore pressure measurements.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in this appendix and on the attached Borehole Locations and Soil Strata Drawing. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy encountered at this site is consistent with that encountered in much of the Highway 11 corridor between Huntsville and North Bay. Glacial outwash soils deposited in glacio-fluvial and glacio-lacustrine environments overlie a deposit of very dense sand with gravel, cobbles and boulders. This latter material typically mantles the bedrock but none of the boreholes at this site encountered bedrock, instead terminating in the very dense sand with cobbles and boulders.

In general terms, the site was found to be underlain by a thin veneer of topsoil over layers of silty sand, silty clay, silt, sand and gravelly sand with cobbles and boulders.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

Topsoil was identified across the site, with the measured thicknesses ranging from 50 to 200 mm. These values represent only the topsoil thicknesses at the borehole locations and should not be relied upon to establish quantities across the site.

5.3 Fill

Although not encountered in the investigation program, earth and granular fill should be expected within the existing Highway 11 embankment and possibly within the TCPL right of way.

5.4 Silty Sand

A layer of fine-grained silty sand was encountered below the topsoil. This layer blankets the slope and covers underlying layers that appear to have been truncated by earlier erosion.

Based on SPT values ranging from 4 to 25 blows for 0.3 m of penetration, the deposit is classified as very loose to compact.

The measured natural moisture contents range from 6 to 26% and the soil is described as moist to wet.

The layer of silty sand ranges in thickness from 2.0 m in the north abutment to at least 6.6 m at the south approach. The base of the layer lies between Elevation 306.6 at the south approach to 297.3 at the north approach. The borehole at the south approach terminated in this soil.

The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheets and shown in Figures 1 and B1 in Appendix B.

5.5 Silt

A 3.2 m thick layer of silt was found to underlie the silty sand at the south abutment. This layer was not encountered at the north abutment or north approach and apparently pinches out within the TCPL ROW. The underside of the layer lies at Elevation 300.6.

Based on SPT values ranging from 8 to 11 blows for 0.3 m of penetration, the deposit is classified as loose to compact.

The measured natural moisture contents range from 23 to 27% and the soil is described as wet.

The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheet and shown in Figure 1 in Appendix B.

5.6 Silty Clay to Clayey Silt

The soils described above are underlain by a deposit of silty clay with clayey silt interbeds that was encountered at both abutments and at the north approach. Based on the recorded SPT values typically ranging from 3 to 11 blows for 0.3 m of penetration, the clay would be classified as soft to stiff. However, taking account of the vane shear strengths, which generally range from 90 to over 100 kPa, the clay is in fact classified as stiff to very stiff.

The clay is described as varved, with the varves typically 5 to 10 mm thick. The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheets and shown in Figures 2 and B2 in Appendix B. The plasticity of the clay ranges from low to high, as shown in Figures 7 and B3 in Appendix B.

The recorded natural moisture contents in the clay ranged from 23 to 53% and the soil is described as moist. At the upper end, the range of moisture contents equalled or slightly exceeded the liquid limit of the clay.

The results of consolidation testing conducted on three samples of the silty clay during the previous investigation and on one sample obtained during the current study are summarized in Table 5.1. The consolidation test report for the current sample is provided in Appendix B.

Table 5.1 – Consolidation Test Parameters

Location	Sample Depth (m)	w (%)	γ (kN/m ³)	e_o	p_o' (kPa)	p_c' (kPa)	OCR	C_c	C_r
BH411-2	4.6-4.9	52	16.7	1.422	55	200	3.8	0.60	0.10
TCPL1 to TCPL3	3.0-8.0	22-47	-	-	-	140-270	-	0.22-0.59	0.05-0.09

The thickness of the clay layer ranges from 4.2 m at the north approach to 8.6 m at the south abutment. The base of the clay layer lies at Elevation 293.1 at the north approach to Elevation 292.0 at the south abutment.

5.7 Silt

A layer of silt was encountered below the silty clay and extending from the south abutment to the north approach. This soil is predominantly silt-sized, with trace to some sand sizes and trace clay-sized particles. Based on SPT values generally ranging from 4 to 47 blows for 0.3 m of penetration, the silt is classified as loose to dense.

The measured natural moisture contents ranged from 21 to 33% and the soil is described as wet.

The thickness of the silt layer ranged from 10.9 m at the north abutment to 13.6 m at the south abutment. The base of the silt layer lay between Elevation 282.1 at the north

abutment and Elevation 278.4 at the south abutment. The north approach borehole terminated at Elevation 286.0 after penetrating 7.2 m into this layer. The supplementary borehole 411-2 was terminated in the silt at 9.8 m depth, Elevation 291.7.

The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheets and shown in Figures 3, 4 and B1 in Appendix B.

5.8 Sand and Silt

The silt layer is underlain by a layer of uniform, fine grained sand and silt that forms a substantial thickness across the site. Based on SPT values ranging from 5 to 102 blows for 0.3 m of penetration, this sand is classified as loose to very dense. Many of the lower SPT values are believed to be due to sample disturbance and the deposit is considered to be better described as compact to very dense.

The measured natural moisture contents ranged from 19 to 27 % and the soil is described as wet.

The thickness of this soil layer varied from 13.9 m at the south abutment to 23.1 m at the north abutment. The underside of the sand layer ranged from Elevation 264.5 at the south abutment to 259.0 at the north abutment.

The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheets and shown in Figure 5 in Appendix B.

5.9 Sand With Cobbles and Boulders

Below the fine grained sand and silt described in the previous paragraph, the boreholes encountered a layer described as sand with gravel. The grading of the sand ranged from fine to coarse and it contained varying percentages of gravel. Frequent cobbles and occasional boulders were encountered. Based on SPT values ranging from 63 blows for 0.3 m of penetration to values in excess of 100 blows for 0.3 m of penetration, this deposit is classified as very dense. All boreholes reaching this deposit were terminated after proving approximately 3.0 m of material with SPT values exceeding 100 blows for 0.3 m of penetration.

Where they could be measured, natural moisture contents ranged from 12 to 23% and the deposit is described as wet.

This deposit was not fully penetrated by any borehole but the thicknesses penetrated by sampling ranged from 8.1 m at the south abutment to 4.4 m at the north abutment. The borehole termination ranged from Elevation 256.4 at the south abutment to Elevation 254.6 at the north abutment.

The grain size distribution of a selected sample of this soil is plotted on the Record of Borehole sheet and shown in Figure 6 in Appendix B.

5.10 Depths to Refusal

The depths at which effective refusal was encountered, defined as an SPT value exceeding 100 blows for 0.3 m of penetration or a high frequency of cobbles and boulders are shown in Table 5.2.

Table 5.2 – Refusal Depths (Elevations)

Location	Borehole	Refusal Elevation (m)	Material
South Abutment	TCPL3	42.7 (264.0)	Very dense sand with gravel, cobbles and boulders
North Abutment	TCPL1	41.5 (260.0)	

5.11 Water Levels

The initial and final groundwater depths and elevations are shown in Table 5.3.

Table 5.3 – Groundwater Depths (in metres) and Elevations

Date	South Abutment				North Abutment				North Approach	
	Upper		Lower		Upper		Lower		Depth	Elev.
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.		
July 9/99	1.7	305.0	10.8	295.9	0.8	300.7	5.8	295.7	4.4	295.7
Feb 28/05	1.8	304.9	11.7	295.0	0.9	300.6	5.8	295.7	-	-

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

6 MISCELLANEOUS

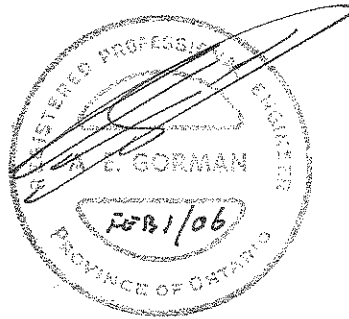
The initial site investigation and field testing work was carried out by AGRA Earth and Environmental Ltd. in 1999. The drill rig and sampling equipment used in the investigation was supplied and operated by Groundworks Drilling Inc of Toronto, Ontario. Full time supervision of field activities was carried out by staff from AGRA. The borehole locations were surveyed by Deardon and Stanton Limited.

Full-time supervision of the supplementary field activities conducted in 2005, including obtaining utility clearances, was carried out by Mr. George Azzopardi of Thurber.

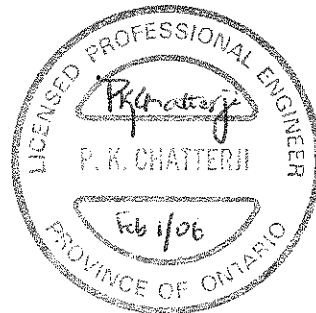
Supervision of the field program, interpretation of the field data, and preparation of the current report were carried out by Mr. Alastair E. Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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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 geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

A single-span, 36 m long, CPCI girder structure is proposed at this site and integral abutments are under consideration.

The site lies on land that slopes down to the north towards the Magnetawan River. Consequently, the south approach will lie on a shallow fill and the north approach will lie on a much more substantial fill.

The finished grade at the south abutment will lie at Elevation 311.4 and the original ground surface is at about Elevation 306.7, resulting in a 4.7 m high embankment at the abutment that diminishes with distance from the structure. At the south approach borehole, the grade enters a shallow cut.

The finished grade at the north abutment will lie at Elevation 310.6 and the original ground surface at this location is at about Elevation 301.5, giving a total embankment height of 9.1 m.

The discussion and recommendations presented in this report are based on our understanding of the project, on the factual data obtained in the course of the investigation conducted in 1999, and on the information obtained in the supplementary borehole drilled for the current study.

8 STRUCTURE FOUNDATIONS

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

Based on the results of the exploratory boreholes drilled at the proposed abutment and pier locations, the stratigraphy consists of approximately 40 to 50 m of silt, silty clay and fine grained sand overlying a stratum of sand and gravel with cobbles and boulders.

Initial consideration was given to the following foundation types:

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

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

8.1 Spread Footings

At the south abutment, the near surface silty sand and silt soils are not considered suitable for the support of spread footings due to the comparatively low geotechnical resistance that could be mobilized and the potential for unacceptably large settlements. At the north abutment, spread footings on native soil are not feasible due to the height of the approach fill.

The abutment design, particularly at the north, is not suitable for footings bearing on engineered fill pads.

A further factor to be considered in the selection of a suitable foundation system is the possibility of future excavation along the TCPL ROW in close proximity to the abutments.

In view of the foregoing discussion, the use of spread footings at this site is not recommended.

8.2 Driven Steel Piles

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

The stratigraphy at this site is such that driving end-bearing H-piles to the very dense sand with gravel, cobbles and boulders at the borehole refusal depths of 41.5 to 42.7 m (Elevation 260.0 to 264.0 m) is not likely to be possible. It is anticipated that skin friction developed in the dense to very dense sands and silts above the refusal depth will limit pile penetration, and that attempting to achieve the refusal depths may result in overdriving or damage to the pile.

Therefore, it is recommended that the piles be designed on the basis of geotechnical resistance developed by skin friction along the pile shaft. The recommended axial geotechnical resistances for design are given in Table 8.1.

Table 8.1 – Pile Geotechnical Resistance

Pile Section	Piles Driven Into Sand and Silt	
	ULS (Factored)	SLS (25 mm Settlement)
HP 310 X 110	1,800 kN	1,200 kN
HP 360 X 132	2,100 kN	1,450 kN

The estimated tip elevations at which the piles will achieve the geotechnical resistance shown above, assuming frictional resistance, are given in Table 8.2.

Table 8.2 – Pile Tip Elevations

Location	Borehole	Elevation (m)
South Abutment	TCPL3	268
North Abutment	TCPL1	262

The pile tip elevations shown in Table 8.2 should be used for cost estimating purposes only. The actual pile tip elevations should be controlled and confirmed by dynamic analysis using a Pile Driving Analyzer as described in Section 8.2.3 Pile Driving.

8.2.1 Pile Tips

The pile tips should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point), APF Hard Bite, or approved equivalent. Use of rock points is recommended to provide a higher level of protection against pile damage when driving through the dense to very dense soils to achieve the required resistance and in case cobbles and boulders are encountered.

8.2.2 Pile Installation

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

The Contract Documents should contain a NSSP alerting the bidders to:

- The possibility of encountering cobbles and boulders in the sands and silts;
- The possibility that some piles may meet refusal on a large boulder;
- The possibility of piles within a group achieving the specified resistance at different elevations.

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

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

Driving of piles adjacent to the TCPL corridor will result in vibration transmission to the utility lines in addition to settlement of the foundation soils as a result of the vibrations. Due to the proximity of the TransCanada Pipeline and FOTTS cable in the pipeline ROW, it is imperative that the proposed pile driving activities be discussed with the utility carriers to determine the acceptable level of vibration and tolerable settlement of the lines. The Contract Documents should include a NSSP to specify pile installation requirements that comply with the restrictions of the utility carriers.

8.2.3 Pile Driving

The piles at this site are designed to achieve axial geotechnical resistance through skin friction along the shaft and not by end-bearing at the refusal depths. Therefore the pile tip elevation should be established by the Pile Driving Analyzer to verify the ultimate pile resistance “R” kN per pile. “R” must have the minimum values shown in Table 8.3.

Table 8.3 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310X110	3,600 kN
HP360X132	4,200 kN

8.2.4 Downdrag

A layer of silty clay underlies the site and long-term secondary consolidation settlements may occur as a result of the planned approach embankment construction. The downdrag forces will develop along the length of pile embedded in the silty clay and the overlying native soil and fill.

A check for the effects of downdrag forces should be performed in accordance with Section 6.8.4 of the CHBDC. For the purpose of this check, the downdrag forces shown in Table 8.4 should be used.

Table 8.4 – Downdrag Forces on Abutment Piles

Pile Type	South Abutment		North Abutment	
	HP 310x110	HP 360x132	HP 310x110	HP 360x132
Factored downdrag force (f = 1.25)	450 kN	500 kN	180 kN	210 kN

Downdrag forces have been calculated using the “ β -method”, assuming that the negative skin friction will be mobilized at the outside perimeter of the “H” pile in the silty clay and overlying soils. At the abutments, negative skin friction is assumed to apply up to the underside of the 3.0 m long CSP installed as part of the integral abutment design.

8.2.5 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:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3) \quad \text{for cohesionless soils}$$

$$= 67 \cdot c_u / D \quad (\text{kN/m}^3) \quad \text{for cohesive soils}$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where	z	=	depth of embedment of pile in metres
	D	=	pile width in metres
	n_h	=	coefficient of horizontal subgrade reaction (Table 8.5)
	c_u	=	undrained shear strength (Table 8.5)
	γ	=	bulk unit weight (Table 8.5), use submerged unit weight below water table
	K_p	=	passive earth pressure coefficient (Table 8.5)

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 150 kN at ULS and 50 kN at SLS.

Table 8.5 – Parameters for Lateral Pile Resistance

Elevation	Soil Type	n_h (kN/m ³)	c_u (kPa)	K_p	Unit Weight* (kN/m ³)
South Abutment					
306.7 to 300.6	Silty sand/silt, loose to compact	1,200	-	2.8	10
300.6 to 292.0	Silty clay, stiff to very stiff	-	100	2.6	10
292.0 to 285.5	Silt, loose to compact	1,600	-	3.0	10
285.5 to 278.4	Silt, dense	3,000	-	3.3	10
278.4 to 264.5	Sand and silt, dense to very dense	5,000	-	3.5	10
Below 264.5	Sand and gravel, very dense	8,000	-	4.0	11
North Abutment					
301.5 to 299.3	Silty sand, compact	1,200	-	2.8	9
299.3 to 293.0	Silty clay, stiff to very stiff	-	100	2.6	10
293.0 to 276.2	Silt to sand and silt, compact to very loose	1,600	-	3.0	10
276.2 to 261.0	Sand and silt, dense	5,000	-	3.5	10
Below 261.0	Sand, very dense	8,000	-	4.0	11

* Submerged unit weight below the water table.

The modulus of subgrade reaction may have to be reduced, based on the pile spacing.

The following reduction factors should be used for a pile group oriented *perpendicular* to the direction of loading.

Pile spacing	Reduction Factor
4D	1.00
1D	0.5

The following reduction factors should be used for a pile group oriented *parallel* to the direction of loading.

Pile spacing	Reduction Factor
8D	1.00
6D	0.7
4D	0.4
3D	0.25

--- where "D" is the breadth of the pile, spacing is centre to centre

Intermediate values may be obtained by linear interpolation.

Except in the case of integral abutments, it is recommended that horizontal loads be resisted by means of batter piles.

8.3 Caissons

The soil conditions, and more particularly the groundwater conditions at this site are not considered to be suitable for caisson foundations. To achieve the high resistance necessary to justify the construction costs, the caissons would have to be founded in the very dense sand with gravel, cobbles and boulders.

When attempting to found in the very dense sand and gravel, it would be impossible to achieve a seal and slurry excavation and tremie concreting would be necessary.

Caissons are also not considered to be suitable for construction on a batter to resist horizontal loads.

On the basis of the installation difficulties and risks assessed for this site, caissons are not recommended.

8.4 Recommended Foundation

The recommended foundation system for all foundation elements at this site is steel H-piles. Pile driving must be controlled by use of the Pile Driving Analyzer to ensure that the required geotechnical resistance is developed.

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. However, the recommended foundation system of H-piles makes integral abutments a feasible option.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by one of the following systems:

- For a “true abutment” supported on top of the piles - a 600 mm diameter CSP filled with sand, or
- For “false abutment” - concentric CSPs in accordance with standard integral abutment design procedures.

The sand must be placed in the CSP after the pile has been driven to avoid the danger of the sand being densified by pile driving and the possibility of the CSP being dragged down by the pile.

Backfill sand should meet the gradation shown in Table 8.6.

Table 8.6 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

If the earth pressures acting on an integral abutment are to be modelled using springs, the following values of the modulus of horizontal subgrade reaction may be used:

Granular "B" Type I $k(s) = 4,500 * z/h \text{ kN/m}^3$

Granular "A" $k(s) = 5,600 * z/h \text{ kN/m}^3$

z = depth from top of abutment wall to point of interest (metres)

h = full height of the abutment wall (metres)

The upper limit of force on the abutment calculated in the analysis is the total passive force that can be mobilized in the backfill, calculated as described elsewhere in this report.

8.6 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.8 m.

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

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1V:1H above the groundwater level.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

10 UNWATERING

Excavations at this site will penetrate below the local groundwater level. Accordingly, dewatering in advance of excavation is recommended. The Contract Documents should contain a NSSP alerting the bidders that the near surface soils at the site are subject to boiling and sloughing under conditions of unbalanced hydrostatic head, and effective dewatering is recommended prior to excavating below the groundwater level.

The design of the dewatering system that may be required is the responsibility of the Contractor, and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering remains with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

The design of the dewatering system should be coordinated with the design of the excavation shoring system, where required.

11 APPROACH EMBANKMENTS

The global and internal stability of the approach embankments was analyzed for both rock fill and earth fill. 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, a factor of safety of 1.3 is considered appropriate to achieve both short and long-term stability for embankments founded on cohesionless soils. For cohesive foundation soils, the recommended factor of safety is 1.3 for short-term conditions and 1.5 for long-term conditions.

The computer output for the stability analysis of the approach embankments is shown in Appendix E.

11.1 South Approach Stability

The soil conditions governing stability of the south approach embankment consist of the approach fill over deposits of silt, silty clay and deeper silt that extend beyond the potential depth of failure. The groundwater level is assumed to be at the base of the fill/top of the sandy silt, for design purposes.

The analysis showed that a 4.7 m high rock fill approach constructed with side slopes of 1.25H:1V has a factor of safety against slope failure of 1.4 under normal circumstances. The factor of safety obtained for short term stability (total stress analysis) in which failure is forced to occur through the clay layer exceeds 2.0. The effective stress analysis was repeated assuming a seismic acceleration factor of 0.08, and a factor of safety of 1.2 was obtained.

The same analyses were repeated for an earth fill approach embankment with side slopes constructed at 2H:1V. The resulting factors of safety are 1.3 under long term circumstances, over 2.0 for short term conditions, and 1.1 under the effects of an earthquake.

It should be noted that the analyses assumed that the foundation soils would not be subject to liquefaction. This issue is addressed in Section 15 of the report.

The factors of safety obtained in the course of the analysis are summarized in Table 11.1.

11.2 North Approach Stability

The soil conditions governing stability of the north approach embankment consist of the approach fill overlying silt, silty clay and deeper silt that extend beyond the potential depth of failure. The groundwater level is assumed to be at the base of the fill/top of the silty clay for design purposes.

The analysis showed that a 9.1 m high rock fill approach constructed with side slopes of 1.25H:1V have a factor of safety against slope failure of 1.5 under normal circumstances. The factor of safety obtained for short term stability (total stress analysis) in which failure is forced to occur through the clay layer was 1.8. The analysis was repeated assuming a seismic acceleration factor of 0.08 and a factor of safety of 1.3 was obtained.

The analyses were repeated for an earth fill approach embankment using design side slopes of 2H:1V. The resulting factors of safety are 1.4 under long term circumstances, 1.7 for short term conditions, and 1.2 under the effects of an earthquake.

The factors of safety obtained in the course of the analysis are summarized in Table 11.1.

Table 11.1 – Approach Embankment Factors of Safety

Location / Material	Condition	Factor of Safety	Figure
South Approach			
Rock Fill	Long Term	1.4	E1A
	Short Term	2.7*	E1B
	0.08 Seismic	1.2	E1C
Earth Fill	Long Term	1.3	E2A
	Short Term	2.6*	E2B
	0.08 Seismic	1.1	E2C
North Approach			
Rock Fill	Long Term	1.5	E3A
	Short Term	1.8*	E3B
	0.08 Seismic	1.3	E3C
Earth Fill	Long Term	1.4	E4A
	Short Term	1.7*	E4B
	0.08 Seismic	1.2	E4C

* When failure is forced to occur through clay; otherwise, the long term condition governs.

11.3 Settlement

Settlement analysis involved computation of the immediate settlement of the foundation soils under the imposed embankment loading using elastic theory, and calculation of long-term consolidation settlement of the silty clay layer using Terzaghi one-dimensional consolidation theory. The engineering properties of the soils used in the analyses were selected based on laboratory oedometer testing conducted during the current and previous studies as well as on correlations developed between index/strength tests and compression parameters from previous investigations involving similar materials.

The results of the settlement analyses are summarized in Table 11.2.

Table 11.2 – Predicted Embankment Foundation Settlement

Location	Immediate Settlement (mm)	Primary Consolidation Settlement (mm)
South Approach	55	60
South Abutment	25	30
North Abutment	50	100
North Approach	100	200
FOTTS Line	25	50

The immediate settlement will occur during construction of the fill. The primary consolidation settlement is expected to be substantially completed within approximately 6 months. It is recommended, therefore, that the approach fill be constructed at least 6 months in advance of foundation construction. Settlement monitoring should be carried out to confirm when primary consolidation nears completion.

Preliminary design indicates that the north abutment will be constructed within about 5 m of the existing FOTTS line. Settlement of the foundation soils under the embankment loading will result in settlement at the location of the FOTTS as well. The predicted magnitude of settlement at the FOTTS is included in Table 11.2. The utility carrier must be consulted to determine if this magnitude of settlement can be tolerated by the utility lines and whether measures (such as use of lightweight fill to reduce settlements or uncovering of the utility to maintain vertical alignment) will be necessary to comply with any settlement limitations.

11.4 Seismic Considerations

The embankments discussed above are considered to be stable under earthquake loading on the assumption of a stable foundation.

This topic is dealt with more completely in Section 15 Seismic Considerations.

11.5 Forward Slopes

It is recommended that the forward slopes, where required, be constructed at the same inclination as the side slopes, i.e. 1.25H:1V for rock fill and 2H:1V for earth fill.

11.6 General Embankment Requirements

All topsoil and organic soils should be stripped from the footprint of the approach fills.

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

Where embankments are higher than 6 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide
- have 2% positive drainage to shed run-off water (earth fill embankments).

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

12 RETAINED SOIL SYSTEMS

RSS walls used in conjunction with bridge abutments must be “High Performance” and “High Appearance”. However, the predicted magnitude of settlement resulting from compression and consolidation of the foundation soils under the weight of the approach fill, derived earlier in this report, is considered unacceptable for this type of wall. The consequence of the settlement could include, though not necessarily be limited to:

- Opening of spaces between precast panels;
- Separation of the RSS wall from the structure where they meet, horizontally and vertically;
- Loss of backfill through the spaces described above;
- Distortion of the plane of the wall and degradation of its appearance;
- Localized crushing of the concrete panels; and
- In extreme cases, possible failure of the wall components.

Construction of the RSS mass is expected to occur within the footprint of the approach fill and to be subject to settlement induced by that fill. The anticipated immediate plus consolidation settlement due to the north approach fill is approximately 300 mm and this settlement must be substantially completed prior to finishing construction of the RSS wall. One solution that is commonly adopted is to construct the approach fill sufficiently far in advance of construction of the structure to accommodate the time requirement for consolidation.

At this site, a conventional fill with a forward slope will encroach over the FOTTS line. If this is acceptable, then we recommend construction of the fill a minimum of 6 months ahead of construction of the structure.

If it is not permissible to place fill over the FOTTS line, then the fill must be restricted to the alignment of the face of the RSS wall. This could be accomplished by constructing a fill with a vertical face retained by a mechanically stabilized earth (MSE) system that does not incorporate a rigid facing. Two possible variations of this treatment that might be considered are:

1. Construct an MSE fill that can tolerate the projected 300 mm of settlement without failing or distorting unacceptably and add an approved facing after primary consolidation is complete.

2. Construct an MSE fill that will be removed and replaced by an approved RSS wall after completion of primary settlement.

Settlement monitoring is recommended to confirm when primary consolidation nears completion.

After primary consolidation is essentially complete, some additional settlement of the RSS wall should be anticipated due to secondary consolidation of the silty clay. The magnitude of this settlement is expected to be in the order of 30 mm in the long-term. The settlement is not expected to impact the performance of the wall but may have an impact on the appearance.

If the foundation settlement is completed prior to RSS wall construction, the following general ground preparation is required under the RSS mass due to the variability of the near surface soils and the presence of the silty clay layer:

1. All topsoil and other deleterious material must be stripped from the footprint of the RSS mass.
2. The RSS mass must be founded on an engineered fill pad at least 2 m thick. The engineered fill must consist of OPSS Granular "A" compacted to 100% of its SPMDD at a moisture content within 2% of optimum.
3. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.
4. The highest permitted founding levels for the underside of the engineered fill are Elevation 305.0 at the south abutment and Elevation 299.0 at the north abutment (see below). Lower founding elevation may be required to accommodate the required thickness of engineered fill.

In addition to the foregoing, global stability analysis indicates that further measures will be required to provide an adequate factor of safety for the RSS at the north abutment. Figure E5 in Appendix E shows the assumptions for the stability analysis. In this analysis, the extent of the Granular A pad and minimum length of the reinforcing strips were selected to force a more favourable slip surface and provide a minimum factor of safety of 1.5 against global instability, which is considered to be an appropriate value for this RSS wall. Based on the results of this analysis, the following additional requirements apply to the RSS wall at the north abutment:

- The base of the RSS mass must not be higher than Elevation 301.0
- The base of the Granular A pad must not be higher than elevation 299.0
- The Granular A pad must extend at least 2.0 m in front of the face of the wall measured at the level of the base of the RSS
- The Granular A pad must extend at least 6 m behind the face of the wall measured at the level of the base of the RSS

- The reinforcing strips in the base of the RSS must be at least 4 m long (the supplier may specify longer).

If the supplier specifies longer reinforcing strips, the Granular A pad must extend at least 0.5 m behind the end of the strips.

For a wall constructed on a prepared subgrade as outlined above, the geotechnical parameters to be used for the design of the RSS walls are presented in Table 12.1.

Table 12.1 – RSS Design Parameters

Parameter	South Abutment	North Abutment
Factored ULS Bearing Resistance at Contact between Wall/Granular Pad	450 kPa	340 kPa
SLS Bearing Resistance at Contact between Wall/Granular Pad	200 kPa	225 kPa
Coefficient of Sliding Resistance at Contact between Wall/Granular Pad	0.55	0.45

The SLS resistances indicated in Table 12.1 are for 25 mm of settlement of the foundation soil under the RSS wall loads. It must be noted that settlements due to secondary consolidation of the underlying clays may be greater than 25 mm in the long term.

The selection of a suitable system will depend, in part, on what is available from the proprietary system suppliers. Suppliers submitting proposals for the provision of a MSE/RSS system must be required to acknowledge that they understand the projected magnitude of settlement and to provide assurances that their system will meet ministry expectations under these conditions.

The design, supply and construction of RSS must be in accordance with SP 599S22.

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular “B” Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3501.000, and rock backfill must be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

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

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.

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

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

The factors in Table 14.1 above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 14.1 – 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 $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

For design purposes, the site is treated as lying in Seismic Zone 1.

15.1 Seismic Design Parameters

The following seismic parameters should be used for design::

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The Soil Profile Type at this site has been classified as Type II. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 1.2 should be used in seismic design.

15.2 Liquefaction Potential

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method¹.

¹ 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.

Using this method, it was determined that the foundation soils are not in danger of liquefaction.

15.3 Retaining Wall Dynamic Earth Pressures

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

In calculating the values of (K_{AE}) and (K_{PE}), the following geotechnical parameters were used:

$$\begin{aligned}\phi &= 35^\circ \text{ for OPSS Granular A or Granular B Type II} \\ \phi &= 32^\circ \text{ for OPSS Granular B Type I} \\ \phi &= 42^\circ \text{ for rock fill} \\ \delta &= 50\% \text{ of } \phi\end{aligned}$$

Where ϕ = the angle of internal friction of the backfill and δ = the angle of friction between the wall and the backfill.

The seismic earth pressure coefficients to be used in design at this site are shown in Table 15.1 at the end of the text.

15.4 Slope Stability Considerations

Seismic effects were taken into account in the slope stability analyses conducted for this site using pseudo-static methods and assuming that the foundation soils would not be subject to liquefaction. Under these conditions, satisfactory factors of safety were obtained from the analysis, i.e. all values exceeded 1.0.

16 CONSTRUCTION CONCERNS

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

- The protection of the existing utility lines from any damage due to pile driving or settlement due to placement of approach fill
- Staging of the embankments and structure construction to accommodate the elastic and primary consolidation settlements
- The potential variability of pile lengths at refusal.

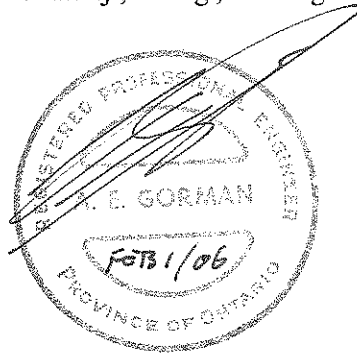
17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer



Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal

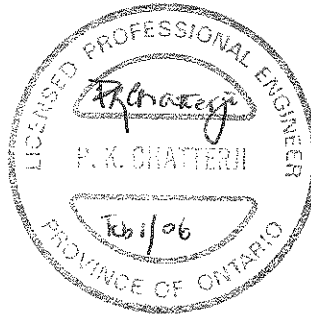


Table 15.1
Earth pressure Coefficients for Seismic Design

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.28	0.46	0.31	0.58	0.21	0.30
Passive (K_{PE})*	7.0	-	5.5	-	14.1	-
At Rest (K_{OE})**	0.53		0.58		0.44	

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

** After Woods

Appendix A

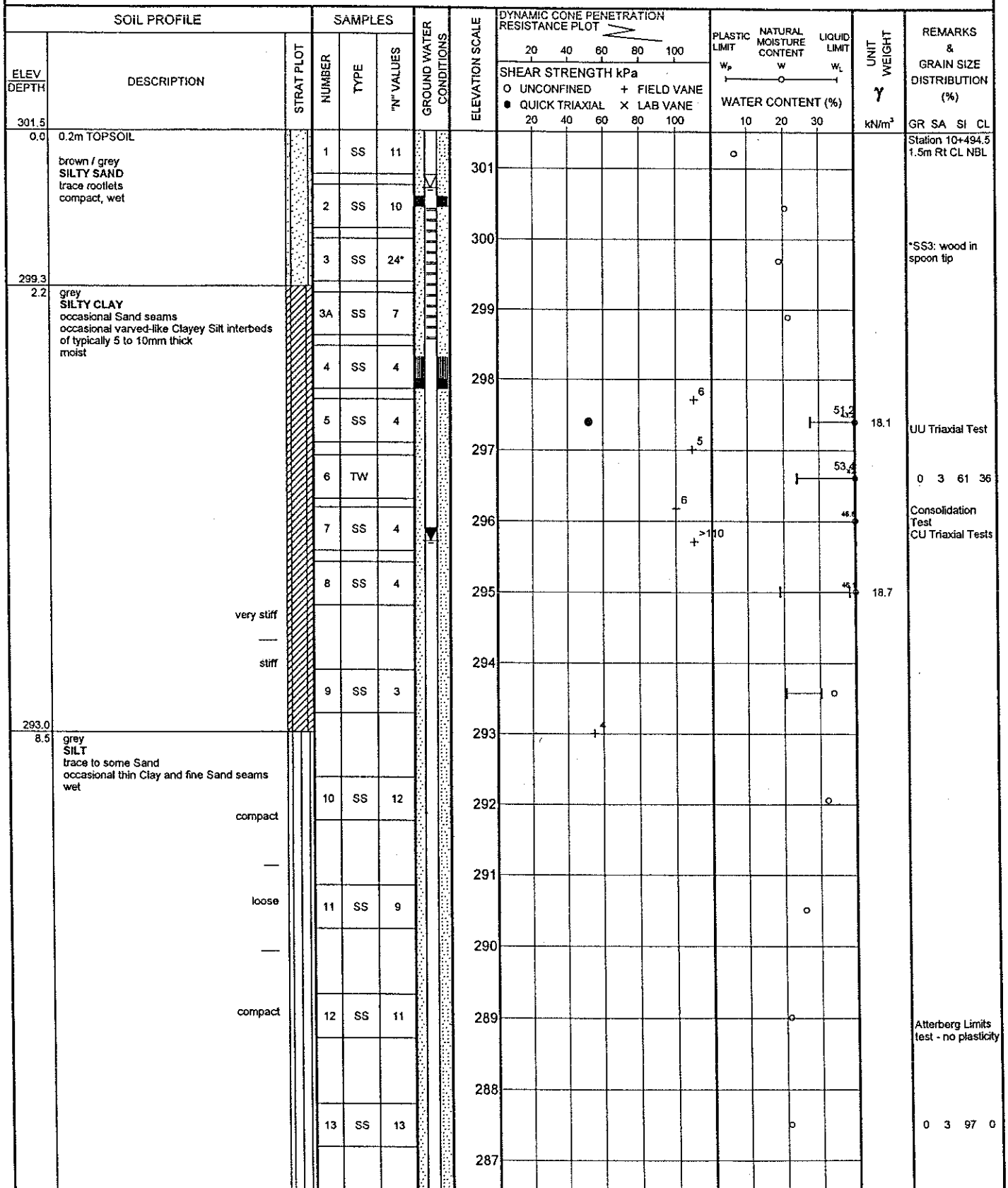
Record of Borehole Sheets

RECORD OF BOREHOLE No TCPL1

1 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046910.3 E 316959.4 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering/Tri-coning COMPILED BY CK
 DATUM Geodetic DATE 4 June 1999 - 5 June 1999 CHECKED BY ZSO



Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

METRIC

ORIGINATED BY AD

COMPILED BY CK

CHECKED BY ZSO

Continued Next Page

○ 3% STRAIN AT FAILURE

METRIC

ORIGINATED BY AD

COMPILED BY CK

CHECKED BY ZSO

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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL1

4 OF 4

METRIC

W.P. 473-93-00

LOCATION N 5046910.3 E 316959.4

ORIGINATED BY AD

DIST 52 HWY 11

BOREHOLE TYPE Hollow Stem Augering/Tri-coning

COMPILED BY CK

DATUM Geodetic

DATE 4 June 1999 - 5 June 1999

CHECKED BY ZSO

SOIL PROFILE						SAMPLES	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS		20 40 60 80 100					
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%)			
								20 40 60 80 100	w _p	w	w _L		
	grey SAND some Silt, some Gravel frequent cobbles, some boulders, very dense, wet	[Pattern]				[Symbol]	256						Coring through cobbles/boulders from 44.2m to 45.1m depths.
254.6			34	SS	75/7.5								
46.9	END OF BOREHOLE		35	SG	68/2.5		255						
	Refusal to split spoon sampler advance at 45.8m and 45.4m depths. Dynamic Cone Penetration Test conducted @ 46.8m depth. Refusal @ 46.9m depth. WL in open borehole on completion: 6.2m Lower piezometer tip at Elev. 256m and upper piezometer tip at Elev. 298.5m WL IN PIEZOMETER July 9/99: Elev. 295.7m depth (lower) Elev. 300.7m depth (upper)		36	SS	100/5.0								

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL2

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5046929.9 E 316958.1 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY CK
DATUM Geodetic DATE 9 June 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIMIT MOISTURE CONTENT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W		
300.1													
0.0	0.2m TOPSOIL		1	SS	2		300						GR SA SI CL Station 10+514.5 2.0m Rt CL NBL
	brown/gray SAND some gravel trace rootlets, decomposed organics very loose to compact (Possible creek deposits)		2	SS	11		299						
			3	SS	12		298						
297.3			3b	SS	5		297						
2.8	gray SILTY CLAY occasional varved-like Clayey Silt interbeds of typically 5 to 10mm thick. very stiff moist		4	TW			296						Consolidation Test 0 17 58 25
			5	SS	6		295						
			6	SS	5		294						
			7	SS	6		293						
			8	SS	8		292						
293.1							291						
7.0	gray SILT some Sand, occasional thin Clay and Sand seams loose to compact wet		9	SS	11		290						0 1 82 17
			10	SS	11		289						
			11	SS	5		288						
			12	SS	5		287						
			13	SS	11		286						0 25 75 0
286.0													
14.2	END OF BOREHOLE												
	WL IN PIEZOMETER July 9/99: Elev. 295.7												

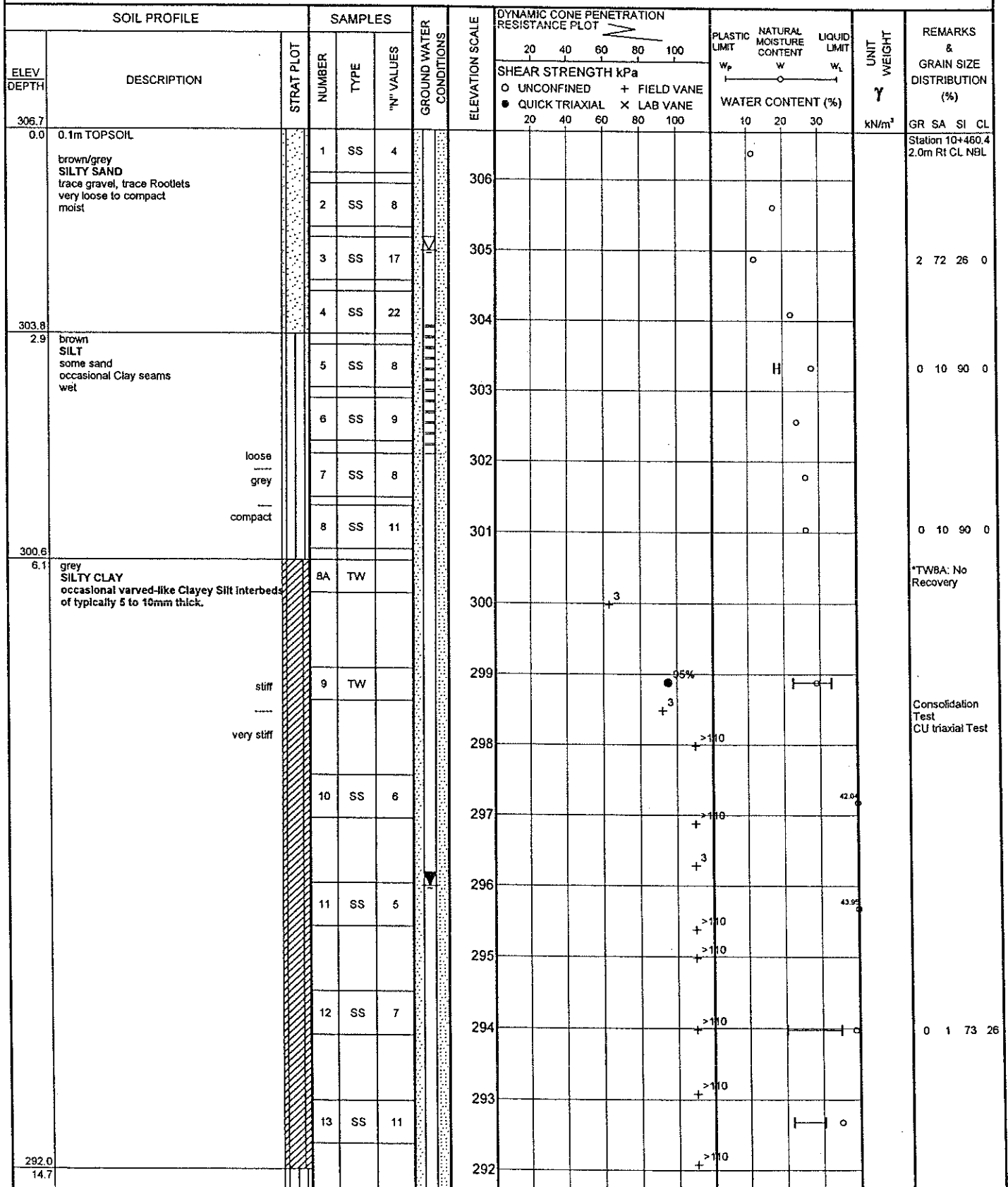
+ 3 . X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL3

1 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046876.5 E 316963.3 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash boring / Rock coring COMPILED BY CK
 DATUM Geodetic DATE 9 June 1999 - 13 June 1999 CHECKED BY ZSO



Continued Next Page

+3, X3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL3

3 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046876.5 E 316963.3 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stern Augering / Wash boring / Rock coring COMPILED BY CK
 DATUM Geodetic DATE 9 June 1999 - 13 June 1999 CHECKED BY ZSO

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		x LAB VANE
							20	40	60	80	100	10	20	30						
	grey SAND and SILT wet		24	SS	45								○		0 57 43 0					
	dense																			

	very dense																			
			25	SS	63								○							
			26	SS	59								○							

	dense		27	SS	48								○							

	compact		28	SS	19								○		0 16 84 0					

	very dense		29	SS	67								○							
			30	SS	62								○							
			31	SS	66								○							
264.5																				
42.2	grey SAND with GRAVEL frequent cobbles very dense, wet		32	SS	100/18								○							
			33	SS	100/19															
															</					

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL3

4 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046876.5 E 316963.3 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash boring / Rock coring COMPILED BY CK
DATUM Geodetic DATE 9 June 1999 - 13 June 1999 CHECKED BY ZSO

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 x LAB VANE												
							20	40	60	80	100									
261.3																				
45.4	grey SAND very dense, wet		34	SS	63		261													
260.4																				
46.3	grey SAND some silt, trace gravel, frequent cobbles occasional boulders very dense, wet		35	SS	94		260									7 79 (14)				
							259													
			36	NQ																
							258													
			37	SS	100/19															
							257													
			38	NQ																
256.4																				
50.3	END of BOREHOLE																			
	Lower piezometer tip at Elev. 263.5m and upper piezometer tip at Elev. 302m																			
	WL IN PIEZOMETER																			
	July 9/99: Elev. 295.9m depth (lower) Elev. 305.0m depth (upper)																			

+ ³ . x ³ : Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL4

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5046853.6 E 316965.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY CK
DATUM Geodetic DATE 13 June 1999 CHECKED BY ZSO

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			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
313.1							20	40	60	80	100	10	20	30		GR SA SI CL				
0.0	0.15m TOPSOIL	loose	1	SS	7								○			Station 10+437.4 CL NBL				
	brown SAND fine, some Silt layers compact, moist		2	SS	13								○							
			3	SS	13								○							
			4	SS	15															
		sand and silt		5	SS	14							○			0 48 52 0				
				6	SS	23														
308.7														○						
4.4	grey SILTY SAND some sandy silt layers fine to medium compact, moist		7	SS	23								○			0 80 20 0				
			8	SS	22								○							
306.6	sandy silt		9	SS	25									○		0 22 78 0				
6.6	END of BOREHOLE																			
	No free water in open borehole on completion.																			

RECORD OF BOREHOLE No 411-2

1 OF 2

METRIC

W.P. 480-93-01 LOCATION N 5 046 910 E 316 960, Trans Canada Pipe Line ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 10.02.05 - 10.02.05 CHECKED BY MA

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES			20 40 60 80 100	20 40 60 80 100					
301.5	TOPSOIL (50 mm)		1	SS	15		301							
0.0 0.1	SAND, trace gravel, trace silt Compact Brown Wet		2	SS	17									
			3	SS	11		300							
299.7	Silty SAND, trace gravel Loose Grey Wet		4	SS	9		299							3 70 23 3
298.9	Silty CLAY, trace sand, occasional silt seams Stiff Grey		5	SS	8									
			5	SS	5		298							0 1 44 54
			1	TW	PH		297							0 1 39 61
			2	TW	PH									
			3	TW	PH		296							
							295							
			6	SS	7		294							
							293							
			7	SS	18									
292.4	SILT, some clay, trace sand Compact Grey Wet						292							0 1 89 11
9.1														
291.7														
9.8	END OF BOREHOLE AT 9.75 m.													

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 411-2

2 OF 2

METRIC

W.P. 480-93-01 LOCATION N 5 046 910 E 316 960, Trans Canada Pipe Line ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 10.02.05 - 10.02.05 CHECKED BY MA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W P W W L	20 40 60		
	BOREHOLE OPEN TO 9.75 m AND WATER LEVEL AT 6.10 m UPON COMPLETION. BOREHOLE GROUTED TO SURFACE.												

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

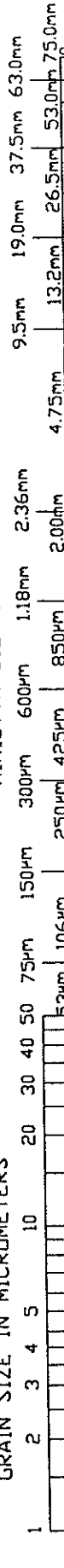
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



PERCENT PASSING (%)

PERCENT PASSING (%)

SAMPLE DATA

SYMBOL

SAMPLE

BOREHOLE NO.

TCPL3

TCPL3

TCPL3

TCPL4

TCPL4

TCPL4

SS3

SS5

SS8

SS5

SS7

SS9

GRAIN SIZE DISTRIBUTION

CLIENT: DELCAN

JOB NO: TT98820 W P 473-93-00

PROJECT: HWY 11

LOCATION: TCPL BRIDGE

DATE: AUGUST 3, 1999

FIGURE: 1



ENGINEERING GLOBAL SOLUTIONS

SURFICIAL SAND, SOME GRAVEL, TO SILT

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

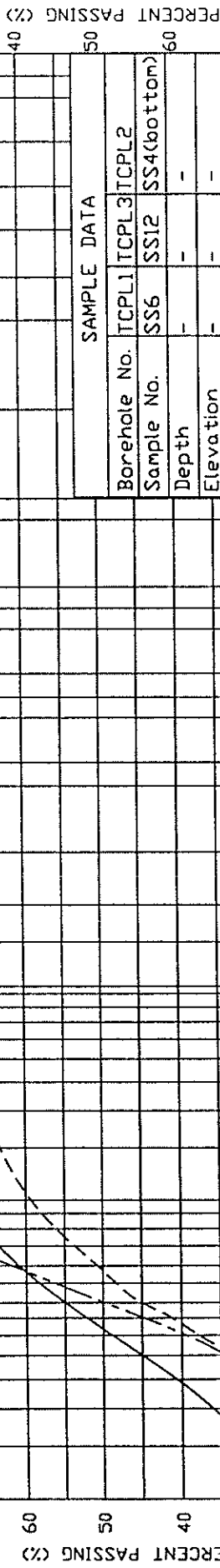
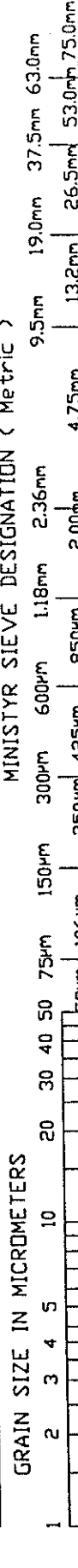
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTYR SIEVE DESIGNATION (Metric)



SAMPLE DATA

Borehole No.	TCPL1	TCPL3	TCPL2
Sample No.	SS6	SS12	SS4(bottom)
Depth	-	-	-
Elevation	-	-	-
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Date Sampled	-	-	-
Test Date	July 9 1999	June 30 1999	July 15 1999

MINISTYR SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SILTY CLAY



ENGINEERING GLOBAL SOLUTIONS

CLIENT: BELCAN

JOB NO.: TT98820 W P 473-93-00

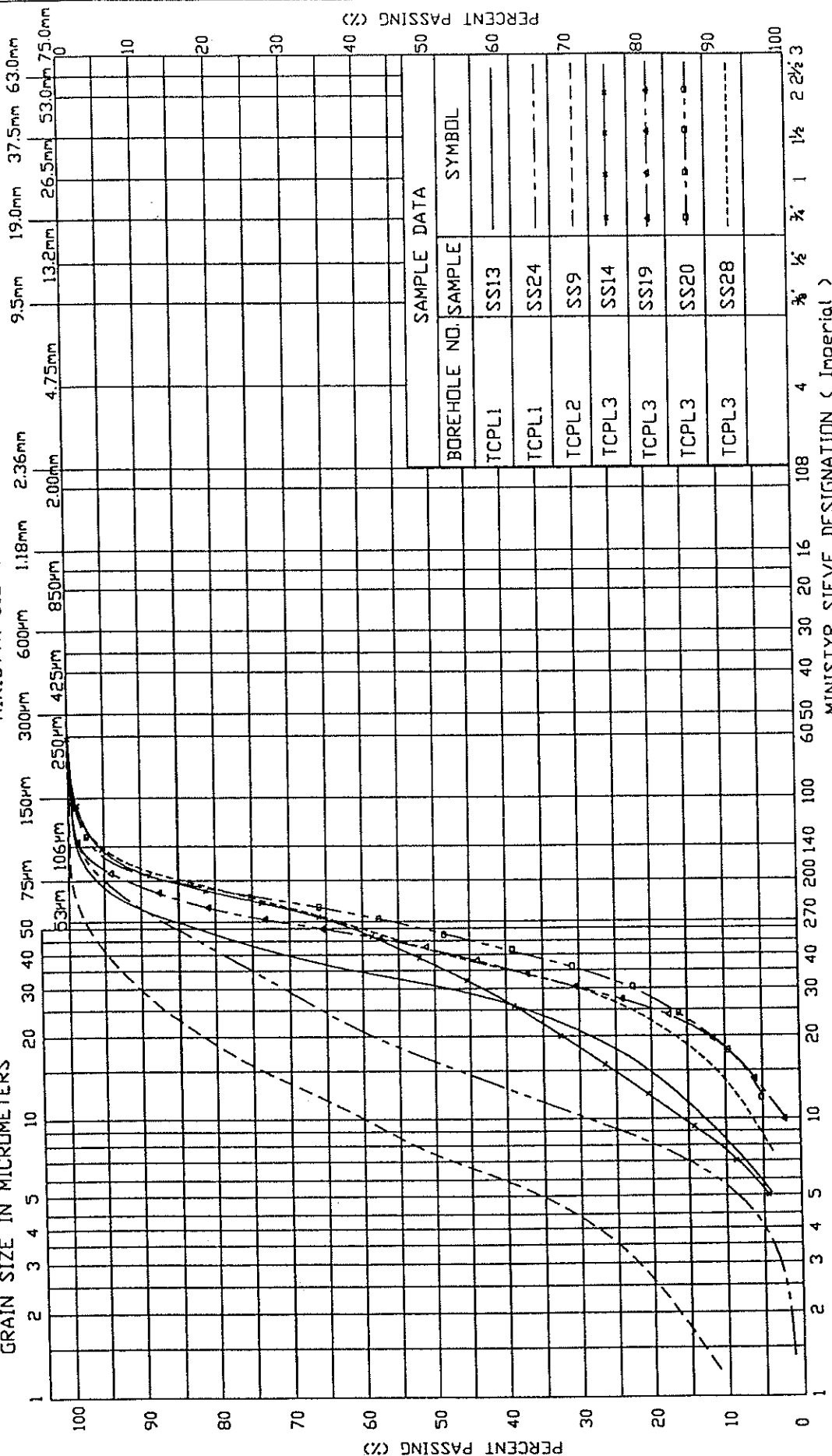
PROJECT: HWY 11

LOCATION: TCPL BRIDGE

DATE: AUGUST 3, 1999

FIGURE: 2

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION

SILT

TRACE TO SOME SAND, OCCASIONAL CLAY

CLIENT:	DELCAN
---------	--------

CELENT	TT98820	W P 473-93-00
--------	---------	---------------

PROJECT:	HWY 11
----------	--------

LOCATION:	TCPL BRIDGE
-----------	-------------

DATE: _____



AGRA

ENGINEERING GLOBAL SOLUTIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

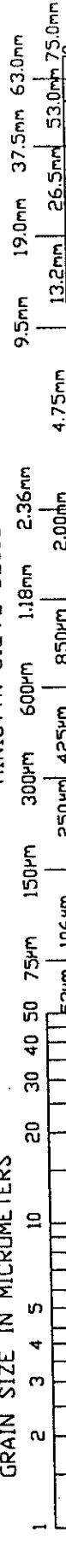
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



PERCENT PASSING (%)

PERCENT PASSING (%)

SAMPLE DATA

Borehole No.	TCPL1	TCPL1	TCPL3
Sample No.	SS19	SS28	SS24
Depth	-	-	-
Elevation	-	-	-
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Date Sampled	-	-	-
Test Date	July 12 1999	July 12 1999	June 30 1999

Test Date

MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SAND AND SILT



ENGINEERING GLOBAL SOLUTIONS

CLIENT: DELCAN

JOB NO: TT98820 W P 473-93-00

PROJECT: HWY 11

LOCATION: TCPL BRIDGE

DATE: AUGUST 3, 1999

FIGURE: 5

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

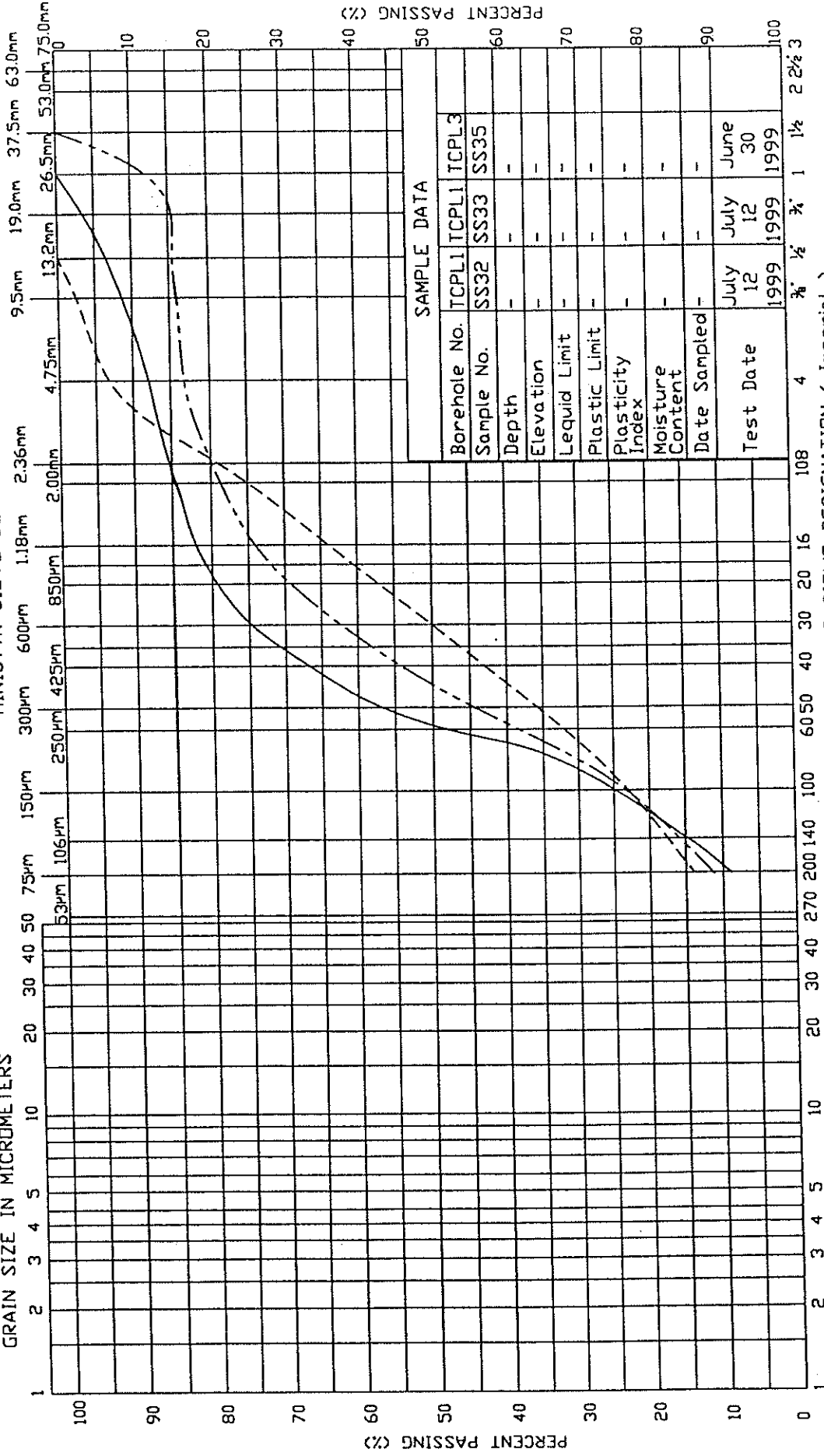
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



SAMPLE DATA

Borehole No.	TCPL1	TCPL1	TCPL3
Sample No.	SS32	SS33	SS35
Depth	-	-	-
Elevation	-	-	-
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Date Sampled	-	-	-
Test Date	July 12 1999	July 12 1999	June 30 1999

MINISTRY SIEVE DESIGNATION (Imperial)

CLIENT:	DELCAN
JOB NO.:	TT98820
PROJECT:	HWY 11
LOCATION:	TCPL BRIDGE
DATE:	AUGUST 3, 1999

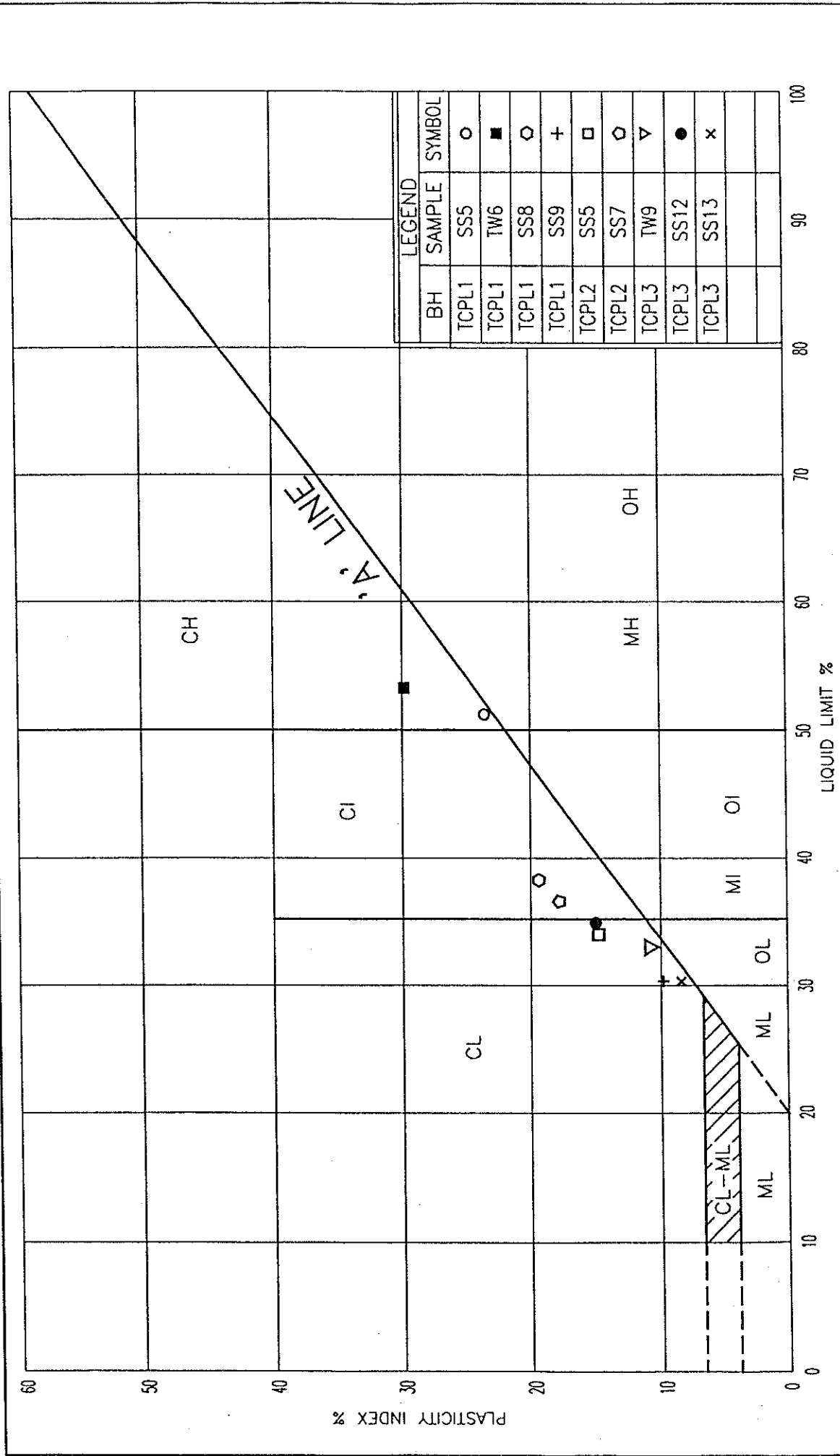
GRAIN SIZE DISTRIBUTION

SAND	TCPL1: SS32
TRACE TO	TCPL1: SS33
SOME GRAVEL	TCPL3: SS35



ENGINEERING GLOBAL SOLUTIONS

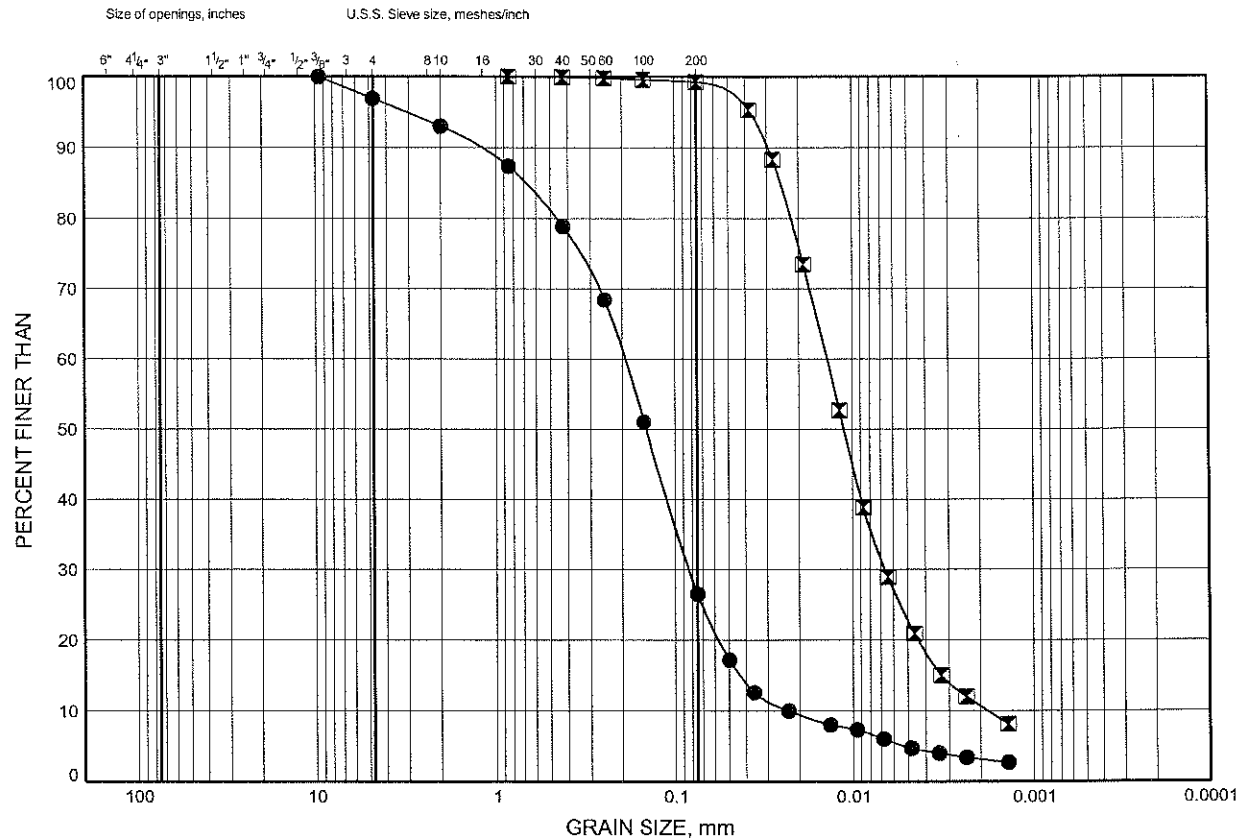
FIGURE: 6



Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B1

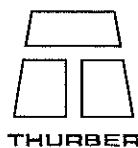
Silty Sand and Silt



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	411-2	2.13	299.37
⊠	411-2	9.45	292.05

Date January 2006
Project 480-93-01

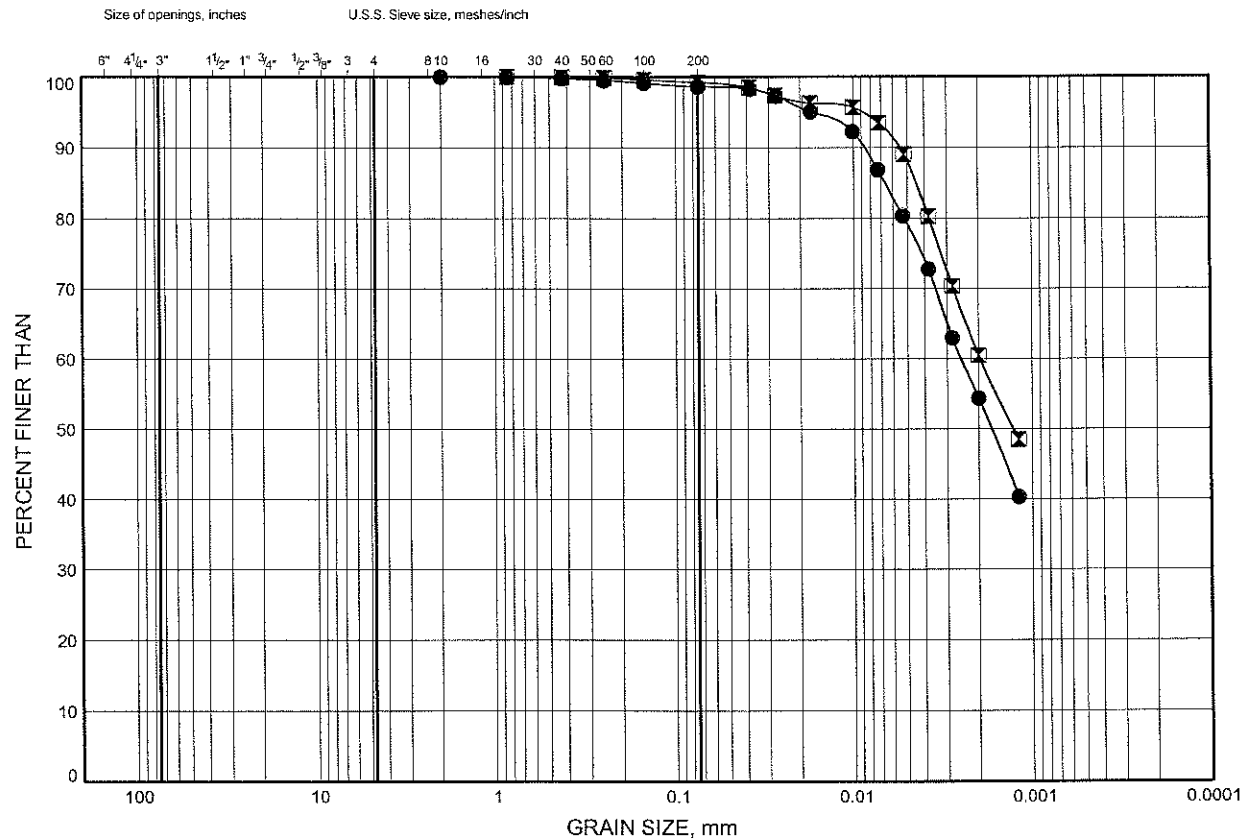


Prep'd WM
Chkd. MRA

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B2

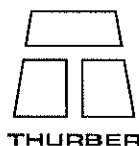
Silty Clay



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	411-2	3.35	298.15
◻	411-2	4.72	296.78

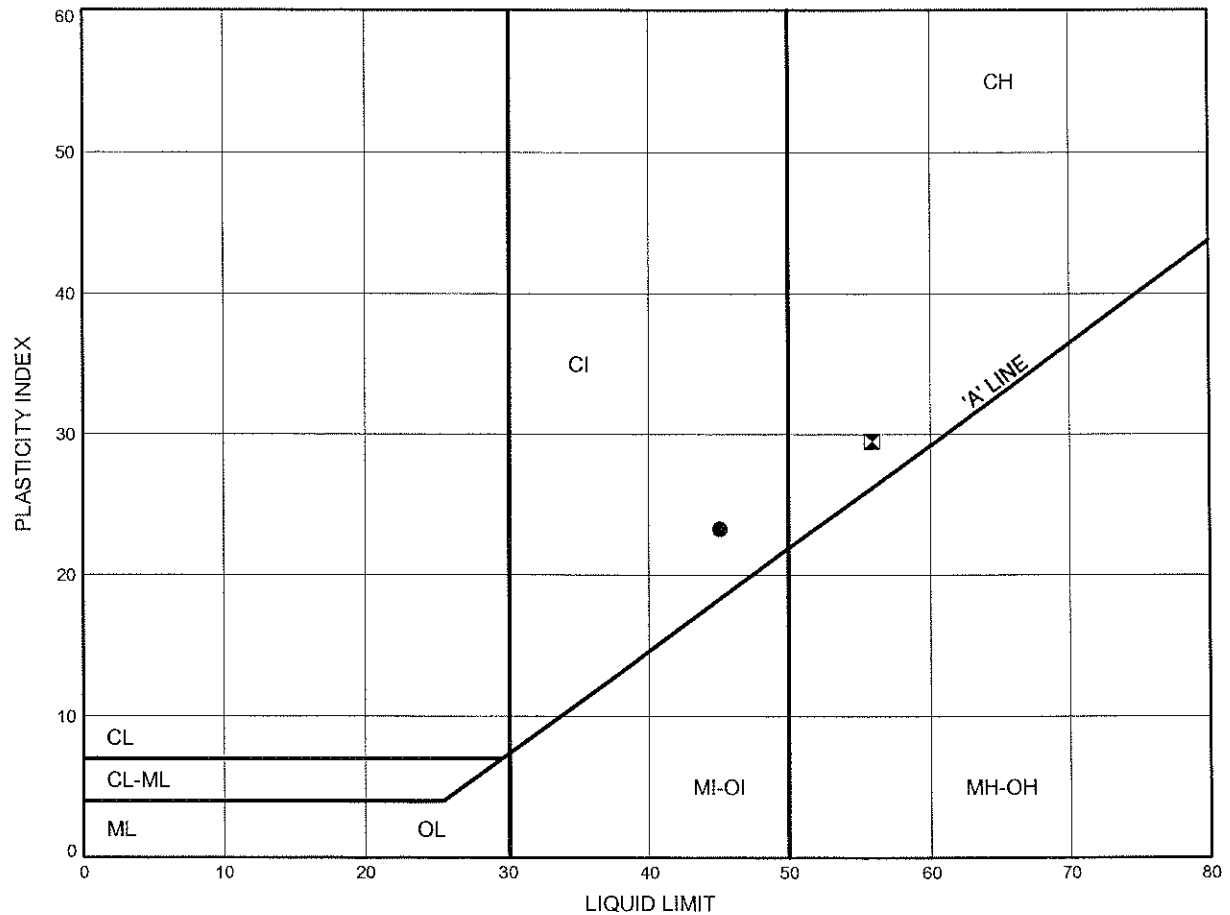
Date January 2006
Project 480-93-01



Prep'd WM
Chkd. MRA

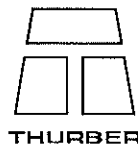
Hwy 11 Katrine
ATTERBERG LIMITS TEST RESULTS

FIGURE B3



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	411-2	3.35	298.15
⊠	411-2	4.72	296.78

Date January 2006
 Project 480-93-01



Prep'd WM
 Chkd. MRA



Consolidation Test Report

CLIENT: **Marshall Macklin Monaghan**

FILE NUMBER: 18-45-1/19-1423-16

PROJECT: Highway 11 - TransCanada Pipeline (TCPL)

REPORT DATE: December 7, 2005

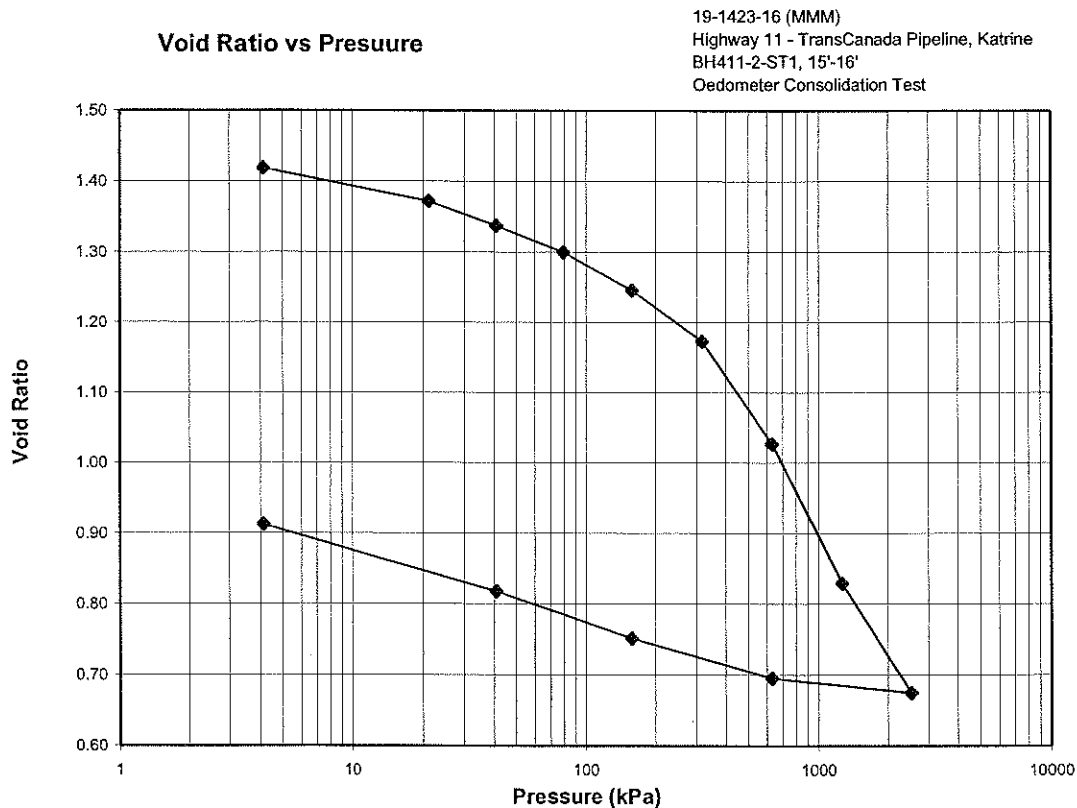
TEST DATES: November 16, 2005 - November 28, 2005

SAMPLE: BH411-2-ST1, 15'-16'
Silty Clay, firm to stiff, grey, uniform, plastic, (CH), 61 % Clay, 38 % Silt & 1 % Sand

PROCEDURE: Tested 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)	1707.5	2047.8
Dry Dens. (kg/m ³)	1120.4	1422.2
Moisture Cont. (%)	52.4	37.1
Void Ratio	1.422	0.908
Saturation (%)	99.9	

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



TEST DONE BY: EA
REVIEWED BY: JPL



Consolidation Test Report

Highway 11 - TransCanada Pipeline (TCPL)

18-45-1/19-1423-16

BH411-2-ST1, 15'-16'

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

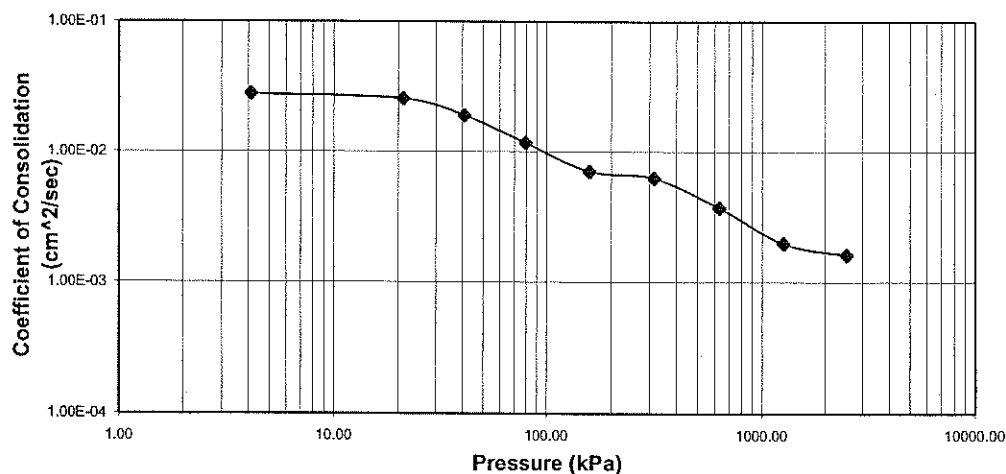
LOADING: A seating load of 4.13 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied and the duration of each load step was 24 hours

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

Pressure (kPa)	Corr. Hgt (mm)	Avg. Hgt. (mm)	T90 (min)	Cv (cm ² /sec)	Void Ratio	mv (m ² /kN)	k (cm/s)
0.00	19.050	19.050			1.422		
4.13	19.027	19.039	0.46	2.77E-02	1.419	1.128E-03	3.062E-06
21.11	18.643	18.835	0.49	2.56E-02	1.372	7.294E-04	1.829E-06
40.95	18.353	18.498	0.64	1.89E-02	1.337	4.005E-04	7.415E-07
79.32	18.045	18.199	1.00	1.17E-02	1.300	2.843E-04	3.261E-07
158.55	17.594	17.819	1.59	7.06E-03	1.246	1.901E-04	1.314E-07
316.46	16.992	17.293	1.69	6.25E-03	1.173	1.909E-04	1.17E-07
632.19	15.784	16.388	2.56	3.71E-03	1.027	1.294E-04	4.701E-08
1263.90	14.146	14.965	4.00	1.98E-03	0.829	5.047E-05	9.785E-09
2527.73	12.867	13.506	4.00	1.61E-03	0.674	4.488E-06	7.087E-10
632.19	13.038	12.952			0.695		
158.55	13.503	13.270			0.751		
40.95	14.051	13.777			0.817		
4.13	14.833	14.442			0.912		

Coefficient of Consolidation vs Pressure

19-1423-16 (MMM)
Highway 11 - TransCanada Pipeline, Katrine
BH411-2-ST1, 15'-16'
Oedometer Consolidation Test



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

TEST DONE BY: EA
REVIEWED BY: JPL



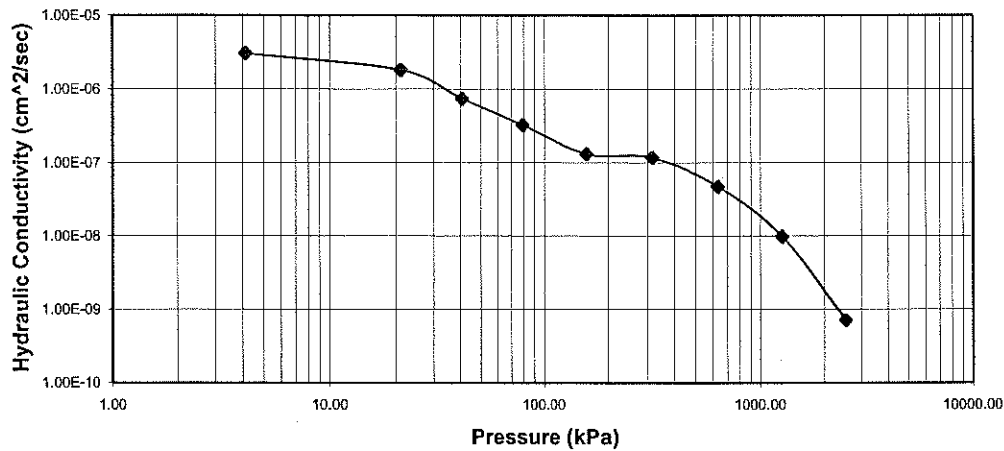
Consolidation Test Report

Highway 11 - TransCanada Pipeline (TCPL)
18-45-1/19-1423-16

BH411-2-ST1, 15'-16'

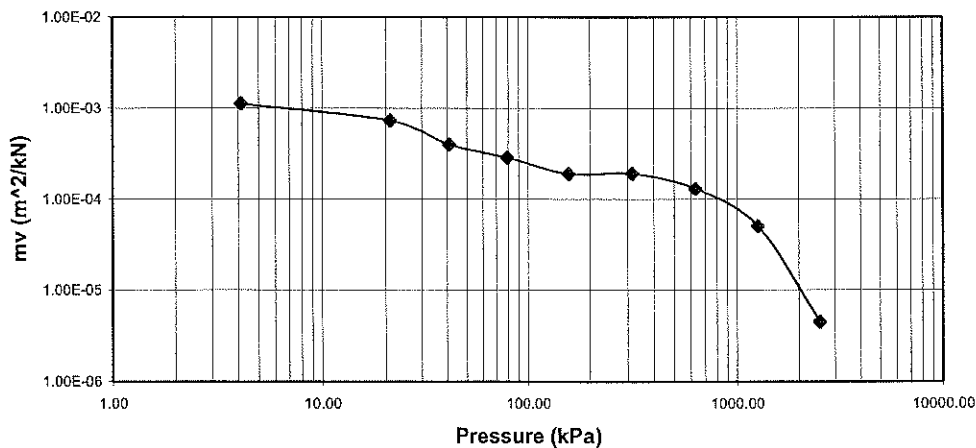
Hydraulic Conductivity vs Pressure

19-1423-16 (MMM)
Highway 11 - TransCanada Pipeline, Katrine
BH411-2-ST1, 15'-16'
Oedometer Consolidation Test



mv vs Pressure

19-1423-16 (MMM)
Highway 11 - TransCanada Pipeline, Katrine
BH411-2-ST1, 15'-16'
Oedometer Consolidation Test



TEST DONE BY: EA
REVIEWED BY: JPL

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil	Footing on Engineered Fill	Caisson
<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance if driven to refusal in the very dense soil. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Construction concerns related to the possibility of pile being obstructed by a boulder during driving. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low geotechnical resistance available at this site. ii. Potential for unacceptable magnitude of settlement. iii. Problems related to possible future excavation in the TCPL ROW <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows choice of conventional or semi-integral abutment. iii. Allows use of perched abutments. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of constructing engineered fill. ii. Low geotechnical resistance available at this site. iii. Potential for unacceptable magnitude of settlement. iv. Problems related to possible future excavation in the TCPL ROW <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded on very dense soil. ii. Construction of caissons could continue in freezing weather. iii. Choice of conventional or semi-integral abutment design. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. High cost ii. Soil conditions encountered at this site are considered to be unsuitable. <p>NOT RECOMMENDED</p>

Appendix D

Special Provisions

Highway 11 Northbound Lanes over TransCanada Pipeline

The following Special provisions are referenced in this report:

- Amendment to OPSS 206, December 1993
- SP 599S22
- Special Provision No. 902S01
- Special Provision No. 903S01

Appendix E

Selected Slope Stability Output

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL South Approach
 Rock Fill 1.25:1 Effective Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock Fill	21	0	42
Silty Sand	21	0	30
Clay	20	0	26
Silt	22	0	30

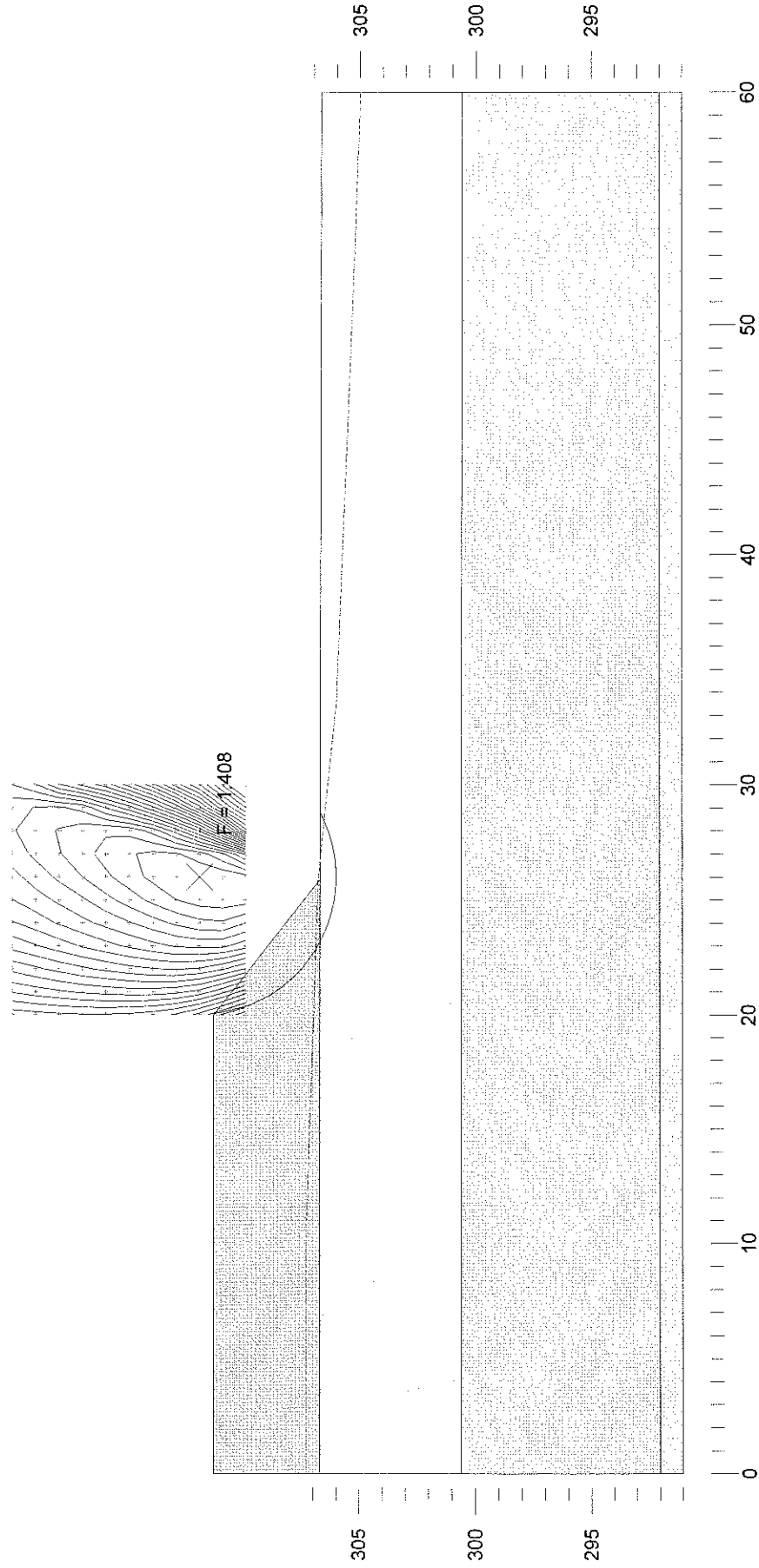


Figure E1A

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL South Approach
 Rock Fill 1.25:1 Total Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock Fill	21	0	42
Silty Sand	21	0	30
Clay	20	60	0
Silt	22	0	30

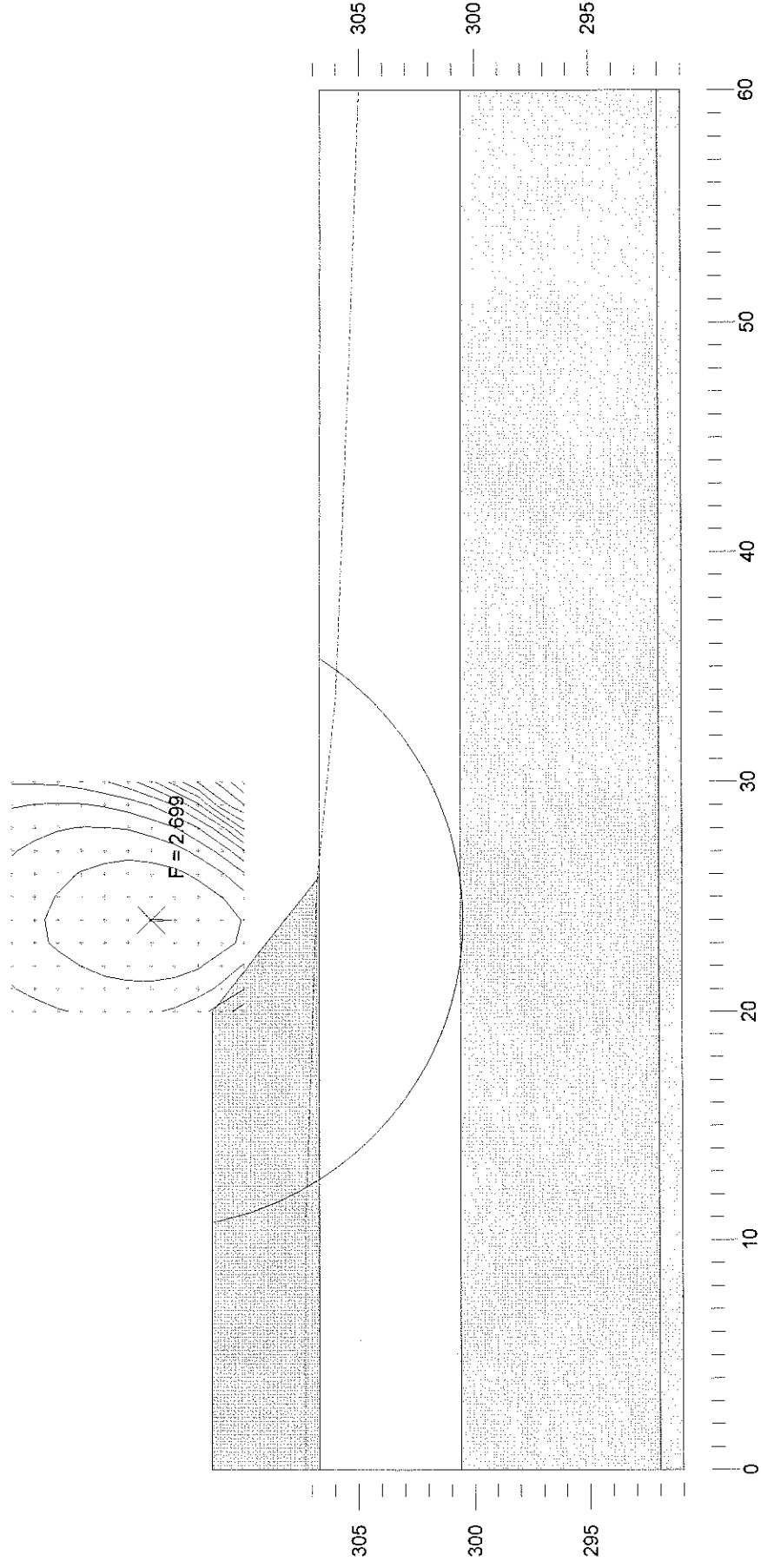


Figure E1B

Thurber Engineering Ltd. - Toronto
19-1423-16
Hwy 11, Katrine
Jan 23, 2006
TCPL NBL South Approach
Rock Fill 1.25:1 Seismic Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock Fill	21	42	1
Silty Sand	21	30	1
Clay	20	26	1
Silt	22	30	1

Seismic coefficient = 0.08

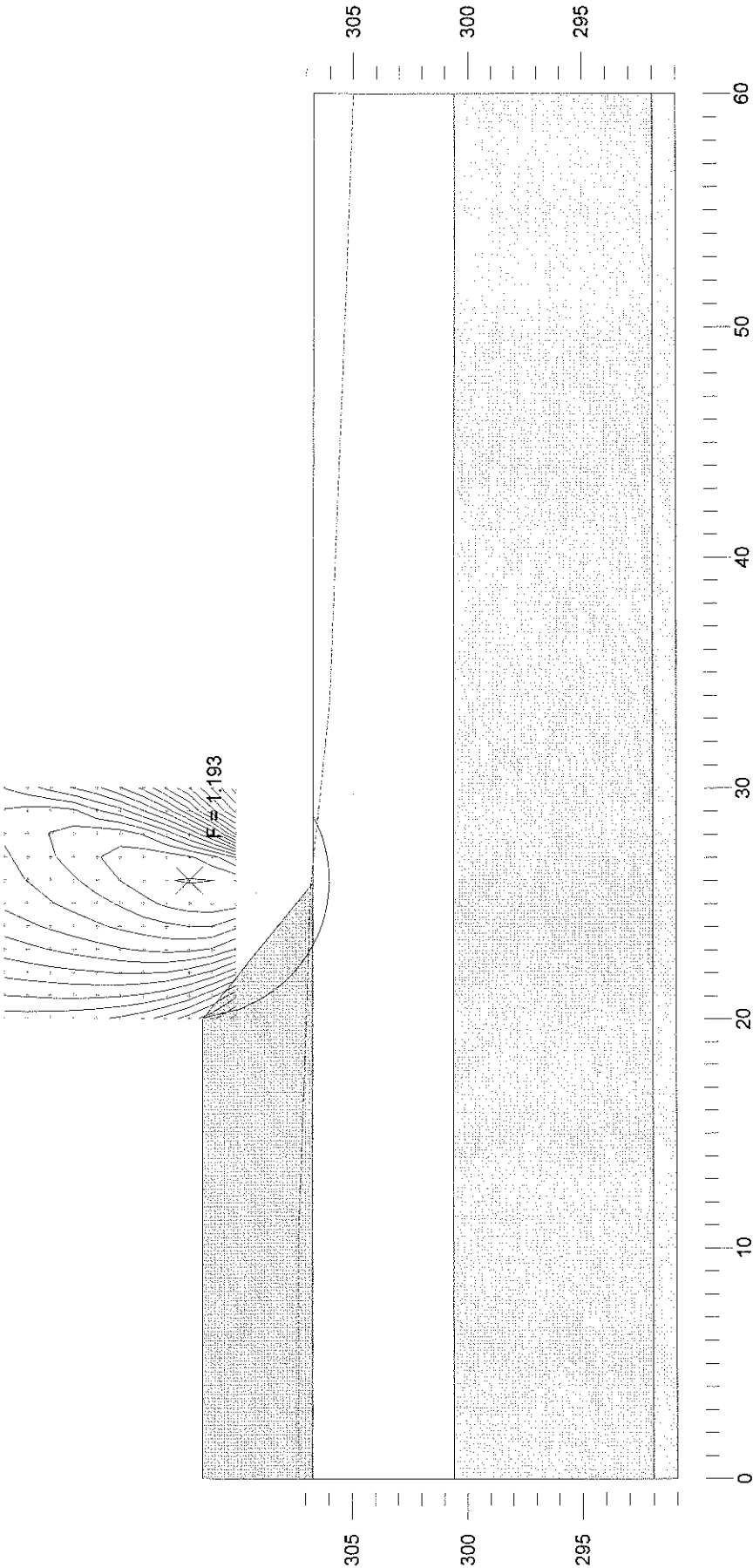


Figure E1C

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL South Approach
 Earth Fill 2:1 Effective Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	22	0	1
Silty Sand	21	0	1
Clay	20	0	1
Silt	22	0	1

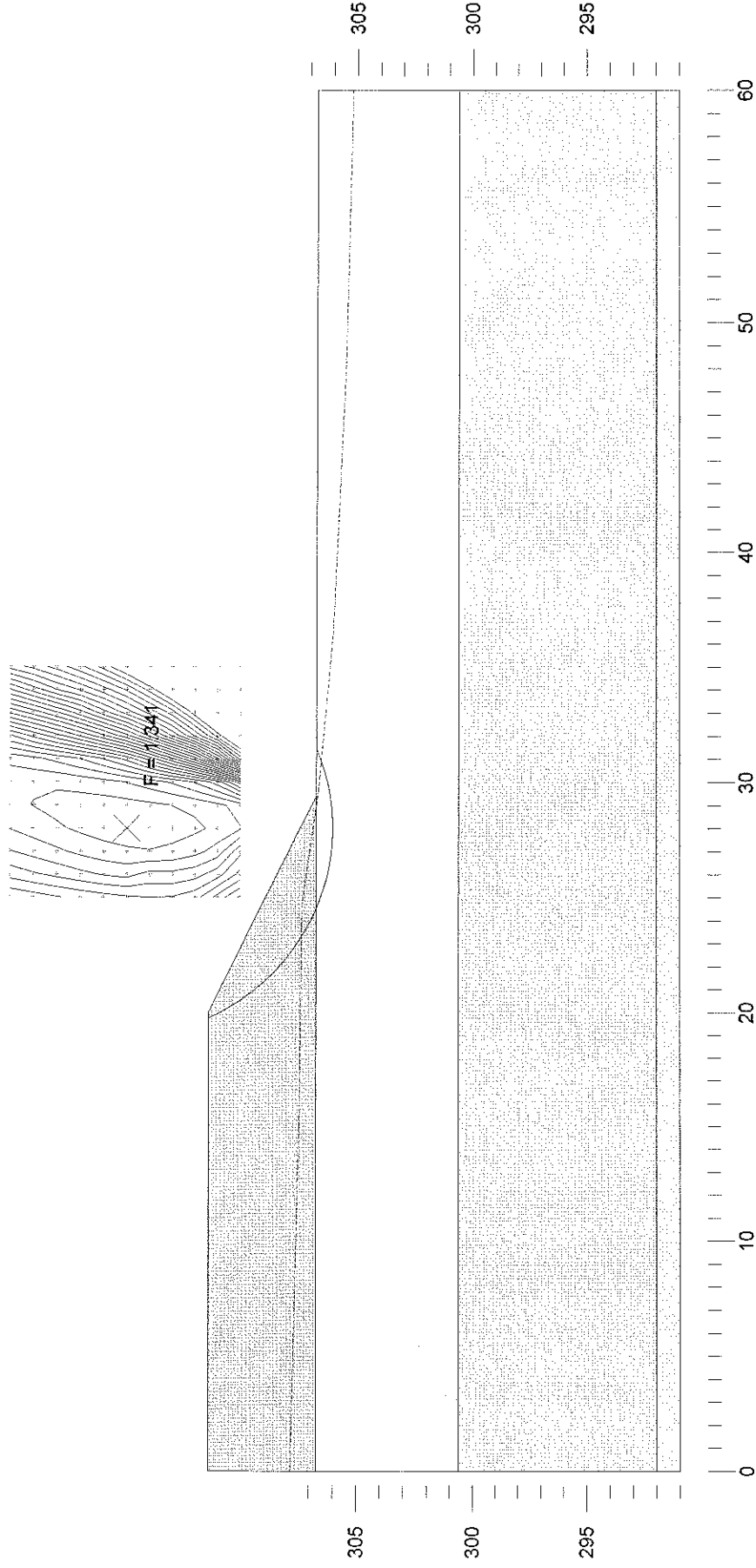


Figure E2A

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL South Approach
 Earth Fill 2:1 Total Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	22	0	1
Silty Sand	21	0	1
Clay	20	60	1
Silt	22	0	1

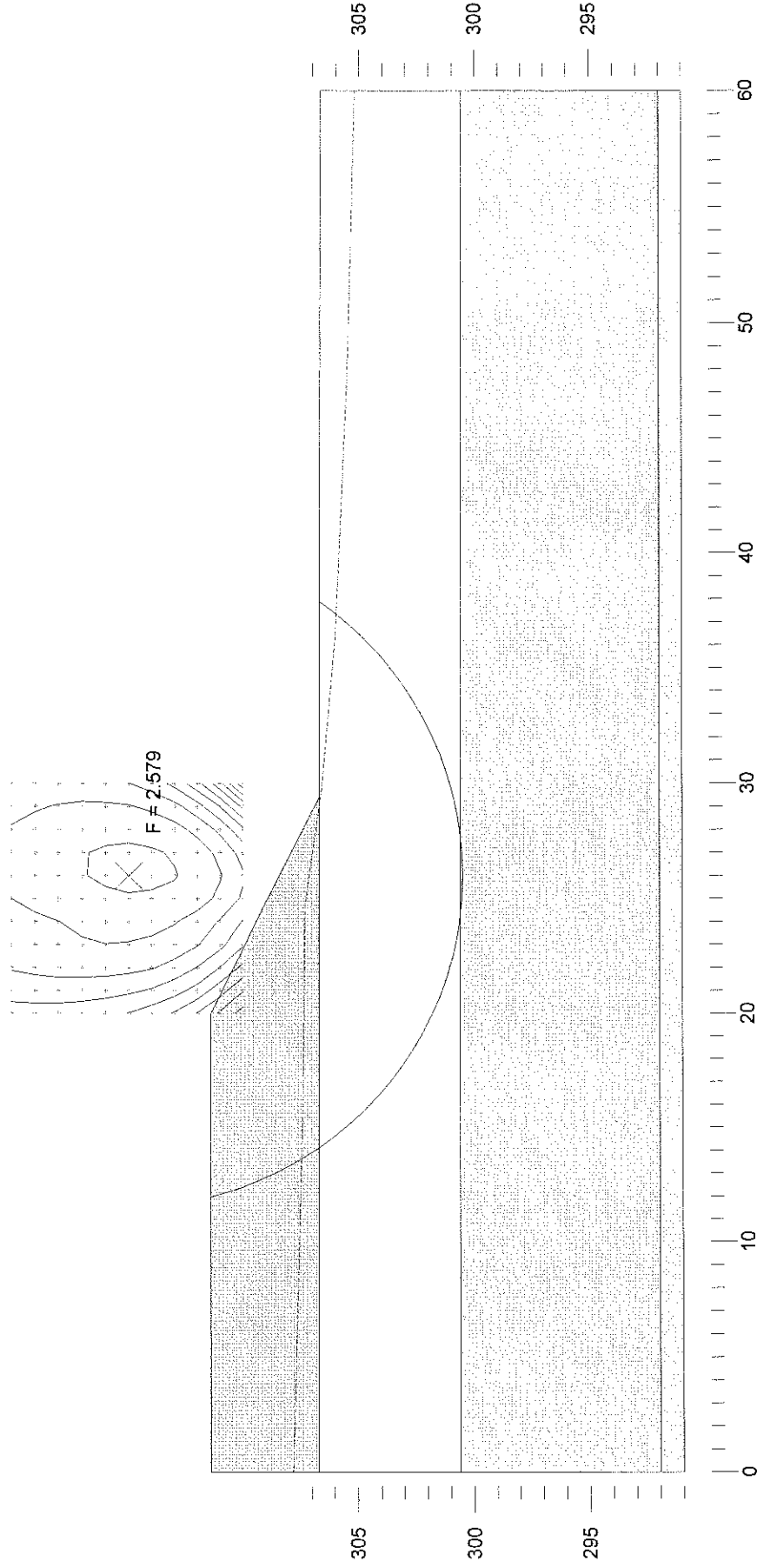


Figure E2B

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Earth Fill	22	0	30	1
Silty Sand	21	0	30	1
Clay	20	0	26	1
Silt	22	0	30	1

Seismic coefficient = 0.08

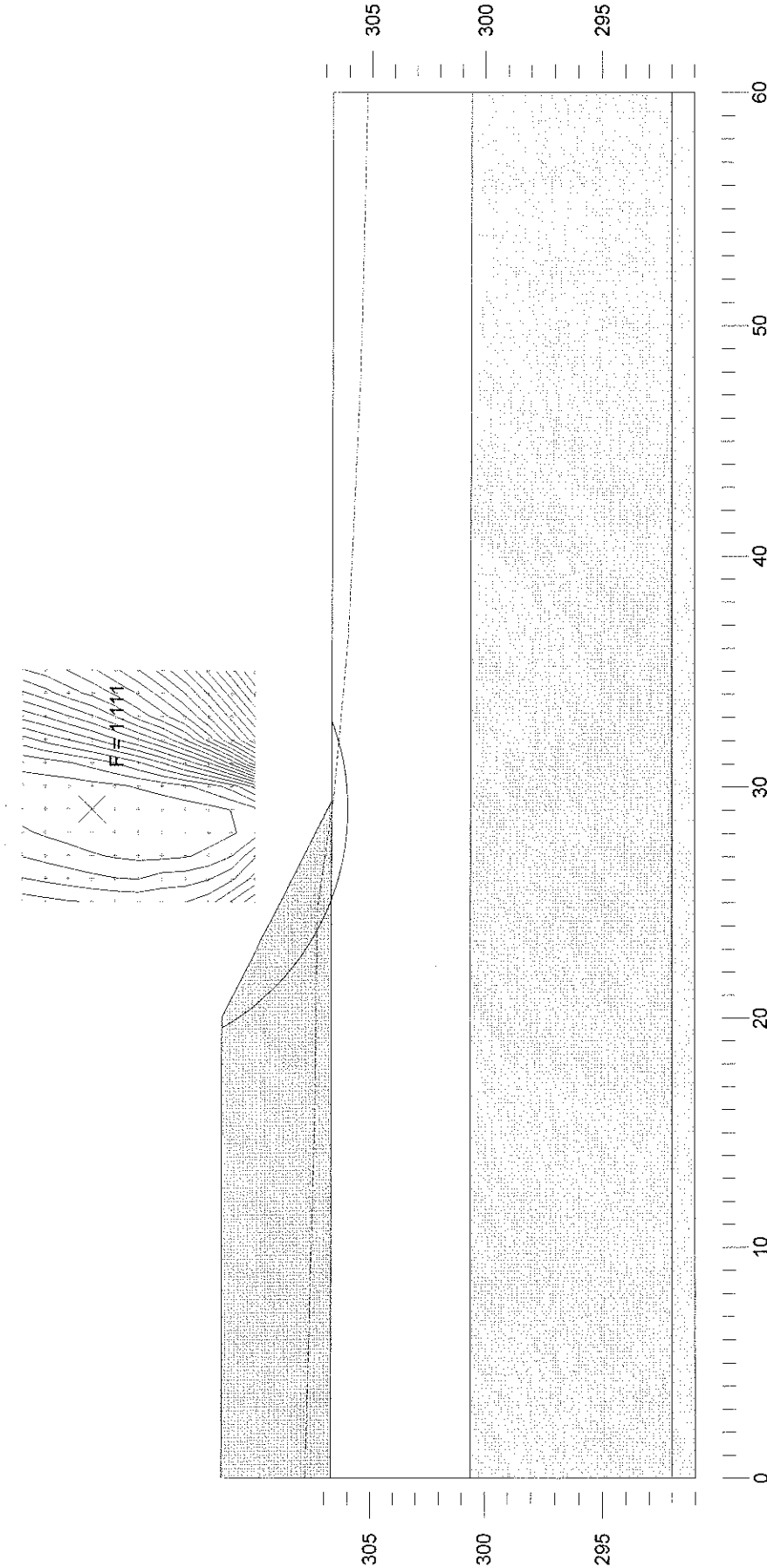


Figure E2C

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Rock Fill 1.25:1 Effective Stress Analysis

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Rock fill	20	0	42	1
Silty Sand	21	0	30	1
Clay	20	0	26	1
Silt	22	0	30	1

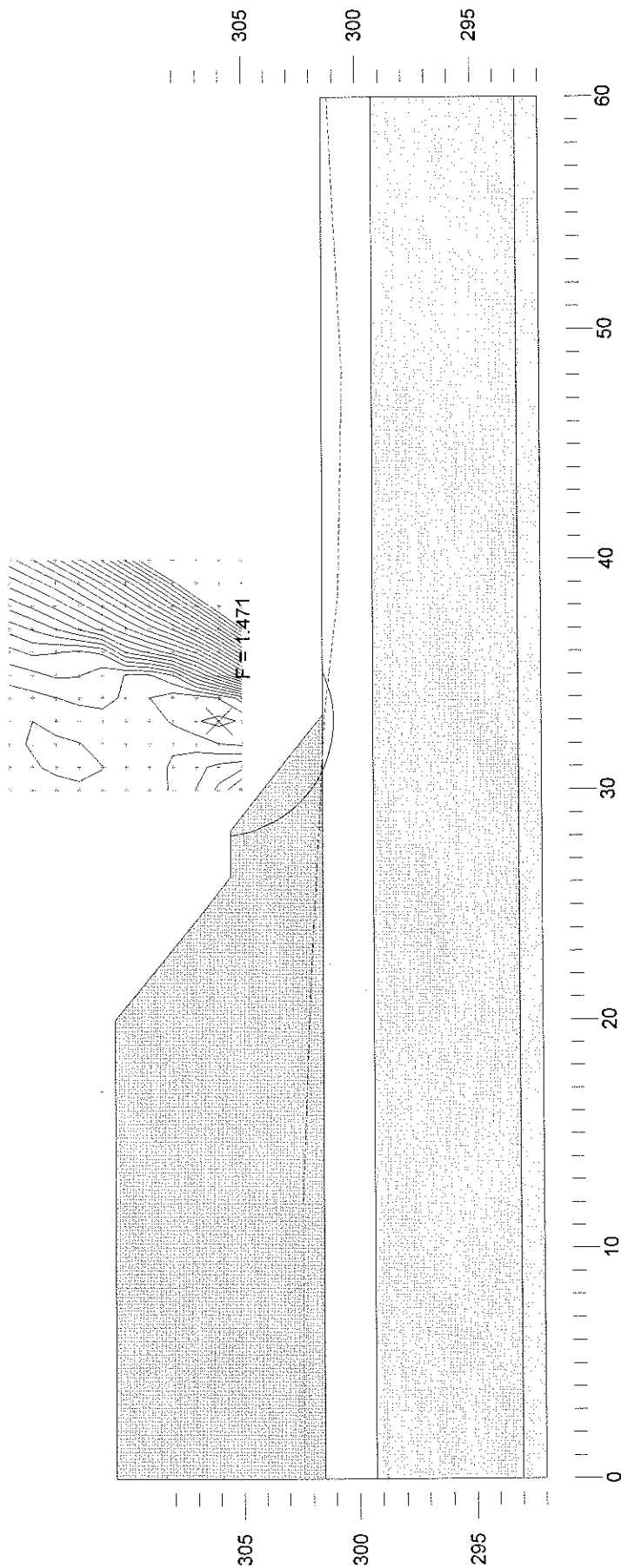


Figure E3A

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Rock Fill 1.25:1 Total Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock fill	20	42	1
Silty Sand	21	30	1
Clay	20	0	1
Silt	22	30	1

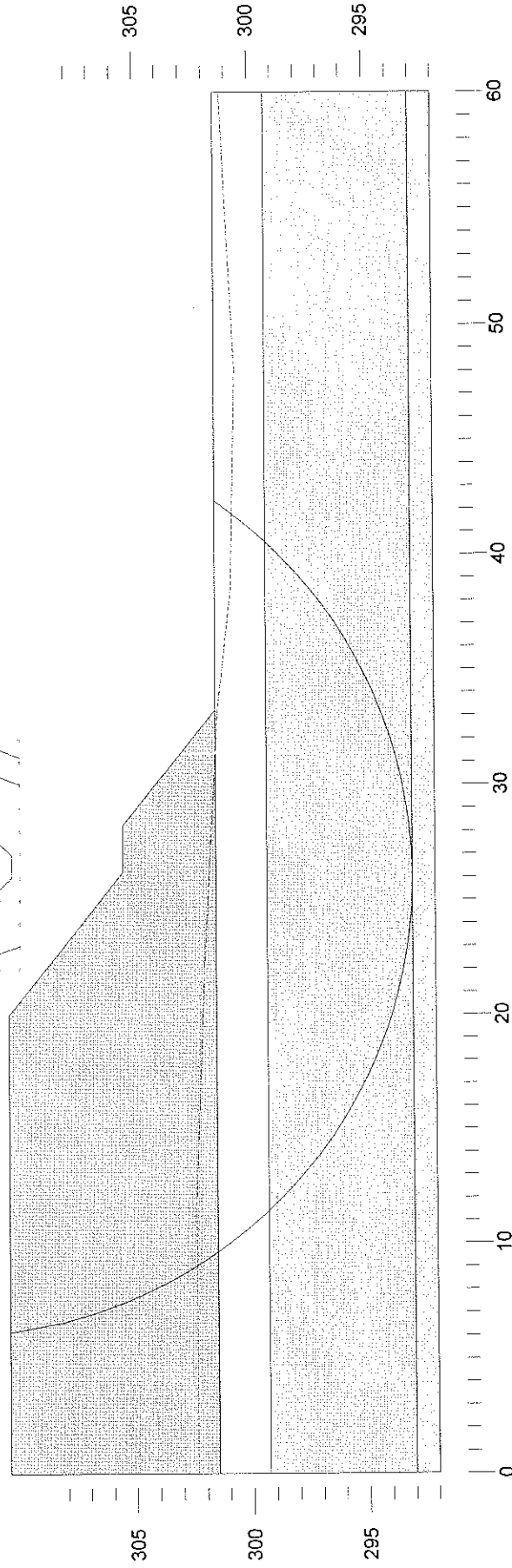
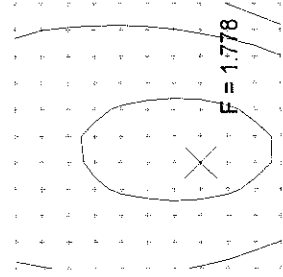


Figure E3B

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Rock Fill 1.25:1 Seismic Analysis

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Rock fill	20	0	42	1
Silty Sand	21	0	30	1
Clay	20	0	26	1
Silt	22	0	30	1

Seismic coefficient = 0.08

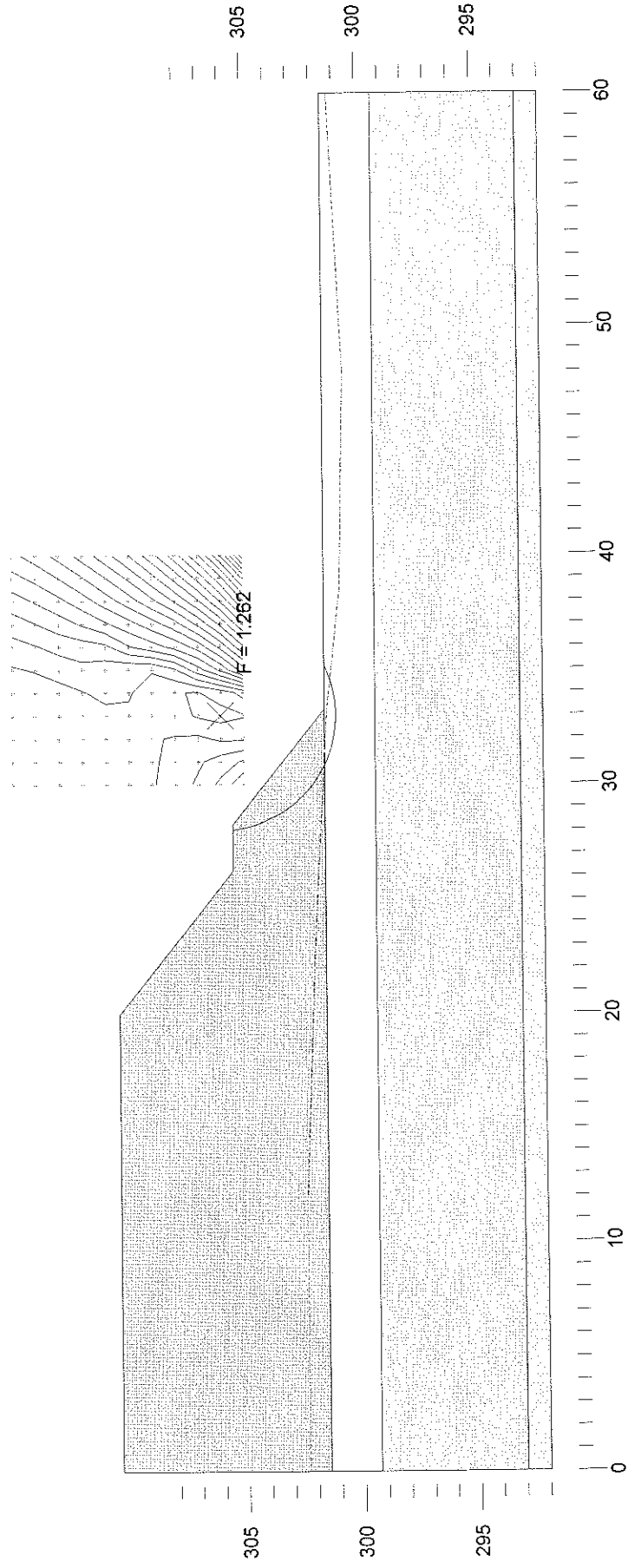


Figure E3C

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Earth Fill 2:1 Effective Stress Analysis

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Earth Fill	22	0	30	1
Silty Sand	21	0	30	1
Clay	20	0	26	1
Silt	22	0	30	1

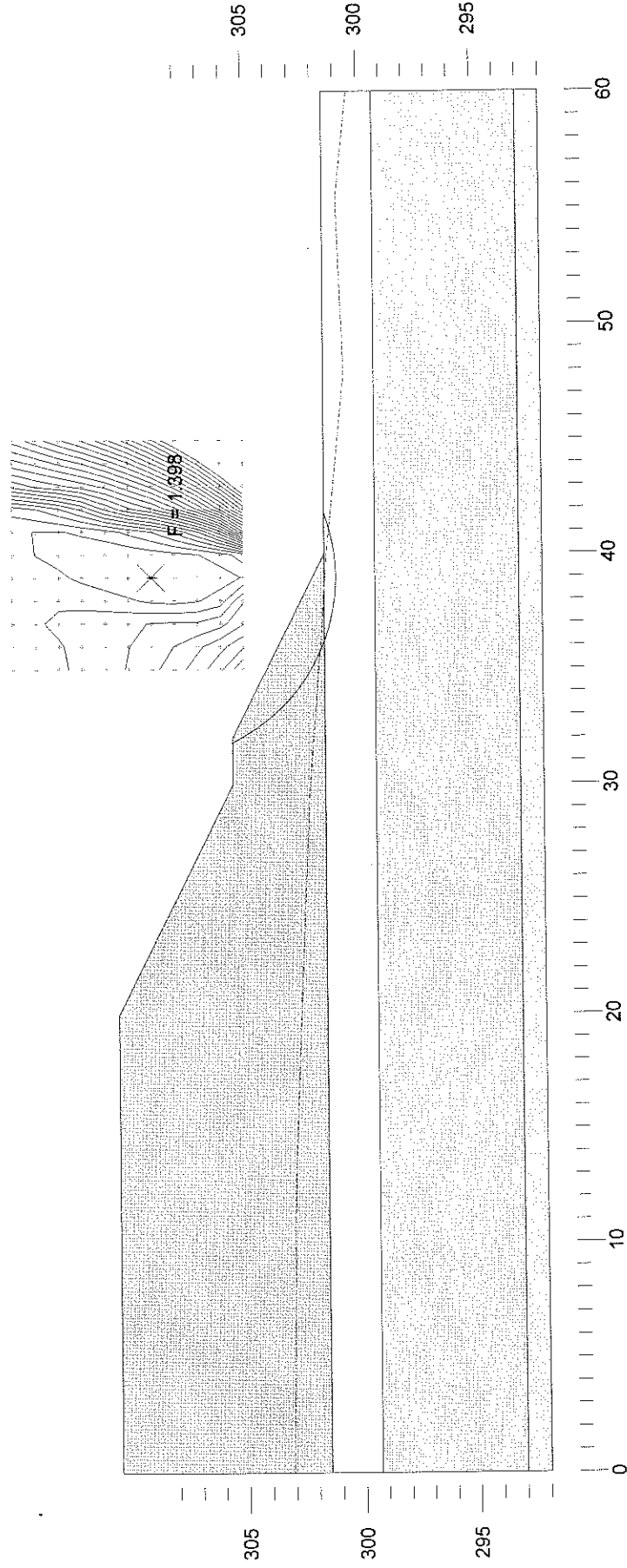


Figure E4A

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Earth Fill 2:1 Total Stress Analysis

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	22	0	1
Silty Sand	21	0	1
Clay	20	60	1
Silt	22	0	1

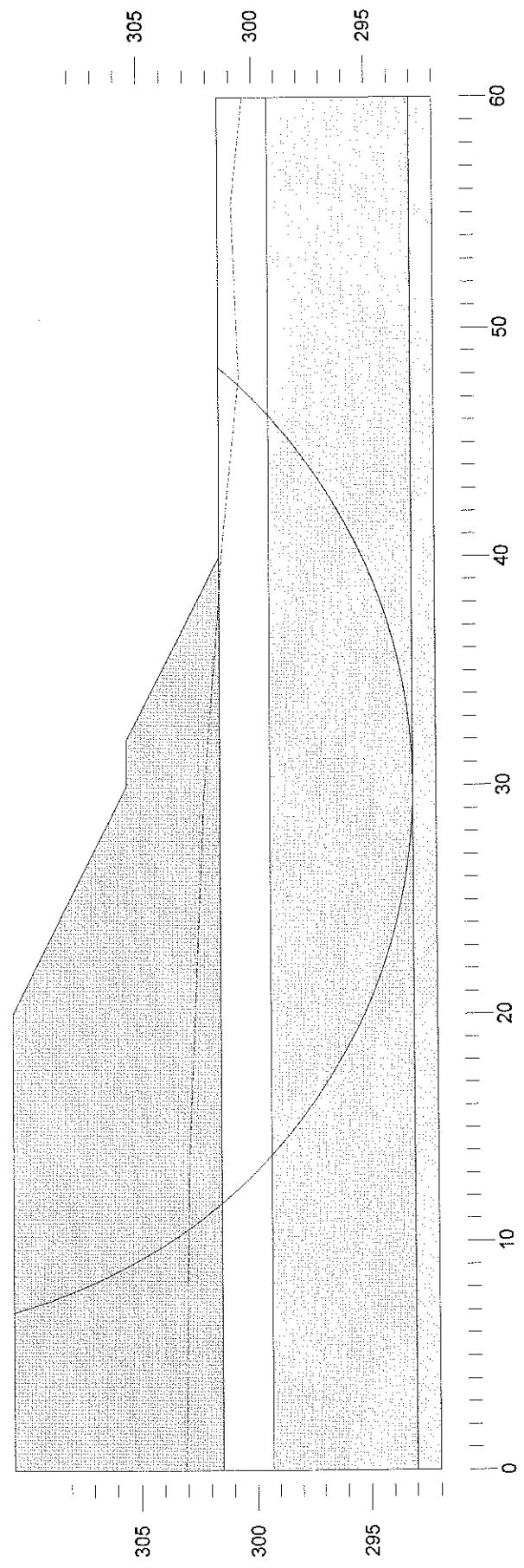
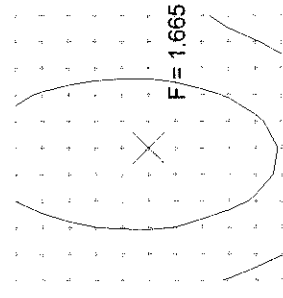


Figure E4B

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Jan 23, 2006
 TCPL NBL North Approach
 Earth Fill 2:1 Seismic Analysis

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Earth Fill	22	0	30	1
Silty Sand	21	0	30	1
Clay	20	0	26	1
Silt	22	0	30	1

Seismic coefficient = 0.08

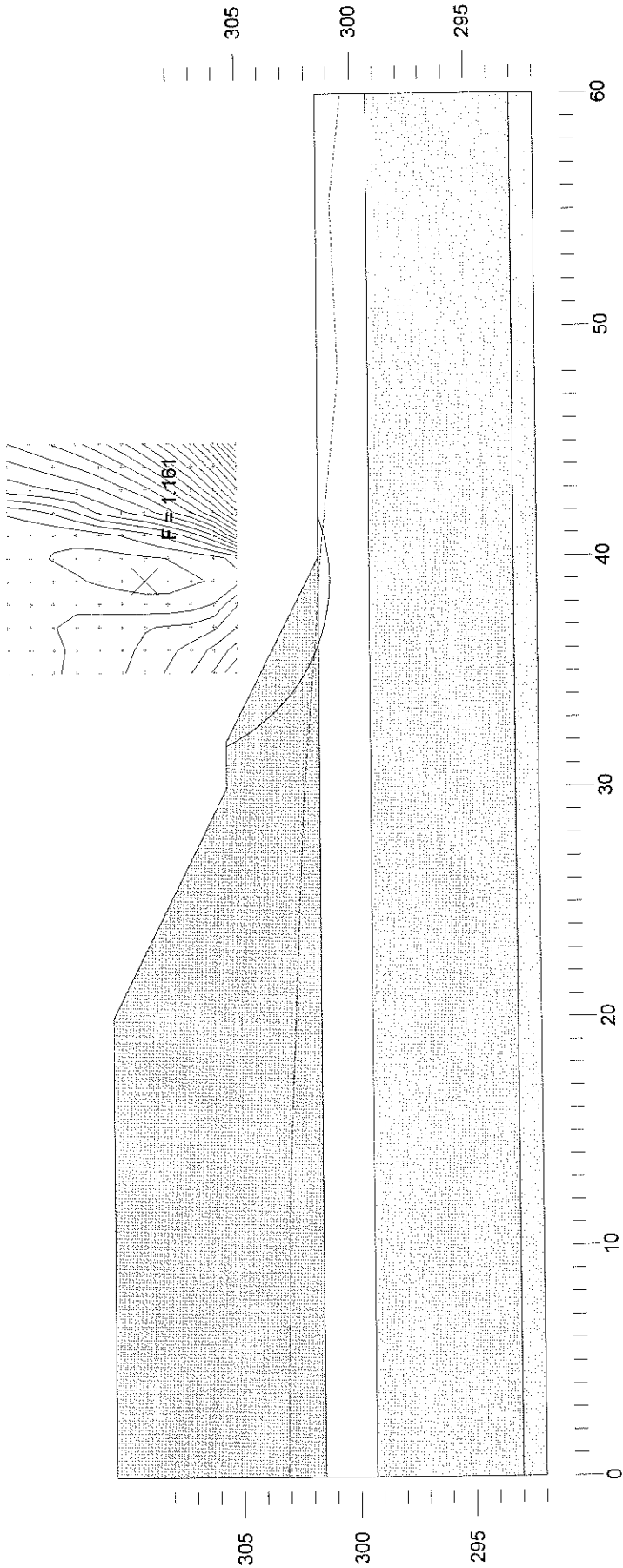


Figure E4C

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 Feb 3, 2005
 TCPL NBL North Approach RSS
 Earth Fill 2:1 Base

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Abutment	.01	33	1
RSS	22	33	1
Backfill	21	33	1
Earth Fill	22	30	1
Silty Sand	21	30	1
Gran A	23	35	1
Silty Sand	21	30	1
Clay	20	26	1
Silt	22	30	1

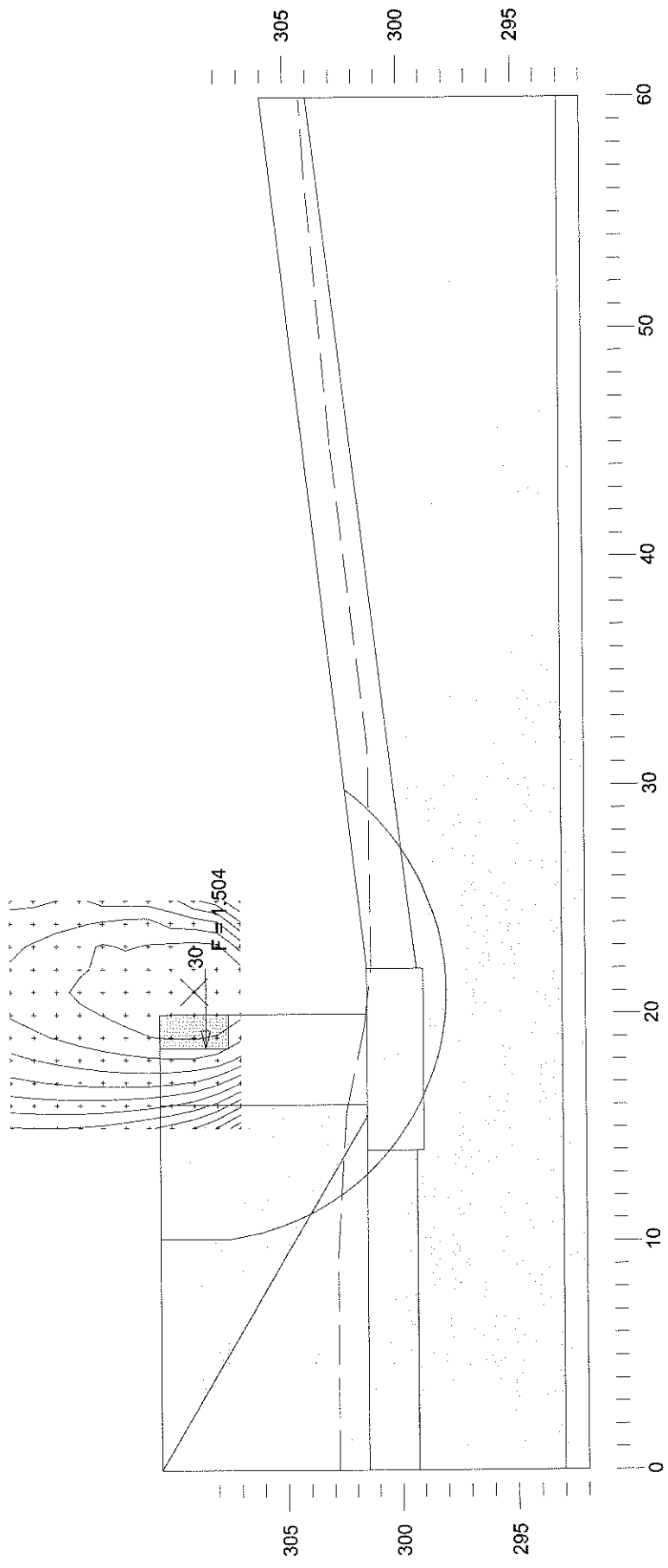


Figure E5

Appendix F

Drawings

