

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 11 SOUTHBOUND LANES OVER
MAGNETAWAN RIVER SOUTH CROSSING
HIGHWAY 11, HIGHWAY 518 WEST to HIGHWAY 520
G.W.P. 480-93-00, W.P. 361-00-01, SITE 44-122S**

Geocres Number: 31E-225

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 Southbound Lanes of the widened Highway 11 over the Magnetawan River at a point south the village of Katrine, Ontario.

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 from the data obtained in the course of the present investigation. This model describes the geotechnical conditions influencing design and construction of the foundations and approach embankments for the bridge and the stability of the riverbanks.

Thurber carried out the investigation 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 on existing Highway 11 where it crosses the Magnetawan River at a location south of the Village of Katrine, Armour Township. The site lies 2.8 km north of Highway 518 West and 1.0 km south of Three Mile lake Road. The new southbound lane centreline will be close to the existing highway centreline and the investigation was carried out in the vicinity of the existing bridge.

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. Locally, however, the site lies in the valley of the Magnetawan River, which is underlain by relatively deep deposits of glacio-fluvial and glacio-lacustrine soils.

The river has a broad, poorly defined flood plain at the site. The river channel is approximately 23 m wide and the maximum channel depth, based on May 2003 data, is 3 m. The riverbanks are low and no global stability problems were observed.

The site is occupied by the existing highway structure and approach embankments. Based on the General Arrangement Drawing for the existing bridge, dated 1959, it is assumed that the existing embankments have been in place for at least 45 years. The embankments are earth fill and constructed with side slopes of approximately 2H:1V.

The area north and south of the crossing is wooded and there are some buildings within the wooded area to the north of the river.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing for this project between October 22 and November 11, 2004.

The current site investigation consisted of drilling and sampling a total of six boreholes to depths of approximately 10 to 11 m at the approach fills and to depths of 35 to 37 m at the foundation locations. Dynamic cone penetration tests (DCPT) were carried out adjacent to two of the foundation boreholes and beyond the depth of sampling in the base of the two approach boreholes. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

Field layout for the site investigation was carried out by surveyors from Marshall Macklin Monaghan, who provided the coordinates and ground surface elevation data to Thurber.

A combination of 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 Testing (SPT). Cohesive soils were encountered in the boreholes and in these zones thin wall tube samples were collected and the shear strength of the soil was assessed by means of in-situ vane shear tests.

The positions of the principal 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
North Approach	122S-10
North Abutment	122S-8
North Pier	122S-7
South Pier	122S-4
South Abutment	122S-3
South Approach	122N-1

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.

A standpipe piezometer, consisting of 19 mm PVC pipe with slotted tips, was installed in two of the four deep boreholes drilled at the foundation elements to monitor the groundwater level.

The completion details for the piezometer are shown in Table 3.2.

Table 3.2 – Piezometer Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation	Completion Details
BH 122S-4	35.1/263.2	Piezometer with 1.5 m tip installed at bottom of borehole. Sand cave to 33.5, sand filter to 32.0, seal to 30.8, grout to the surface.
BH 122S-8	36.6/262.5	Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 33.1, seal to 32.5, grout to 2.4, bentonite seal to the surface.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The inspector logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and the results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

One sample of the cohesive soil was selected for one-dimensional consolidation testing at the laboratory of Golder Associates and the results are shown in Appendix B.

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 these appendices 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. None of the boreholes at this site encountered bedrock, instead terminating in the very dense sand. Locally, the surface soils have been reworked and re-deposited by the Magnetawan River.

In general terms, the site was found to be underlain by a thin veneer of topsoil, fill, a layer of sandy silt, silty clay; silt and sand, and sand containing cobbles and boulders.

More detailed descriptions of the individual strata are presented below.

5.2 Pavement Structure

The four boreholes drilled at foundation element locations encountered asphalt pavement that was typically 200 to 300 mm thick.

5.3 Fill

All boreholes encountered fill placed to construct the existing highway approach embankments.

The fill is described as sand, trace to some silt, trace to some gravel, with occasional cobbles. Based on SPT values ranging generally from 13 to 52 blows for 0.3 m of penetration, the fill has been classified as compact to very dense. Some lower SPT values were recorded, indicating loose to compact zones and, conversely, occasional higher values may indicate the presence of cobbles in the fill. The measured moisture content of the fill ranged from 2 to 18% and it is described as dry to moist.

In the area from the south approach through the south abutment and south pier, the fill extends to depth ranging from 3.7 to 4.4 m, or to Elevation 294.1 to 293.5.

In the area from the north approach through the north abutment and north pier, the fill extends to depths ranging from 3.0 to 4.4 m, or to Elevation 296.7 to 294.6.

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

5.4 Gravelly Sand

A layer of very dense gravelly sand, 1.2 m thick, was encountered at the north pier. This soil is brown and wet and is considered to be a thin deposit of river gravel.

5.5 Silt to Silty Sand

A layer of fine-grained non-cohesive soils ranging from silt to silty sand was encountered south of the river and extending from the south pier to the south approach. Based on SPT values ranging from 1 to 24 blows for 0.3 m of penetration, the deposit is classified as very loose to compact. The DCPT results at the south abutment indicate compact conditions.

The measured natural moisture contents range from 25 to 28% and the soil is described as wet. One very high reading of 187% may be attributable to an organic inclusion in the soil.

The layer of silt ranges in thickness from 2.4 m in the south approach to 4.2 m at the south abutment. The base of the layer lies between Elevation 291.7 at the south approach and Elevation 290.1 at the south pier.

The grain size distributions of selected samples of this soil are plotted on the record of Borehole sheets and shown in Figure B2 in Appendix B.

5.6 Silty Clay

The soils described above are underlain by a deposit of silty clay that extends across the entire site. Based on the recorded SPT values ranging from 3 to 29 blows for 0.3 m of penetration, the clay would be classified as firm to very stiff. However, taking account of the vane shear strengths which generally range from 50 to 80 kPa, the clay is in fact classified as stiff. One test at the south pier identified a zone of very stiff clay, vane strength of 110 kPa.

The clay is silty and layered, with the percentage of silt varying between layers. The plasticity of the clay ranges from low to high, as shown in Figures B8 and B9 in Appendix B.

The recorded natural moisture contents in the clay ranged from 29 to 56% and the soil is described as moist.

The thickness of the clay layer ranges from 3.1 to 3.8 m at the north approach and abutment to 12.8 m at the south abutment. The base of the clay layer lies at Elevation 293.6 at the north approach to Elevation 276.5 at the south abutment. The clay was not fully penetrated at the south approach and the borehole terminated in this layer.

The grain size distributions of selected samples of this soil are plotted on the record of Borehole sheets and shown in Figures B3 and B4 in Appendix B.

5.7 Silt

A layer of silt was encountered below the 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 9 to 27 blows for 0.3 m of penetration, the silt is classified as compact. Lower SPT values were recorded for some samples but these are attributed to sample disturbance due to unbalanced groundwater conditions at the base of the borehole.

The measured natural moisture contents ranged from 21 to 35% and the soil is described as moist.

The thickness of the silt layer ranged from 3.0 m at the north approach to 6.4 m at the south abutment. The base of the silt layer lay between Elevation 290.6 at the north approach and Elevation 270.1 at the south abutment.

The grain size distributions of selected samples of this soil are plotted on the record of Borehole sheets and shown in Figure B5 in Appendix B.

5.8 Sand

The silt layer is underlain by a layer of fine to medium grained sand that forms a substantial thickness at the north approach and north abutment but pinches out south of the river. Based on SPT values ranging generally from 14 to 35 blows for 0.3 m of penetration, this sand is classified as compact to dense. In one instance, a SPT value of 2 blows for 0.3 m of penetration was recorded but this is considered to be due to unbalanced groundwater conditions in the borehole resulting in sample disturbance.

The measured natural moisture contents ranged from 16 to 23% and the soil is described as wet.

The thickness of this soil layer varied from 10.2 m at the north pier to 17.4 m at the north abutment. The underside of the sand layer ranged from Elevation 271.3 at the north abutment to 271.0 at the north pier.

The grain size distributions of selected samples of this soil are plotted on the record of Borehole sheets and shown in Figure B6 in Appendix B.

5.9 Sand With Cobbles and Boulders

Below the fine grained sand described in the previous paragraph, the boreholes encountered a layer described as sand, trace gravel, some silt to silty, cobbles and boulders. Based on SPT values generally in excess of 100 blows for 0.3 m of penetration, this deposit is classified as very dense. Occasional lower values were recorded for samples in the upper levels of the deposit, indicating compact to dense conditions.

Where they could be measured, natural moisture contents ranged from 15 to 20% and the deposit is described as wet.

This deposit was not fully penetrated by any borehole but the thicknesses proved by sampling ranged from 8.9 m at the north abutment to 7.1 m at the north pier.

The grain size distributions of selected samples of this soil are plotted on the record of Borehole sheets and shown in Figure B7 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, are shown in Table 5.1.

Table 5.1 – Refusal Depths (Elevations)

Location	Borehole	Refusal Elevation (m)	Material
North Abutment	122S-8	268.5	Very dense sand with cobbles and boulders
North Pier	122S-7	267.0	
South Pier	122S-4	269.5	
South Abutment	122S-3	266.0	

5.11 Water Levels

The initial and final groundwater depths and elevations are shown in Table 5.2.

Table 5.2 – Groundwater Depths (in metres) and Elevations

Date	South Pier		North Abutment	
	Depth	Elevation	Depth	Elevation
October 28/04	-	-	3.8	295.3
November 3/04	3.6	294.7	3.6	294.5
November 11/04	3.6	294.7	4.0	295.1

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

6 MISCELLANEOUS

Surveying of the locations of the boreholes was carried out by staff from Marshall Macklin Monaghan.

The drill rig and sampling equipment used in the investigation were supplied and operated by All-Terrain Drilling of Waterloo, Ontario.

Full time supervision of field activities, including obtaining utility clearances was carried out by Mr. Mark Farrant, B.Sc. and Mr. Stephane Loranger, C.E.T. of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd. Consolidation testing was carried out by Golder Associates of Mississauga, Ontario.

Overall supervision of the field program, interpretation of the data 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.

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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 three-span, 90 m long, CPCI girder structure is proposed at this site and integral abutments are under consideration. A span configuration of 25:40:25 has been selected based on structural and hydraulic considerations, among others.

The south approach will lie on comparatively flat, low-lying land of the flood plain to the south of the river. The finished grade at the south abutment will lie at Elevation 300.4 and the existing highway grade is at Elevation 297.8, resulting in a 2.6 m high grade raise. The original ground surface at this location is estimated to be Elevation 295, giving a total embankment height of 5.4 m.

The finished grade at the north abutment will lie at Elevation 301.1 and the existing highway grade is at Elevation 299.1, resulting in a 2.0 m high grade raise. The original ground surface at this location is estimated to be Elevation 297, giving a total embankment height of 3.1 m.

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

8 STRUCTURE FOUNDATIONS

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme 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 25 to 30 m of generally loose to compact fill, sandy silt, firm to stiff silty clay, and sand overlying dense to very dense gravelly sand 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

8.1.1 Footings on Native Soil

The existing highway fill and the native silt and clay soils lying immediately below the fill are considered unsuitable for the support of spread footings. In addition to bearing resistance considerations, the risk of scour undermining spread footings at this site is considered to be high.

Accordingly spread footings founded on native soil were eliminated from further consideration.

8.1.2 Footings on Engineered Fill

These soil conditions are considered unsuitable for the support of an engineered fill pad due to the low bearing resistance available in the native soil underlying the engineered fill and the potential for comparatively large settlements. In addition, the risk of scour undermining spread footings at this site is considered to be high.

Accordingly spread footings founded on engineered fill pads were eliminated from further consideration.

8.2 Driven Steel Piles

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

The piles are expected to develop bearing resistance in the layer of very dense sand containing cobbles and boulders below Elevation 270.

The piles should be designed on the basis of the axial geotechnical resistances given in Table 8.1.

Table 8.1 – Pile Geotechnical Resistance

Pile Section	Piles Driven Into Sand with Cobbles and Boulders	
	ULS (Factored)	SLS (25 mm Settlement)
HP 310 X 110	1,800 kN	1,600 kN
HP 310 X 125	1,800 kN	1,600 kN
HP 360 X 132	2,100 kN	1,800 kN

The piles are expected to achieve the design bearing resistance at or below the elevations given in Table 8.2.

Table 8.2 – Pile Tip Elevations

Location	Borehole	Elevation (m)
North Abutment	122S-8	268.5
North Pier	122S-7	267.0
South Pier	122S-4	269.5
South Abutment	122S-3	266.0

The pile tip elevations shown in Table 8.2 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.3 Pile Installation.

8.2.1 Pile Tips

Due to the presence of cobbles and boulders in the expected founding layer, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or APF hard Bite or approved equivalent.

The use of rock points is recommended for the following reasons:

- The piles will be driven into soil containing cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock

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 presence of cobbles and boulders in the expected bearing stratum.

- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

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.

8.2.3 Pile Constructability

The pile groups required to support the piers may be driven partially or wholly in the river. The depth to the underside of the pile cap will be dictated by other studies, e.g. scour protection, but from preliminary information it is understood that the underside of the pile cap will lie about Elevation 290.0.

Based on the subsurface information gathered in the course of the investigation, excavation for the north pier pile cap will penetrate through the existing fill, some gravely sand and into the stratum of silty clay. At the south pier pile cap, the excavation will penetrate through existing fill, loose to compact silt and silty sand and end more-or-less at the top of the silty clay stratum. However, local variations in stratigraphy can occur between boreholes, particularly in a river setting, and it should not be assumed that the base of the excavation will necessarily lie in the silty clay.

Since the base of the pile cap will lie at a depth of approximately 3.5 m below the river level/groundwater level, unwatering the interior of the cofferdam to that level may lead to instability in the base of the excavation. The instability may take the form of boiling (a quicksand condition) or base heave, depending on the underlying soil conditions.

The following steps comprise one possible method to achieve construction of the pier pile caps within a stable excavation:

1. Drive an outer steel liner of sufficient diameter to accommodate driving the H-piles. The tip of the liner must be at least 1.0 m below the underside of the pile cap.
2. Excavate inside the outer liner to a level 0.5 m above the tip of the liner, keeping the liner flooded to approximately the river level to avoid base instability.
3. Drive the H-piles for the foundation.
4. Place an appropriately sized Sonotube (e.g. 1.5 m diameter) so that the tip is at least 0.5 m below the underside of the pile cap.
5. Place a minimum 0.5 m thick slab of concrete in the bottom of the Sonotube using tremie methods.

6. Allow the tremie concrete to harden and unwater the Sonotube.
7. Construct the pile cap and pier inside the unwatered Sonotube.
8. Remove the outer steel liner.

The foregoing procedure is illustrated schematically in the attached sketch Sk1, based on the preliminary GA for the structure. It is also intended primarily for design and estimating purposes. The Contractor should design the liner and unwatering scheme and may elect to implement a variation of the procedure that better suits his equipment, schedule and experience, provided it achieves the objective of constructing the pile cap and pier without destabilizing the river bank or creating unacceptable impacts on the river environment.

The design must take account of probable river levels during construction. In particular, the top of the liner must be set to be above the expected highest river level during construction.

8.2.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are approaching the bearing stratum below Elevation 270. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile".

"R" must have 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
HP 310X125	3,600 kN
HP360X132	4,200 kN

8.2.5 Downdrag

A layer of silty clay underlies the site and both primary and secondary consolidation settlements may occur as a result of the planned grade raise. 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			South Pier		
	HP 310x110	HP 310x125	HP 360x132	HP 310x110	HP 310x125	HP 360x132
Factored downdrag force (f = 1.25)	565 kN	565 kN	600 kN	565 kN	565 kN	600 kN
Pile Type	North Abutment			North Pier		
	HP 310x110	HP 310x125	HP 360x132	HP 310x110	HP 310x125	HP 360x132
Factored downdrag force (f = 1.25)	N/A	N/A	N/A	190 kN	190 kN	220 kN

Downdrag forces have been calculated assuming that the negative skin friction will be mobilized at the outside perimeter of the “H” pile, between the underside of the pile cap and the base of the silty clay layer.

Downdrag forces will only be generated at the piers if fill is placed around the piers. If the final grade is at or below existing, there is no anticipated downdrag issue.

8.2.6 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:

Non-cohesive

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

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

Cohesive

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

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

- where
- z = depth of embedment of pile in metres
 - D = pile width in metres
 - n_h = value from Table 8.5
 - S_u = undrained soil strength (MPa)
 - γ = unit weight (Table 8.5)
 - 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, $K = k_s * L * D$ (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, $P_{ult} = p_{ult} * L * D$. 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

Location	Elevation	n_h (kN/m ³)	K_p	S_u kPa	Unit Weight* (kN/m ³)	Soil Conditions
South Abutment	OGI to 293.5	2,000	3.3	-	21	Sand, loose to compact
	293.5 to 289.0	1,200	3.0	-	10	Silt, sandy, loose to compact
	289.3 to 280.0	-	2.8	50	9	Clay, silty, firm to stiff
	280.0 to 276.5	-	2.8	80	9	Clay, silty, stiff
	276.5 to 268.0	3,000	3.0	-	10	Silt, compact
North Abutment	Below 268.0	8,000	4.0	-	11	Sand, grave, cobbles, compact to very dense
	OGI to 296.0	2,000	3.3	-	21	Sand, gravel, compact
	296.0 to 292.0	-	2.8	50	19	Clay, silty, firm to stiff
	292.0 to 271.3	3,000	3.0	-	10	Silt, compact
	Below 271.3	8,000	4.0	-	11	Sand, grave, cobbles, boulders, very dense

*Buoyant 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.

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

When attempting to found caissons in the very dense sand, 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 preferred foundation system for all foundation elements at this site is steel H-piles driven to effective refusal as controlled by application of the Hiley formula.

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. At this site, the upper 3 m of the pile length will lie in sand fill, very loose sandy silt or silty clay which, in its original state, would provide sufficient flexibility. However, if the upper 3 m of the piles lies in compacted fill or if the native soil became compacted by the construction processes, the required flexibility may be compromised. 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.

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 $k(s) = 4,500 \cdot z/h \text{ kN/m}^3$

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

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.

Due to the proximity of the river, control of groundwater in an open excavation will be difficult and consideration may have to be given to excavating inside a cofferdam. The design of the cofferdam is the responsibility of the Contractor. The Contract Documents should alert him to the requirement to maintain a stable excavation and to the fact that any shoring system should be designed by a specialist, taking account of the need to control groundwater and prevent basal instability within the excavation.

10 UNWATERING

Based on the preliminary GA for the bridge structure, it is not expected that work at the abutments will require excavation below the groundwater level. However, the Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation.

The design of the dewatering system that may be required should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering should remain 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 sheet-pile cofferdam, where required.

11 APPROACH EMBANKMENTS

The global and internal stability of the approach embankments was analyzed for both rock fill and earth fill. The computer output for the stability analysis of the approach embankments is shown in Appendix E.

11.1 Long Term Stability

11.1.1 South Approach Stability

The soil conditions governing stability of the south approach embankment consist of the approach fill over approximately 4 m of sandy silt overlying silty clay. The groundwater level is at the base of the fill/top of the sandy silt.

Stability analysis was carried out using the proposed cross-section approximately 30 m south of the south abutment.

The analysis showed that a 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 analysis was repeated assuming water levels at the 100 year flood level (Elevation 296.2) and a seismic acceleration factor of 0.08 as a "worst case scenario" and a factor of safety of 1.1 was obtained. The theoretical failure plane intersected the side slope but did not intersect the road platform. These results indicate that the slope will reach a state of incipient failure under the combination of a 100 year flood and an earthquake. The risk of both conditions occurring together is considered to be low, and either condition occurring separately yielded a factor of safety higher than 1.0.

The same analyses were repeated for an earth fill approach embankment with side slopes constructed at 2H:1V. The resulting factors of safety are 1.7 under normal circumstances and 1.3 under the combined effects of the 100 year flood and 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.

11.1.2 North Approach Stability

The soil conditions governing stability of the north approach embankment consist of the approach fill overlying silty clay, which is in turn underlain by silt and silty sand. The groundwater level lies just below the base of the fill, in the silty clay layer.

Stability analysis was carried out using the proposed cross-section approximately 30 m north of the north abutment.

The analysis showed that a rock fill approach constructed with side slopes of 1.25H:1V have a factor of safety against slope failure of 1.3 under normal circumstances. The analysis was repeated assuming water levels at the 100 year flood level and a seismic acceleration factor of 0.08 as a "worst case scenario" and a factor of safety of 1.0 was

obtained. The theoretical failure plane extended to approximately 75% of the height of the side slope but did not intersect the road platform. These results indicate that the slope will only approach a state of incipient failure under the combination of a 100 year flood and an earthquake. The risk of both conditions occurring together is considered to be low, and either condition occurring separately yielded a factor of safety higher than 1.0.

The same analyses were repeated for an earth fill approach embankment with side slopes constructed at 2H:1V. The resulting factors of safety are 1.4 under normal circumstances and 1.2 under the combined effects of the 100 year flood and 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	Normal W.L. No Seismic	1.7	E1
Rock Fill	100 yr flood, 0.08 Seismic	1.1	E2
Earth Fill	Normal W.L. No Seismic	1.9	E3
Earth Fill	100 yr flood, 0.08 Seismic	1.3	E4
North Approach			
Rock Fill	Normal W.L. No Seismic	1.3	E5
Rock Fill	100 yr flood, 0.08 Seismic	1.0	E6
Earth Fill	Normal W.L. No Seismic	1.4	E7
Earth Fill	100 yr flood, 0.08 Seismic	1.2	E8

11.2 Construction Stage Stability

Computer output for the construction stage analysis is shown in Figures F9 through F12 in Appendix F.

11.2.1 South Approach

The stability of the embankment at the end of construction was checked by assuming:

- A pore pressure coefficient B_{bar} of 0.4 for the clayey silt and silt
- Undrained shear strength of 50 kPa in the silty clay
- That the entire embankment was placed at once.

The result of analysis gave a factor of safety of 1.3 for rock fill and 1.7 for earth fill. These are acceptable factors of safety and the factors in reality will be higher as the embankment will not be built “instantaneously”.

11.2.2 North Approach

The stability of the embankment at the end of construction was checked by assuming:

- A pore pressure coefficient B_{bar} of 0.4 for the clayey silt and silt
- Undrained shear strength of 50 kPa in the silty clay
- That the entire embankment was placed at once.

The result of analysis gave a factor of safety of 2.0 for rock fill and 2.1 for earth fill. These are acceptable factors of safety and the factors in reality will be higher as the embankment will not be built “instantaneously”.

11.3 Site Preparation for Rock Fill

The analyses described above were carried out using the proposed cross-sections and allowing for construction of the grade raise on top of the existing embankment.

If rock fill will be used, a 1.25V:1H side slope is permissible. However, the grading templates must be checked to ensure that the existing earth fill is totally enveloped within the rock fill. If necessary, the existing fill must be benched to ensure that the final toe of the existing fill lies at least 5.0 m inside the toe of the new rock fill slope.

Figures E5 and E6 show, schematically, the existing slope cut back to provide a sufficiently wide envelop of rock fill. Failure to do this may allow development of failure within the existing fill and invalidates the analysis that was conducted.

11.4 Settlement

Based on the fact that the existing embankment appears to have been constructed around 1960, it has been assumed that the primary consolidation of the silty clay layer under the existing approach fill loading has essentially been completed. The additional primary consolidation settlement to be expected at the abutments/immediate approaches as a result of the proposed grade raises are shown in Table 11.2.

Table 11.2 – Consolidation Settlement

Location	Settlement	Time Period*
South Abutment	270 mm	7 months
North Abutment	80 mm	5 months

* Time from the completion of construction.

If this magnitude of settlement or time delay is unacceptable, it is recommended that further analysis be carried out to determine the requirements for staged construction with surcharging to accelerate the time frame in which construction can proceed.

11.5 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.

11.6 Forward Slopes

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

In the case of earth fill slopes, rip rap protection should be provided and advice should be provided by a river hydrologist regarding potential scour forces to be resisted. This protection must be designed to prevent the river eroding beyond its existing channel or eroding the approach embankments.

11.7 Recommended Approach Fill

In view of the location of the south approach fill in the river flood plain, it is recommended that it be constructed using rock fill as this will be more resistant to the impact of flood water and does not require separate rip rap protection.

The north approach fill is expected to lie above the level of most floods and the choice of fill material is less important.

11.8 General Embankment Requirements

All topsoil and organic soils should be stripped from the footprint of the immediate 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 earth fill embankments are higher than 8 m, berms should be incorporated at a height of 8 m below the subgrade level. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be 2 m wide
- have 2% positive drainage to shed run-off water.

Where rock fill embankments are higher than 10 m, berms should be incorporated at a height of 10 m below the subgrade level. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide

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, typically, “High Appearance”. The geotechnical parameters that can be used for the design of RSS walls at this site are presented in Table 12.1.

Table 12.1 – RSS Design Parameters

Parameter	South Abutment	North Abutment
Bearing resistance on native soil	ULS _f = 280 kPa SLS = 180 kPa	ULS _f = 220 SLS = 140
Coefficient of sliding resistance	0.5	0.4
Estimated settlement	90 mm	42 mm

It is considered that the predicted magnitude of settlement is unacceptable for “High Performance, High Appearance”.

The consequence of these settlements could include, though not necessarily be limited to:

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

However, if other design requirements warrant over-riding this recommendation then the following ground preparation is required under the RSS mass:

1. 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.
2. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.
3. The highest permitted founding levels for the underside of the engineered fill are Elevation 292.0 at the south abutment and Elevation 293.5 at the north abutment. Lower founding elevation may be required to accommodate the required thickness of engineered fill.

4. Construction of the RSS mass as described in (1) through (3) above is expected to induce settlement in the underlying very loose to loose soils, especially at the south abutment. Any design of a RSS wall must take account of the possible settlement of the top of the wall. The magnitude of the settlement is difficult to predict accurately, but is estimated to be in the range of 20 to 40 mm. This settlement is not expected to affect the performance of the RSS wall but it may have an impact on the appearance.

If RSS is used, it must conform to the requirements of SP599S22.

13 BACKFILL TO ABUTMENTS

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

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 75 mm.

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 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 I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" of 1.0 should be used in seismic design.

15.2 Liquefaction Potential

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

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

At the south abutment, the soils surcharged by the approach fill are not in danger of liquefaction under earthquake loading. However, the near surface soils at the toe of the embankment may liquefy under earthquake loading and further study of this problem is recommended.

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.

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.

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

ϕ	= 35° for OPSS Granular A or Granular B Type II
ϕ	= 32° for OPSS Granular B Type I
ϕ	= 42° for rock fill
δ	= 50% of ϕ

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.

The stability of these slopes will be compromised if the soils at the toe of the embankment liquefy in the event of an earthquake. Initial indications are that the failure surface would extend only part way up the side slope and that the roadway should remain serviceable in the short term after an earthquake but that remedial work would be required to maintain the highway platform.

The slope stability issues must be revisited after detailed liquefaction studies have been completed.

16 CONSTRUCTION CONCERNS

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

- The possibility of piles reaching refusal on large boulders. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- Excavation and unwatering close to the river.
- Problems associated with installation of piles for piers that may be totally or partially in the river. The Contractor must exercise care in constructing the cofferdam (or outer liner) and the

tremie concrete plug to achieve an unwatered condition in which to construct the pile cap and pier.

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



Figure 1 displays 12 histograms, labeled (a) through (l), showing the distribution of the number of non-zero elements in the vector x for different values of n (1 to 12). The x-axis represents the number of non-zero elements, and the y-axis represents the count. The distributions are centered around 0, with the peak count increasing as n increases.

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

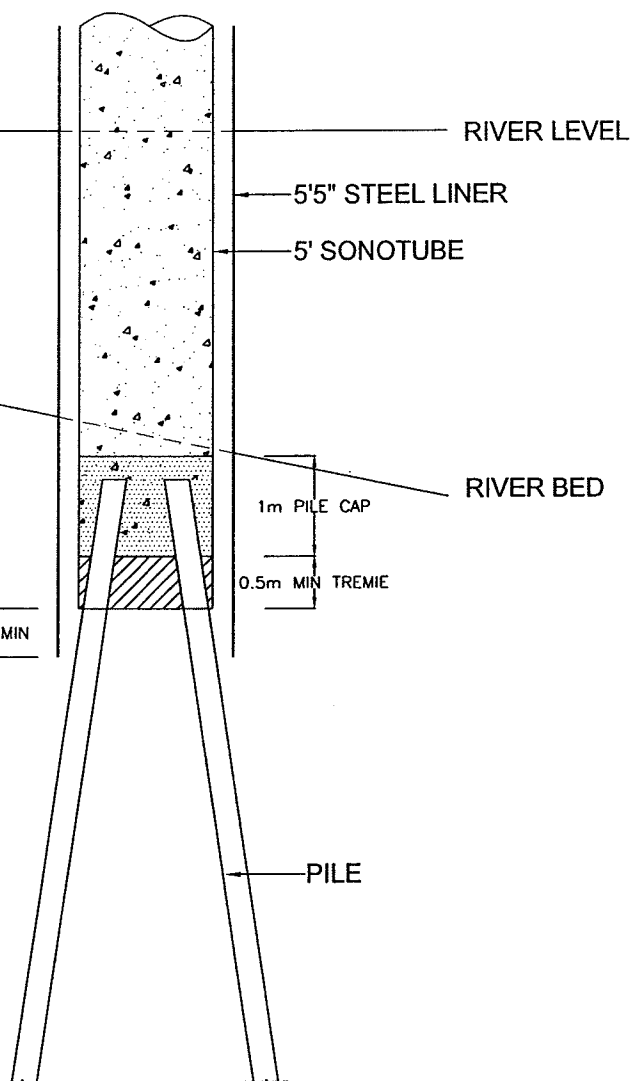
N.B.

WEIGHT AND SHEAR RESISTANCE
OF TREMIE PLUG MUST RESIST
FULL HYDROSTATIC UPLIFT.

NOTES:

- 1)* DRIVE STEEL LINER AT LEAST 0.5m
BEYOND PROPOSED TIP OF SONOTUBE
AND AT LEAST 2m INTO RIVER BED
- 2) MUCK OUT TO REQUIRED BASE OF TREMIE
PLUG
- 3) DRIVE PILES
- 4)* INSTALL SONOTUBE
- 5) PLACE TREMIE CONCRETE PLUG UNDER
WATER AND LET HARDEN
- 6) DEWATER INSIDE SONOTUBE
- 7) CUT TOP OF PILES TO REQUIRED ELEVATION
- 8) INSTALL REINFORCING AND CONCRETE
PILE CAP
- 9) INSTALL REINFORCING AND CONCRETE PIER
TO ABOVE RIVER LEVEL
- 10) REMOVE TEMPORARY STEEL LINER

* LINER TO BE ABOVE RIVER LEVEL AND
SONOTUBE ALSO TO EXTEND ABOVE
RIVER LEVEL

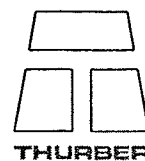


RISKS:

- A) BOILING OF BASE IF LINER MUCKED OUT TOO FAST
- B) DANGER OF LINER GRABBING SONOTUBE/
PILE CAP ON REMOVAL

ENGINEER	AEG
DRAWN	HS
DATE	DEC , 2004
APPROVED	
SCALE	NTS

INSTALLATION OF PIER FOUNDATION IN RIVER
(FOR ILLUSTRATION ONLY)



THURBER

DWG. NO.

SK1

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
 Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		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 122S-1

1 OF 2

METRIC

W.P. 361-00-01 LOCATION N 5047485.6 E 316703.6 Magnetawan South, SBL 122S-1 ORIGINATED BY MF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
 DATUM Geodetic DATE 29.10.04 - 29.10.04 CHECKED BY SS/AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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 (%)
								20 40 60 80 100						
								20 40 60 80 100						
297.8														
0.0	SAND, fine grained, some gravel, trace silt Compact to Dense Brown Dry to Moist (FILL)		1	SS	19									
			2	SS	30									

Continued Next Page

+³ × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 122S-1

2 OF 2

METRIC

W.P. 361-00-01 LOCATION N 5047485.6 E 316703.6 Magnetawan South, SBL 122S-1 ORIGINATED BY MF
HWY 11 BOREHOLE TYPE Hollow Stem Augers / Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
DATUM Geodetic DATE 29.10.04 - 29.10.04 CHECKED BY SS/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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100	○ UNCONFINED + FIELD VANE						
								20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE						
										20 40 60					

METRIC

[illegible][illegible]




+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 122S-1A

2 OF 3

METRIC

W.P. 480-93-00 LOCATION Magnetawan South, SBL, 122S-1A ORIGINATED BY WRW
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test (DCPT) COMPILED BY WM
 DATUM Geodetic DATE 05.04.05 - 05.04.05 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 (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE					w _p w w _L		
								● QUICK TRIAXIAL × LAB VANE							
							20 40 60 80 100								
287.4	SAMPLING STARTED AT 10.4m.														
10.4	Silty CLAY Very Stiff Grey		1	SS	9										
			1	TW	PH										
285.2															
12.6	Sandy SILT, some clay to clayey Compact to Very Loose Grey														
			2	SS	14										
			3	SS	7										
			4	SS	0										
			5	SS	4										
279.2															
18.6	END OF SOIL SAMPLING AT 18.59 m. DCPT STARTED AT 18.59 m.														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 122S-1A

3 OF 3

METRIC

W.P. 480-93-00 LOCATION Magnetawan South, SBL, 122S-1A ORIGINATED BY WRW
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test (DCPT) COMPILED BY WM
 DATUM Geodetic DATE 05.04.05 - 05.04.05 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100	○ UNCONFINED + FIELD VANE						
								20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE						
274.0															
23.8	END OF DCPT AT 23.77 m. WATER LEVEL AT 6.1 m UPON COMPLETION OF DRILLING. BOREHOLE GROUTED WITH BENSEAL BENTONITE GROUT TO SURFACE.														

RECORD OF BOREHOLE No 122S-3

1 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047515.4 E 316699.4 Magnetawam South, SBL 122S-3 ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
 DATUM Geodetic DATE 09.11.04 - 10.11.04 CHECKED BY SS/AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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	× LAB VANE			
297.8							20 40 60 80 100					
0.0	ASPHALT (263 mm)											
297.5												
0.3	SAND and GRAVEL (FILL)											
297.1												
0.7	SAND, trace gravel, trace silt Dense to Loose Brown Moist (FILL)											
			1	SS	30							
			2	SS	8							
293.5												
4.3	Sandy SILT, Loose to Compact Grey Wet		3	SS	4							
			4	SS	22							
			5	SS	24							
289.3												
8.5	Silty CLAY Stiff to Firm Grey		6	SS	15							

Continued Next Page

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

ONTMT4 122N&S.GPJ 14/12/04

RECORD OF BOREHOLE No 122S-3

2 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047515.4 E 316699.4 Magnetawam South, SBL 122S-3 ORIGINATED BY SL
HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
DATUM Geodetic DATE 09.11.04 - 10.11.04 CHECKED BY SS/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							
								20 40 60 80 100		20 40 60						
			7	SS	5		287							0 1 53 47		
								3.7 +								
			1	TW	PH		286									
							285									
			8	SS	4		284									
								2.4 +								
							283									
			9	SS	4		282							0 1 38 61		
							281									
			10	SS	6		280									
								2.7 +								
							279									
			11	SS	10		278									

Continued Next Page

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

RECORD OF BOREHOLE No 122S-3A

2 OF 3

METRIC

W.P. 361-00-01 LOCATION N 5047515.4 E 316699.4 Magnetawan South, SBL 122S-3A ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
 DATUM Geodetic DATE 11.11.04 - 11.11.04 CHECKED BY SS/AEG

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			SHEAR STRENGTH kPa		W _p	W	W _L		
							20 40 60 80 100	20 40 60 80 100	20 40 60					
							287							
							286							
							285							
							284							
							283							
							282							
							281							
							280							
							279							
							278							

Continued Next Page

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

RECORD OF BOREHOLE No 122S-4

1 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047529.0 E 316683.5 Magnetawan South, SBL 122S-4 ORIGINATED BY SL/MF
HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
DATUM Geodetic DATE 28.10.04 - 02.11.04 CHECKED BY SS/AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							20 40 60							
298.3														
0.0	ASPHALT (300 mm)													
298.0														
0.3	SAND, fine grained, some gravel, trace silt Very Dense to Compact Brown Moist (FILL)						298							
			1	SS	73		297							
							296							
			2	SS	13		295					4 93 4 (SI+CL)		
293.9							294							
4.4	Silty SAND, fine grained Very Loose to Loose Brown to Grey Moist to Wet		3	SS	1		293							
							292					0 69 28 3		
			4	SS	6		291							
							290							
			5	SS	8		289							
290.1														
8.2	Silty CLAY Stiff to Firm Grey													
			6	SS	9									

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+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

ONTMT4S 122N&S.GPJ 16/12/04

RECORD OF BOREHOLE No 122S-4

2 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047529.0 E 316683.5 Magnetawan South, SBL 122S-4 ORIGINATED BY SL/MF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
 DATUM Geodetic DATE 28.10.04 - 02.11.04 CHECKED BY SS/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			20 40 60 80 100	20 40 60 80 100					
	occasional silt layers		7	SS	6		288							
							287	3.5						
			8	SS	4		286			4				0 0 55 45
							285							
			9	SS	3		284	3						
							283							
			10	SS	6		282							
							281							
			11	SS	7		280							0 0 61 38
							279	3.1						
			12	SS	8									
279.1														
19.2	Clayey SILT, occasional sand seams Stiff to Very Stiff Grey													

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

ONTMT4 122N&S.GPJ 14/12/04

RECORD OF BOREHOLE No 122S-4

3 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047529.0 E 316683.5 Magnetawan South, SBL 122S-4 ORIGINATED BY SL/MF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
 DATUM Geodetic DATE 28.10.04 - 02.11.04 CHECKED BY SS/AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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		
276.7			13	SS	24		278							
21.6	SILT, some sand, trace clay Compact Grey Wet						277							
			14	SS	19		276							
							275							0 13 81 5
							274							
							273							
			15	SS	24		272							
271.5							271							
26.8	SAND, trace silt, trace gravel, occasional cobbles and boulders Very Dense Grey Wet						270							
	200 mm boulder encountered													
	100 mm boulder encountered		16	SS	60/		269							
					.100									

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+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC[illegible][illegible]

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 122S-7

2 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047573.8 E 316664.2 Magnetawan South, SBL 122S-7 ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / NW Casing COMPILED BY WM/SS
 DATUM Geodetic DATE 02.11.04 - 08.11.04 CHECKED BY SS/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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
							289							
			6	SS	8		288							0 0 67 33
							287							
			7	SS	7		286							
							285							
			8	SS	29		284							
284.4							283							
14.6	SILT, trace sand, trace clay Compact Grey Wet		9	SS	27		282							0 2 91 7
							281							
			10	SS	15		280							
281.2														
17.8	SAND and SILT Dense Grey Wet		11	SS	30									
	Becoming Compact													

Continued Next Page

+³ × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

W.P.	361-00-01	LOCATION	N 5047573.8 E 316664.2 Magnetawan South, SBL 122S-7	ORIGINATED BY	SL
HWY	11	BOREHOLE TYPE	Hollow Stem Augers / NW Casing	COMPILED BY	WM/SS
DATUM	Geodetic	DATE	02.11.04 - 08.11.04	CHECKED BY	SS/AEG

[illegible]

(%) STRAIN AT FAILURE

METRIC

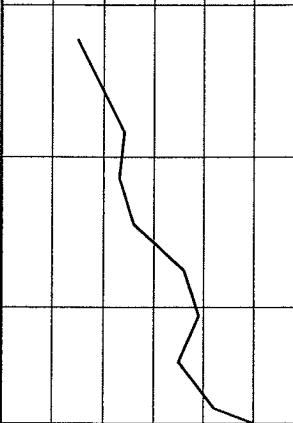
[illegible]

RECORD OF BOREHOLE No 122S-7A

1 OF 1

METRIC

W.P. 361-00-01 LOCATION N 5047573.8 E 316664.2 Magnetawan South, SBL 122S-7A ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
 DATUM Geodetic DATE 11.11.04 - 11.11.04 CHECKED BY SS/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							
299.0								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
0.0	DCPT started from surface.						299								
							298								
							297								
296.2															
2.8	END OF DCPT AT 2.84 m.														

ONTMT4 122N&S.GPJ 14/12/04

RECORD OF BOREHOLE No 122S-8

1 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047590.4 E 316647.0 Magnetawan South, SBL 122S-8 ORIGINATED BY MF
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM/SS
DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY SS/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										
299.1							20	40	60	80	100	20	40	60				
0.0	ASPHALT (275 mm)																	
298.8																		
0.3	Gravelly SAND, trace silt, trace cobbles Compact to Loose Brown Moist (FILL)																	
			1	SS	17													
295.9			2	SS	6													
3.2	Silty CLAY, trace to some sand Firm Grey-Brown																	
			3	SS	6										0 2 57 42			
			4	SS	7													
292.1																		
7.0	SILT, some sand, trace clay Compact to Loose Grey Wet																	
			5	SS	14										0 19 77 3			
			6	SS	2													

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 Φ 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 122S-8

2 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047590.4 E 316647.0 Magnetawan South, SBL 122S-8 ORIGINATED BY MF
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM/SS
DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY SS/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						
288.7	SAND, fine grained, some to trace silt Compact to Very Dense Grey Wet						289							0 75 25 (SI+CL)		
10.4			7	SS	15		288									
							287									
			8	SS	15		286									
							285									
			9	SS	23		284									
							283									
			10	SS	21		282									
							281									
			11	SS	24		280									
					12	SS	30									0 80 20 (SI+CL)

ONT/TMT4S 122N&S.GPJ 16/12/04

Continued Next Page

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

METRIC

W.P.	361-00-01	LOCATION	N 5047590.4 E 316647.0	Magnetawan South, SBL 122S-8	ORIGINATED BY	MF
HWY	11	BOREHOLE TYPE	Hollow Stem Augers		COMPILED BY	WM/SS
DATUM	Geodetic	DATE	25.10.04 - 28.10.04		CHECKED BY	SS/AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT										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					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L						
						20	40	60	80	100	20	40	60					
			13	SS	35													
			14	SS	21													
			15	SS	26													
271.3																		
27.8	SAND, trace silt, trace gravel, occasional cobbles and boulders Very Dense Grey Wet																	
			16	SS	74													

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 122S-8

4 OF 4

METRIC

W.P. 361-00-01 LOCATION N 5047590.4 E 316647.0 Magnetawan South, SBL 122S-8 ORIGINATED BY MF
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM/SS
DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY SS/AEG





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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												
							20	40	60	80	100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	WATER CONTENT (%)					
							20	40	60	80	100			20	40	60				
							269													
			17	SS	66/ .200														16 74 10 (SI+CL)	
							268													
			18	SS	118/ .225		267													
							266													
			19	SS	100/ .075		265													
							264													
			20	SS	100/ .075		263													
262.4			21	SS	110/ .100															
36.7	END OF BOREHOLE AT 36.68 m. Piezometer installation consist of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 28-OCT-04 3.8 03-NOV-04 3.6 11-NOV-04 4.0																			

RECORD OF BOREHOLE No 122S-10

1 OF 2

METRIC

W.P. 361-00-01 LOCATION N 5047615.9 E 316642.3 Magnetawan South, SBL 122S-10 ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
 DATUM Geodetic DATE 22.10.04 - 22.10.04 CHECKED BY SS/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			20 40 60 80 100										
								SHEAR STRENGTH kPa										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
							20 40 60 80 100				WATER CONTENT (%)				GR	SA	SI	CL
299.7																		
0.0	SAND and GRAVEL, trace silt Dense Brown (FILL)		1	SS	48													
			2	SS	45													
			3	SS	30													
			4	SS	32													
296.7																		
3.0	Silty CLAY, occasional silt layers Stiff to Very Stiff Grey		5	SS	11													
			6	SS	20													
293.6																		
6.1	SILT Loose to Compact Grey Wet		7	SS	9													
			8	SS	15													
290.6																		
9.1	SILT and SAND, to Silty SAND Very Loose Grey Wet		9	SS	2													

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

ONTMT4 122N&S.GPJ 14/12/04

RECORD OF BOREHOLE No 122S-10

2 OF 2

METRIC

W.P. 361-00-01 LOCATION N 5047615.9 E 316642.3 Magnetawan South, SBL 122S-10 ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/SS
 DATUM Geodetic DATE 22.10.04 - 22.10.04 CHECKED BY SS/AEG

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			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100										
289.0																		
10.7	Silty SAND Compact Grey Wet		10	SS	14													
286.6																		
13.1	END OF BOREHOLE AT 13.11 m. BOREHOLE OPEN TO 10.67 m AND WATER LEVEL AT 10.06 m UPON COMPLETION. BOREHOLE GROUTED TO SURFACE.																	

ONT/T4 122N&S.GPJ 14/12/04

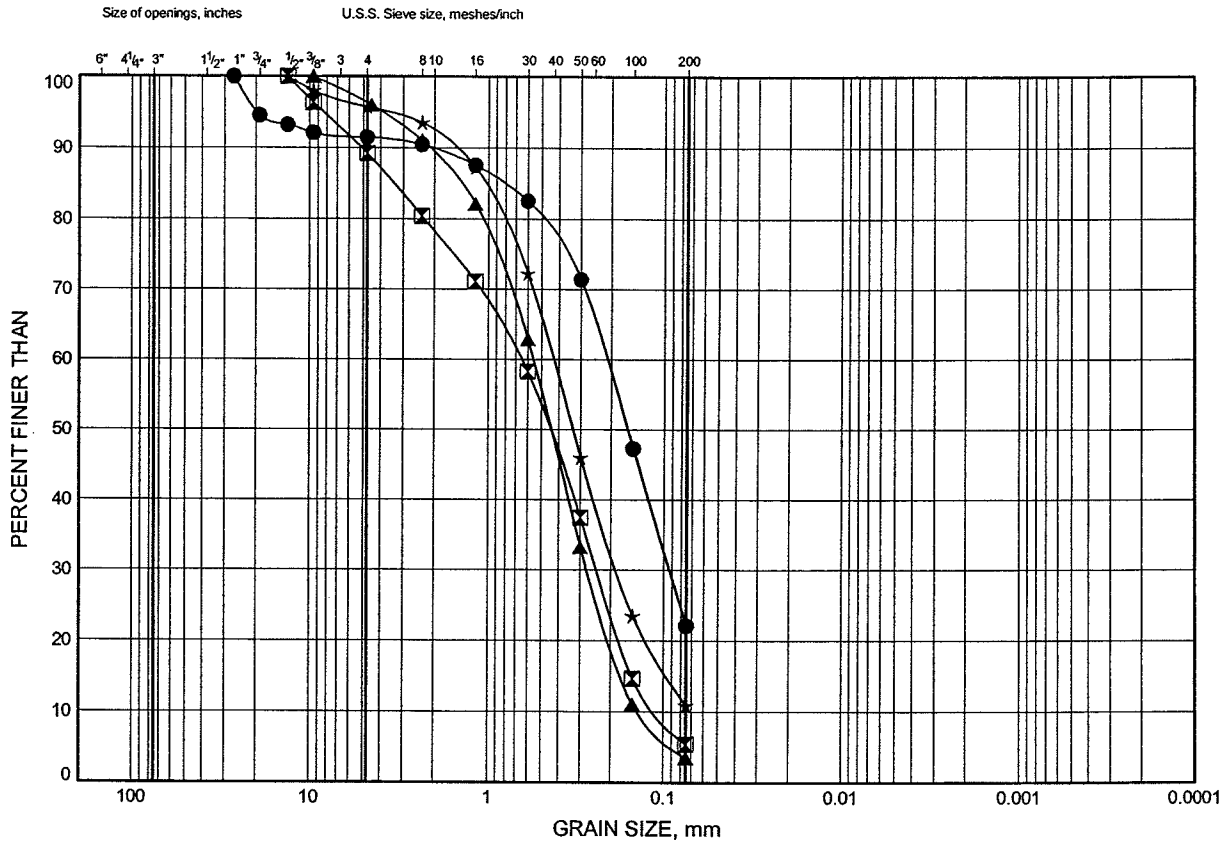
Appendix B

Laboratory Test Results

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL
(SAND, TRACE TO SOME SILT, TRACE TO SOME GRAVEL)

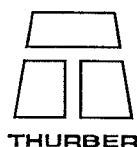


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-1	1.83	295.97
⊠	122S-10	1.07	298.63
▲	122S-4	3.05	295.25
★	122S-7	3.35	295.65

Date December 2004

Project 361-00-01



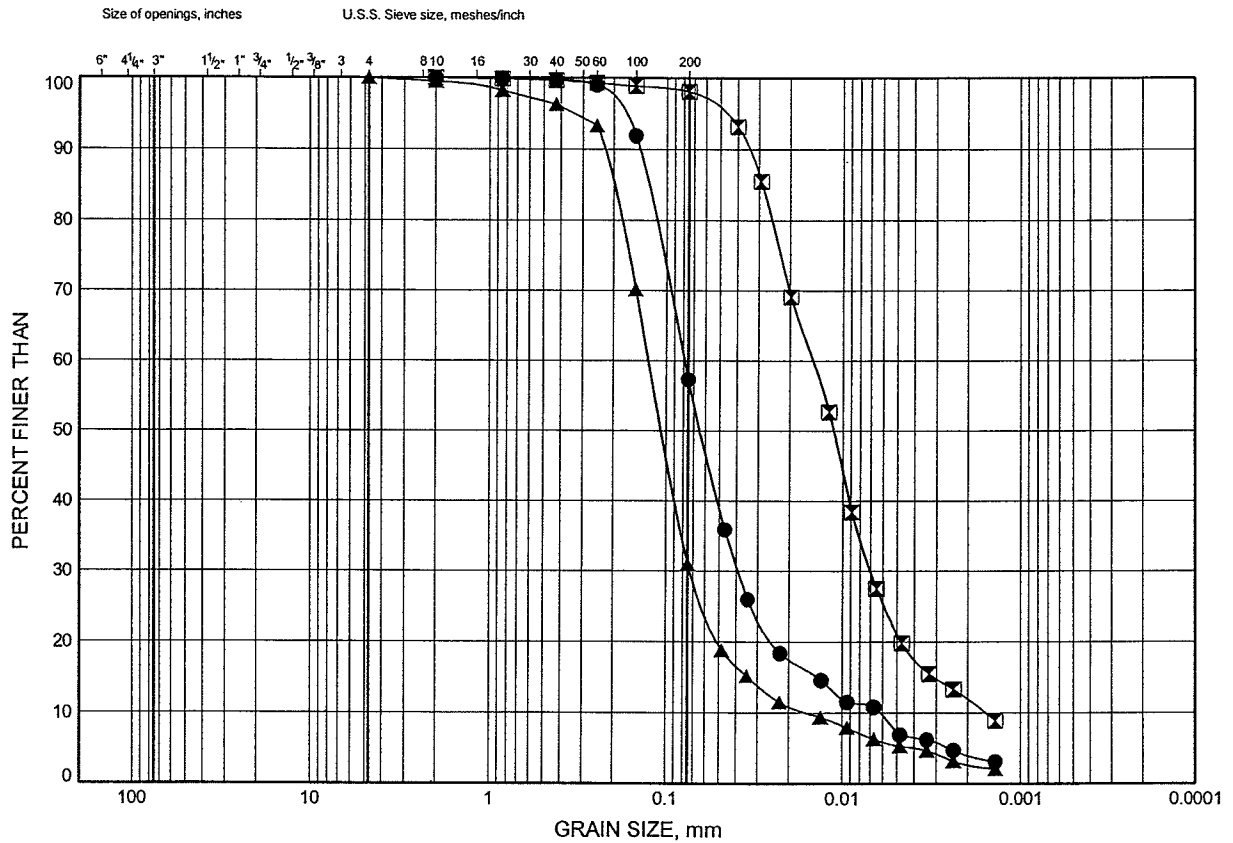
Prep'd SS

Chkd. AEG

Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B2

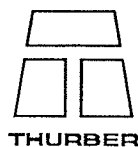
SANDY SILT TO SILTY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-3	4.88	292.92
⊠	122S-3	7.77	290.03
▲	122S-4	6.10	292.20

Date December 2004
Project 361-00-01

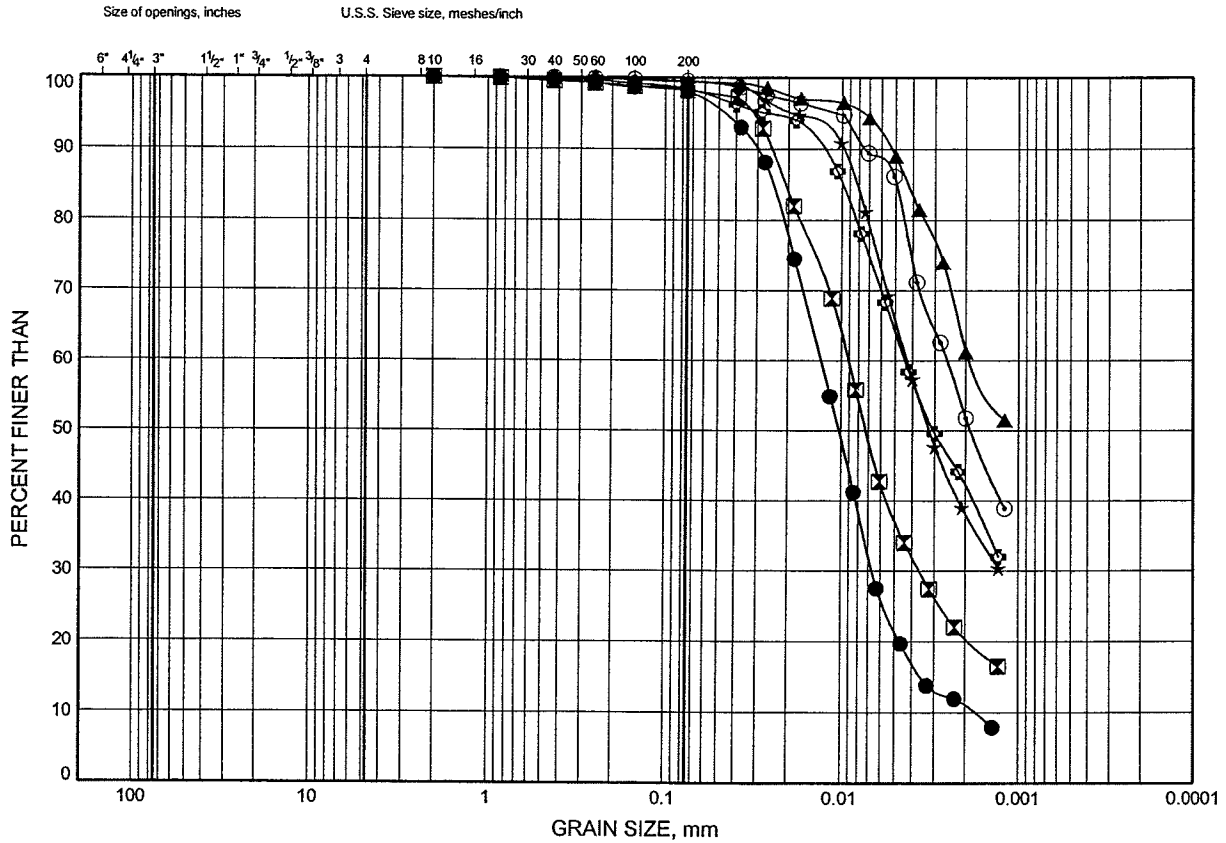


Prep'd SS
Chkd. AEG

Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B3

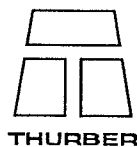
SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-1	6.40	291.40
⊠	122S-10	3.35	296.35
▲	122S-3	15.39	282.41
★	122S-4	16.92	281.38
⊙	122S-7	7.77	291.23
⊛	122S-8	4.72	294.38

Date December 2004
Project 361-00-01

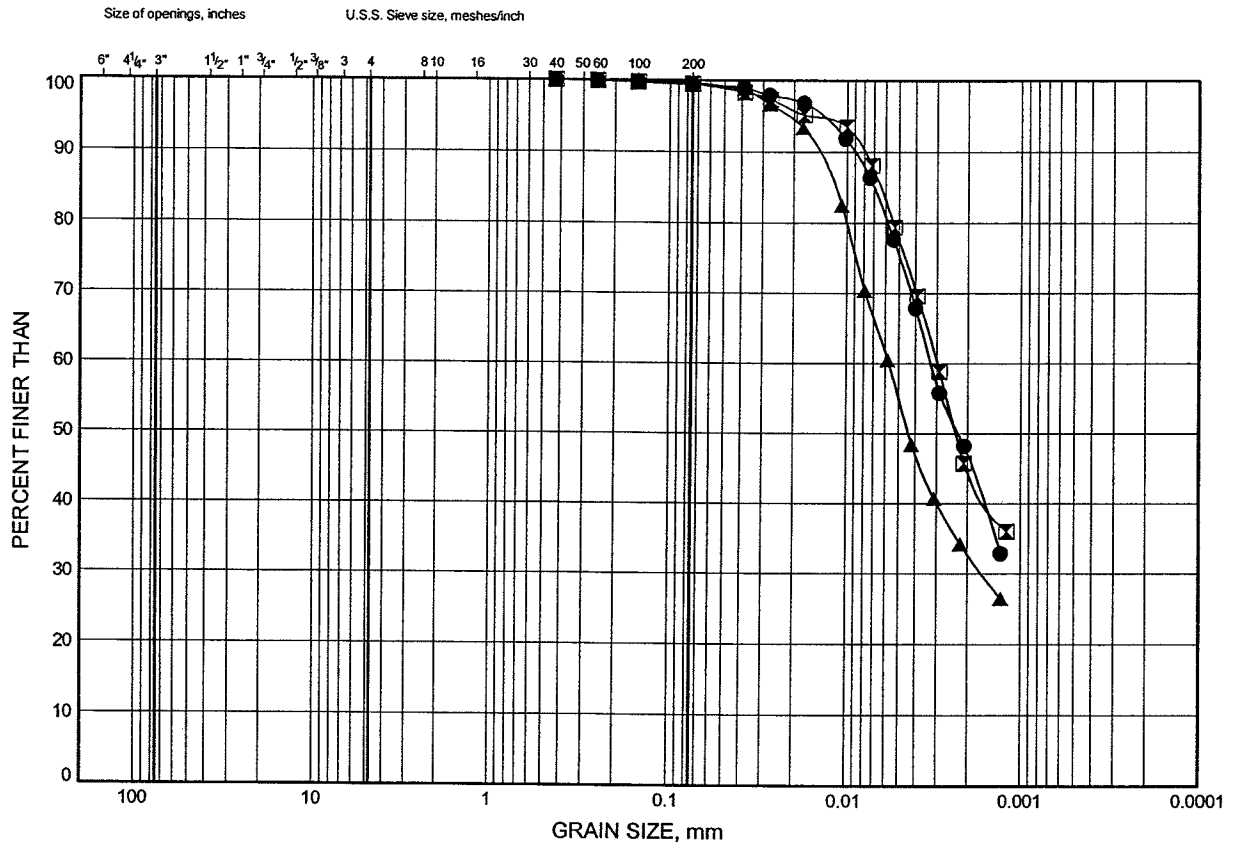


Prep'd SS
Chkd. AEG

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B4

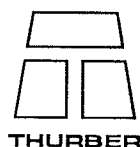
SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-3	10.82	286.98
⊠	122S-4	12.34	285.95
▲	122S-7	10.82	288.18

Date December 2004
Project 361-00-01

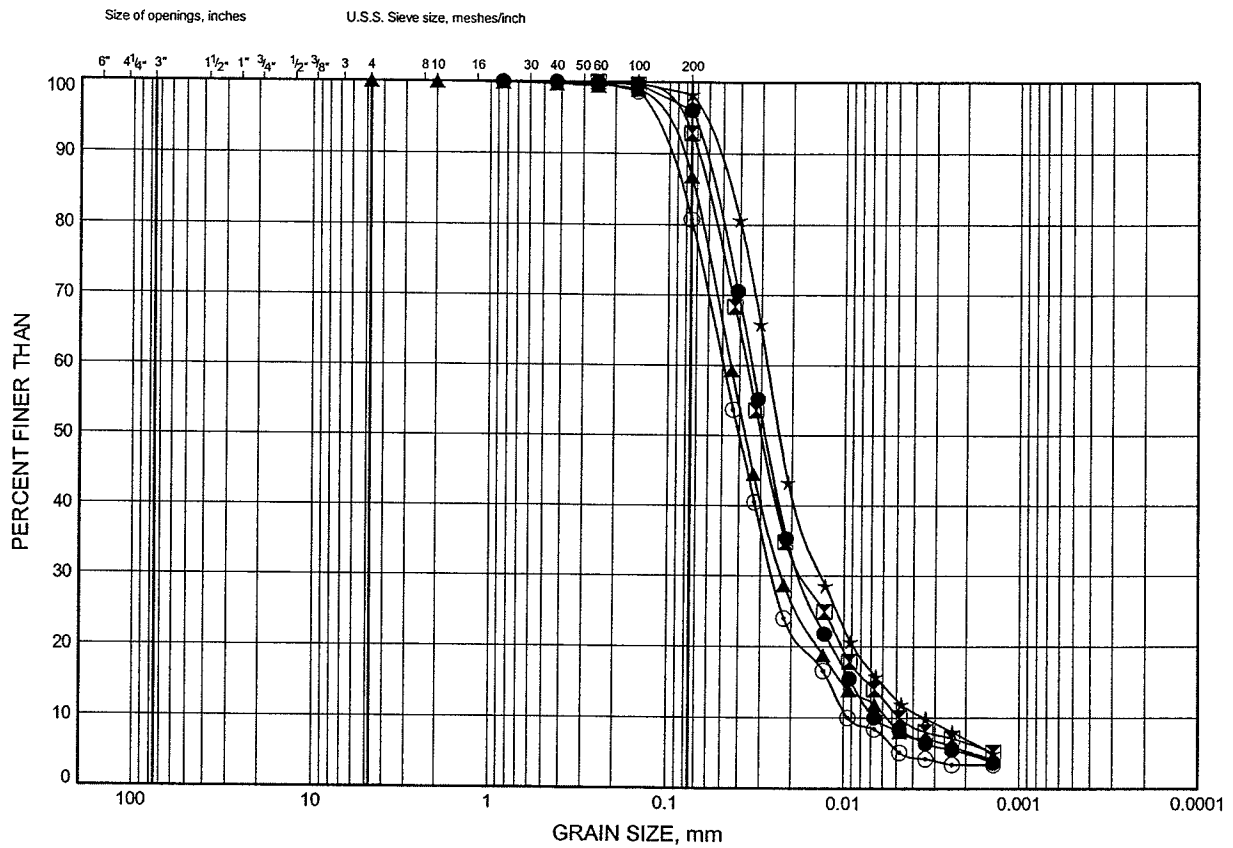


Prep'd SS
Chkd. AEG

Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B5

SILT

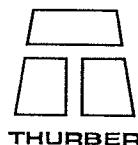


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-10	7.85	291.85
⊠	122S-3	26.06	271.74
▲	122S-4	23.01	275.29
★	122S-7	15.39	283.61
⊙	122S-8	7.92	291.18

Date December 2004

Project 361-00-01



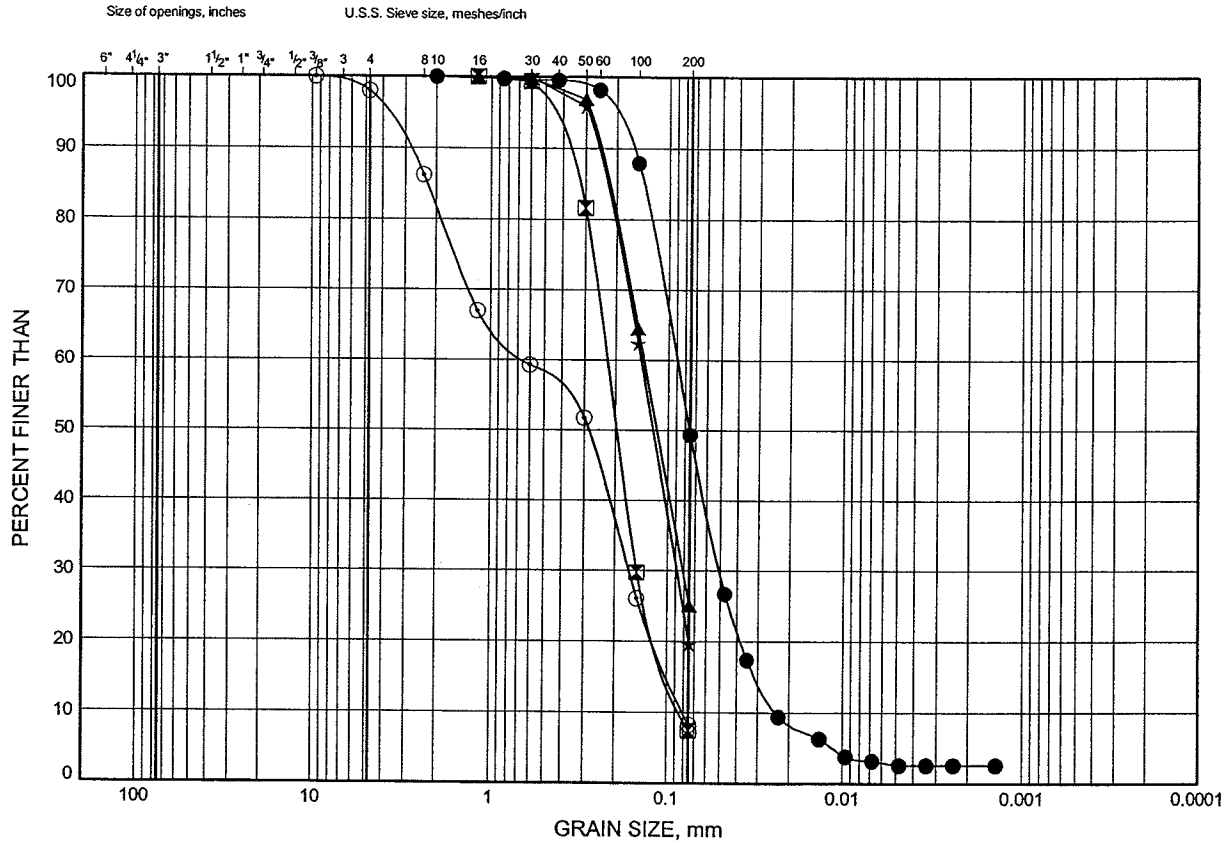
Prep'd SS

Chkd. AEG

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B6

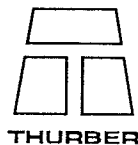
SAND TO SAND AND SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-10	9.37	290.33
⊠	122S-7	23.01	275.99
▲	122S-8	14.02	285.08
★	122S-8	17.07	282.03
⊙	122S-8	23.16	275.94

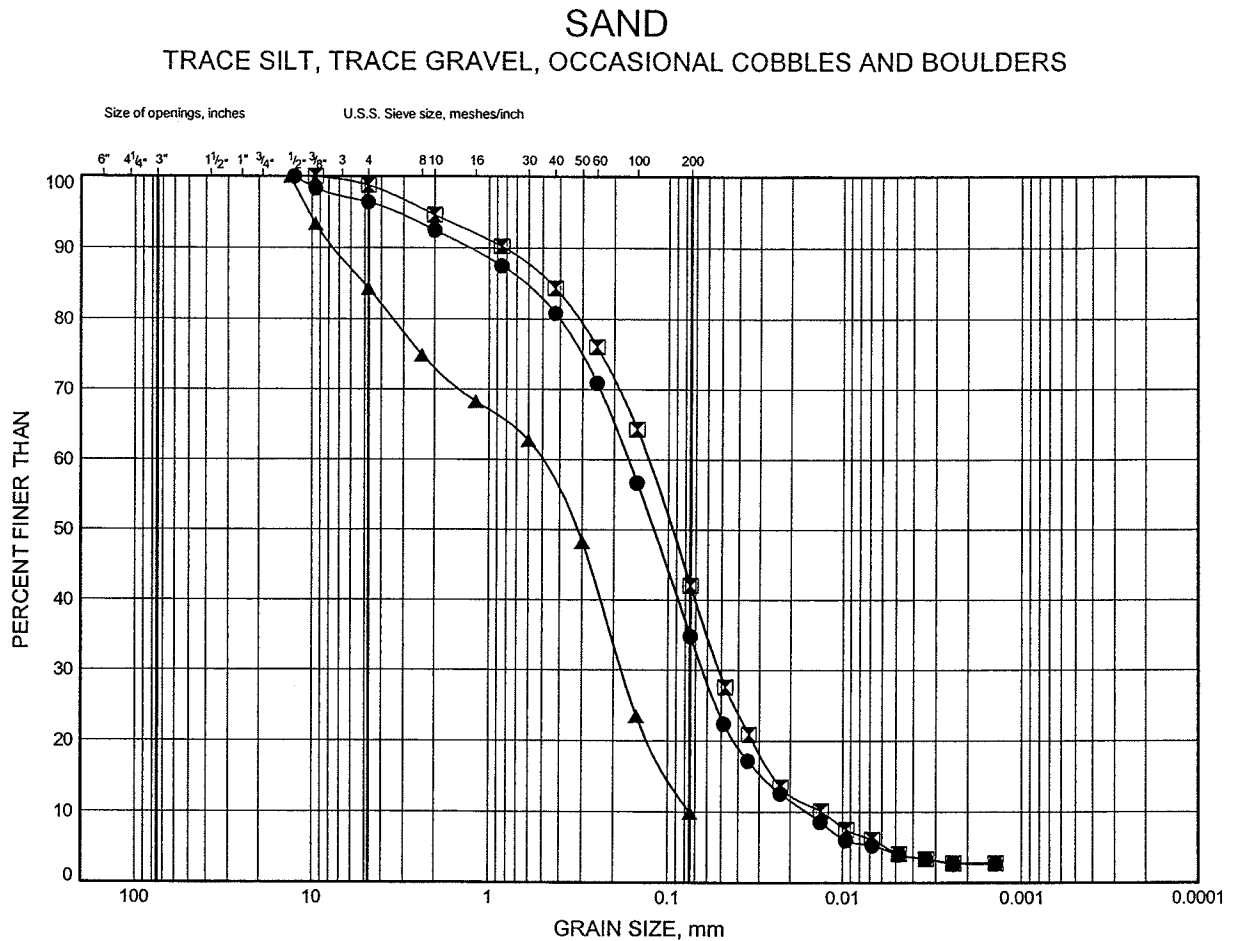
Date December 2004
Project 361-00-01



Prep'd SS
Chkd. AEG

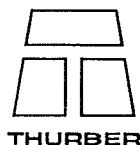
Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B7



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-3	34.98	262.82
⊠	122S-4	33.47	264.83
▲	122S-8	30.78	268.32

Date December 2004
Project 361-00-01

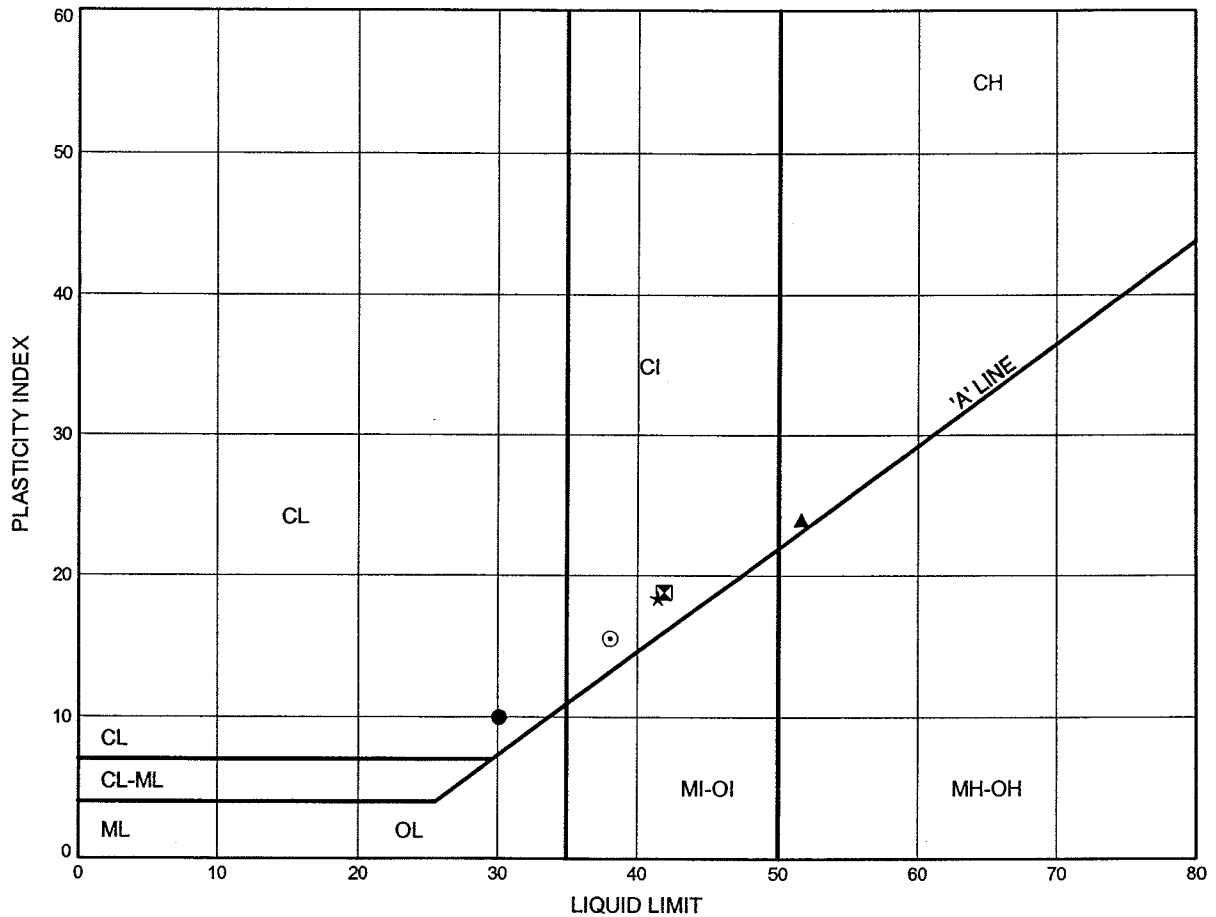


Prep'd SS
Chkd. AEG

Hwy 11 Katrine
ATTERBERG LIMITS TEST RESULTS

FIGURE B8

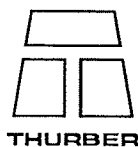
SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-10	3.35	296.35
⊠	122S-3	10.82	286.98
▲	122S-3	15.39	282.41
★	122S-4	12.34	285.95
⊙	122S-4	16.92	281.38

Date December 2004

Project 361-00-01



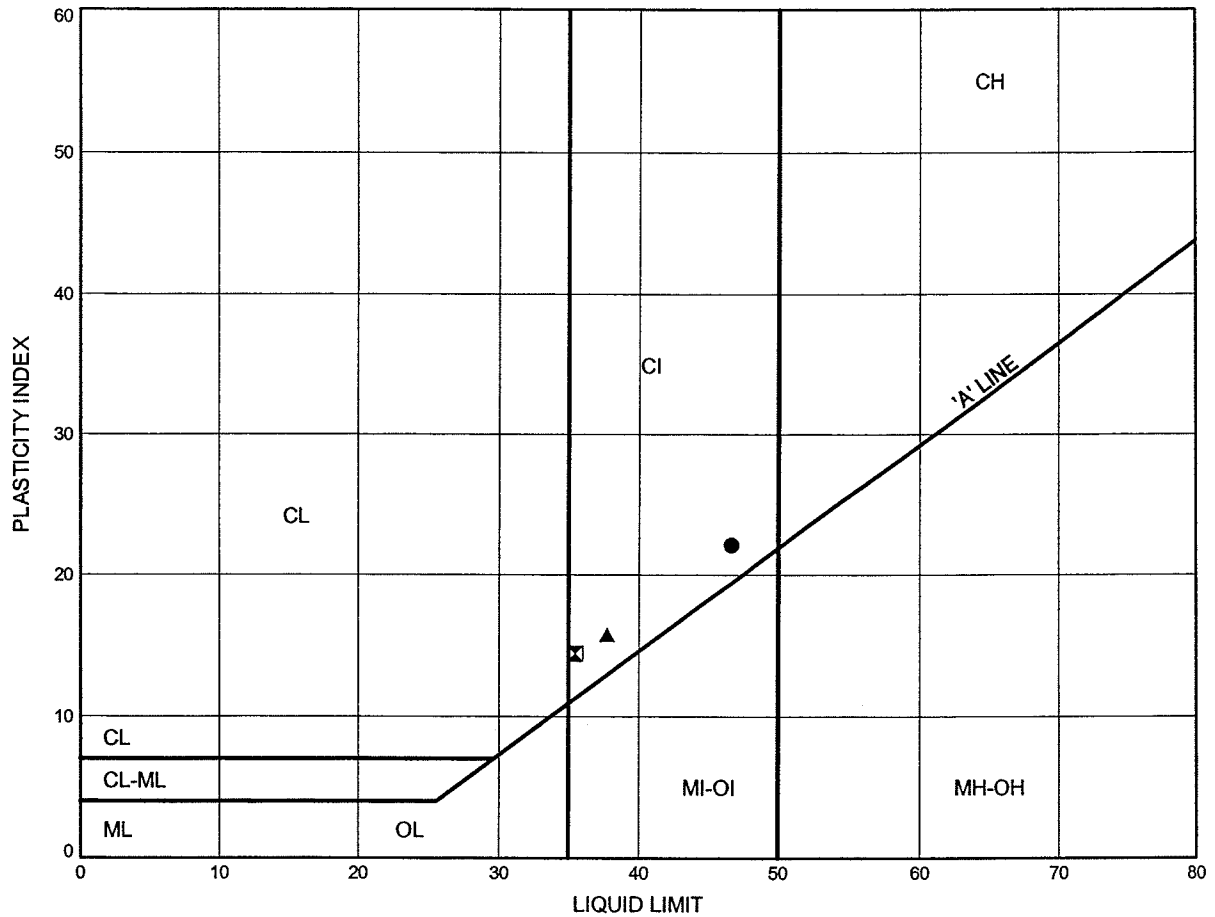
Prep'd SS

Chkd. AEG

Hwy 11 Katrine ATTERBERG LIMITS TEST RESULTS

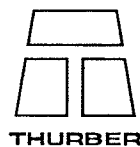
FIGURE B9

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122S-7	7.77	291.23
⊠	122S-7	10.82	288.18
▲	122S-8	4.72	294.38

Date December 2004
Project 361-00-01



Prep'd SS
Chkd. AEG

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1116-123	Sample Number	122S-3
Borehole Number	-	Sample Depth, m	12.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	12/21/2004		
Date Completed	01/05/2005		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	18.14
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	13.12
Area, cm ²	31.67	Specific Gravity, measured	2.74
Volume, cm ³	60.17	Solids Height, cm	0.928
Water Content, %	38.26	Volume of Solids, cm ³	29.38
Wet Mass, g	111.31	Volume of Voids, cm ³	30.79
Dry Mass, g	80.51	Degree of Saturation, %	100.0

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	1.048	1.900				
4.75	1.895	1.042	1.898	141	5.41E-03	5.54E-04	2.94E-07
9.54	1.889	1.036	1.892	98	7.74E-03	6.59E-04	5.00E-07
19.25	1.878	1.024	1.884	124	6.07E-03	5.96E-04	3.54E-07
38.68	1.861	1.006	1.870	89	8.33E-03	4.60E-04	3.76E-07
77.38	1.839	0.982	1.850	76	9.55E-03	2.99E-04	2.80E-07
154.57	1.805	0.945	1.822	60	1.17E-02	2.32E-04	2.66E-07
309.39	1.757	0.894	1.781	60	1.12E-02	1.63E-04	1.79E-07
619.29	1.695	0.827	1.726	53	1.19E-02	1.05E-04	1.23E-07
1238.59	1.631	0.758	1.663	53	1.11E-02	5.44E-05	5.90E-08
2475.62	1.560	0.681	1.596	53	1.02E-02	3.02E-05	3.01E-08
1238.59	1.571	0.693	1.566				
309.39	1.592	0.716	1.582				
77.38	1.621	0.747	1.607				
19.25	1.658	0.787	1.640				
4.75	1.684	0.815	1.671				

Notes:

k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.68	Unit Weight, kN/m ³	19.48
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.80
Area, cm ²	31.67	Specific Gravity, measured	2.74
Volume, cm ³	53.33	Solids Height, cm	0.928
Water Content, %	31.59	Volume of Solids, cm ³	29.38
Wet Mass, g	105.94	Volume of Voids, cm ³	23.95
Dry Mass, g	80.51		

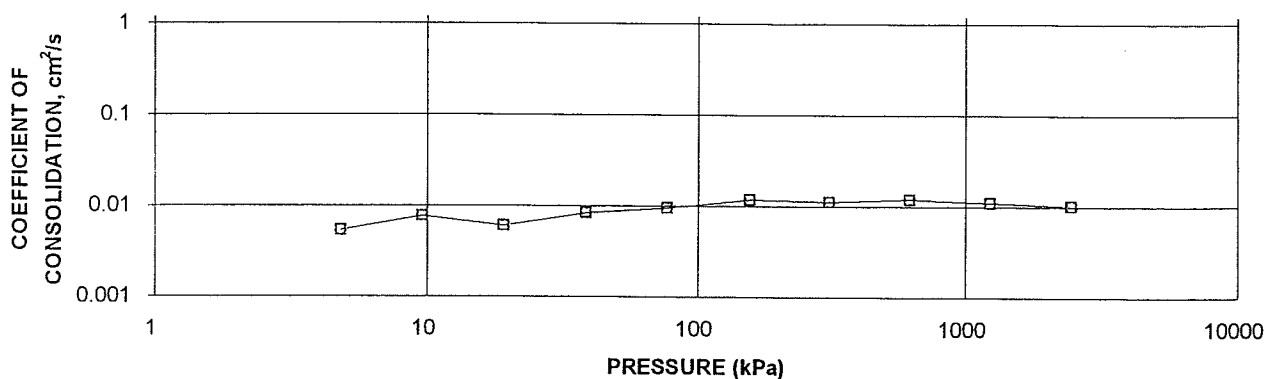
Prepared By: LFG

Golder Associates

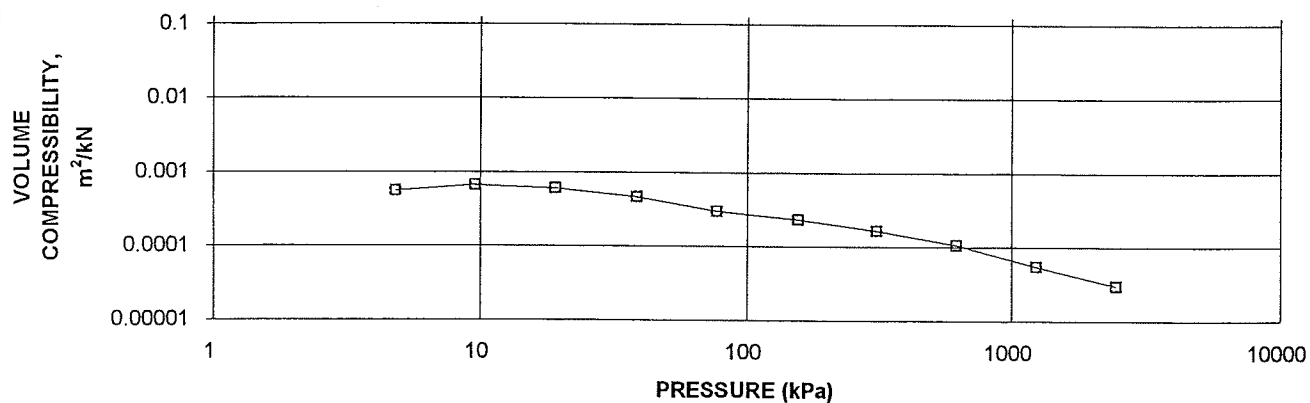
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OEDOMETER CONSOLIDATION SUMMARY

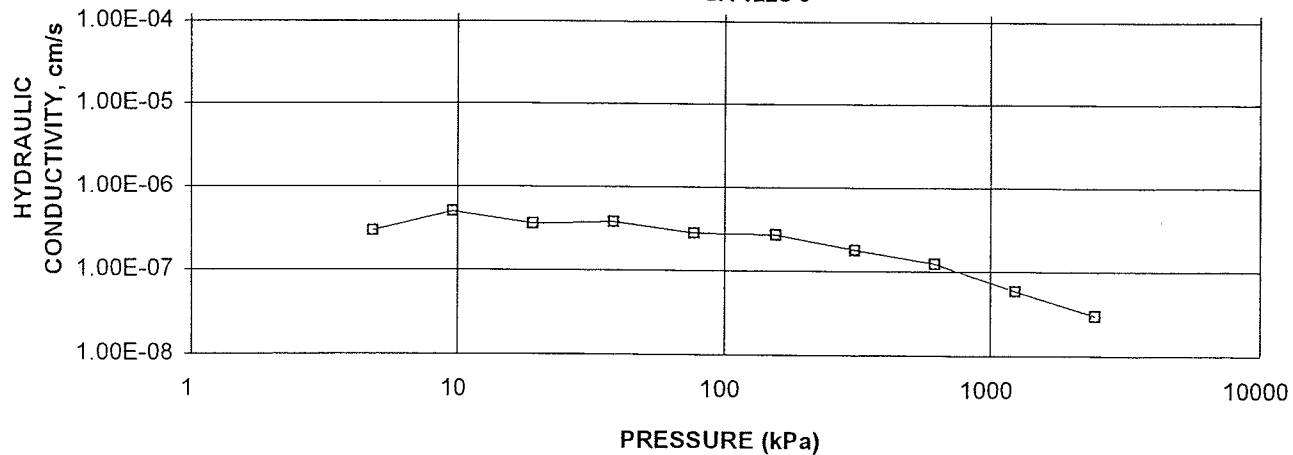
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
SA 122S-3



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
SA 122S-3



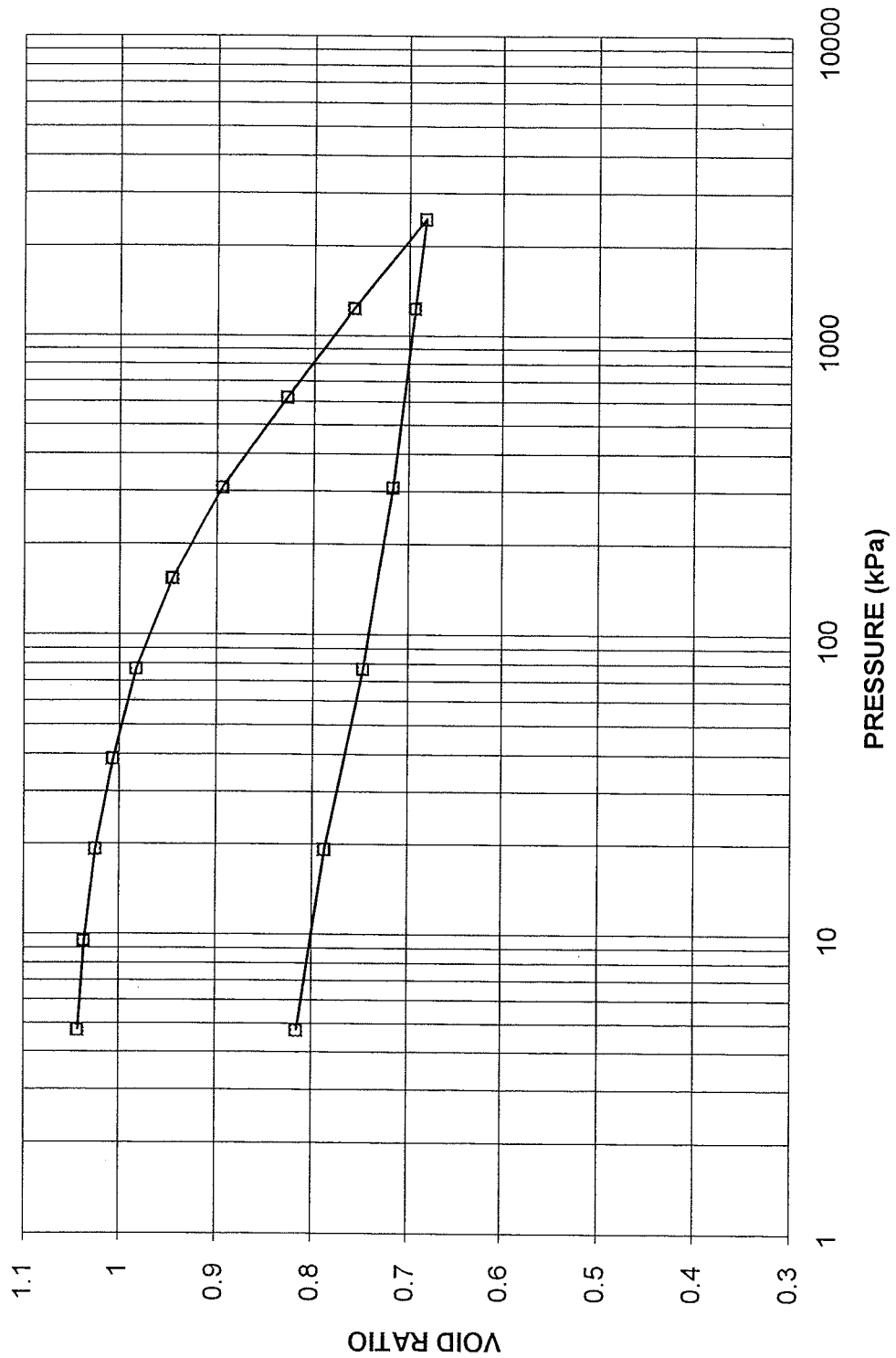
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
SA 122S-3



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
SA 122S-3



SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	04-1116-123	
PROJECT NAME	Thurber / Lab Testing / 19-1423-16	
DATE TESTED	December, 2004	
Borehole	Sample	Specific
No.	No.	Gravity
-	122N-8	2.75
-	122S-3	2.74

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

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. <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"> iii. Cost of constructing engineered fill. iv. Low geotechnical resistance available at this site. v. Potential for unacceptable magnitude of settlement. <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. Soil conditions encountered at this site are considered to be unsuitable. <p>NOT RECOMMENDED</p>

DRAFT

Highway 11 Southbound Lanes Over Magnetawan River South Crossing

Appendix D

Special Provisions

Highway 11 Southbound Lanes Over Magnetawan River South Crossing

The following Special provisions are referenced in this report:

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

Suggested text for a NSSP on Pile Installation should contain the following:

“The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 275. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- *The need to provide protection to the pile tips in the form of rock points*
- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving to reach bedrock*
- *Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving*
- *As a result of the presence of boulders, piles may meet refusal at varying depths*
- *Pile driving must be controlled according to the criteria specified for the site.*

Appendix E

Selected Slope Stability Output

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL South Approach
 Rock Fill as on x-section but 1.25:1

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
New Fill	20	0	0	1
Earth Fill	22	0	0	1
Silt	20	0	0	1
Clay	19	0	0	1
Clayey Silt	19	0	0	1
Silt/Sand	21	0	0	1

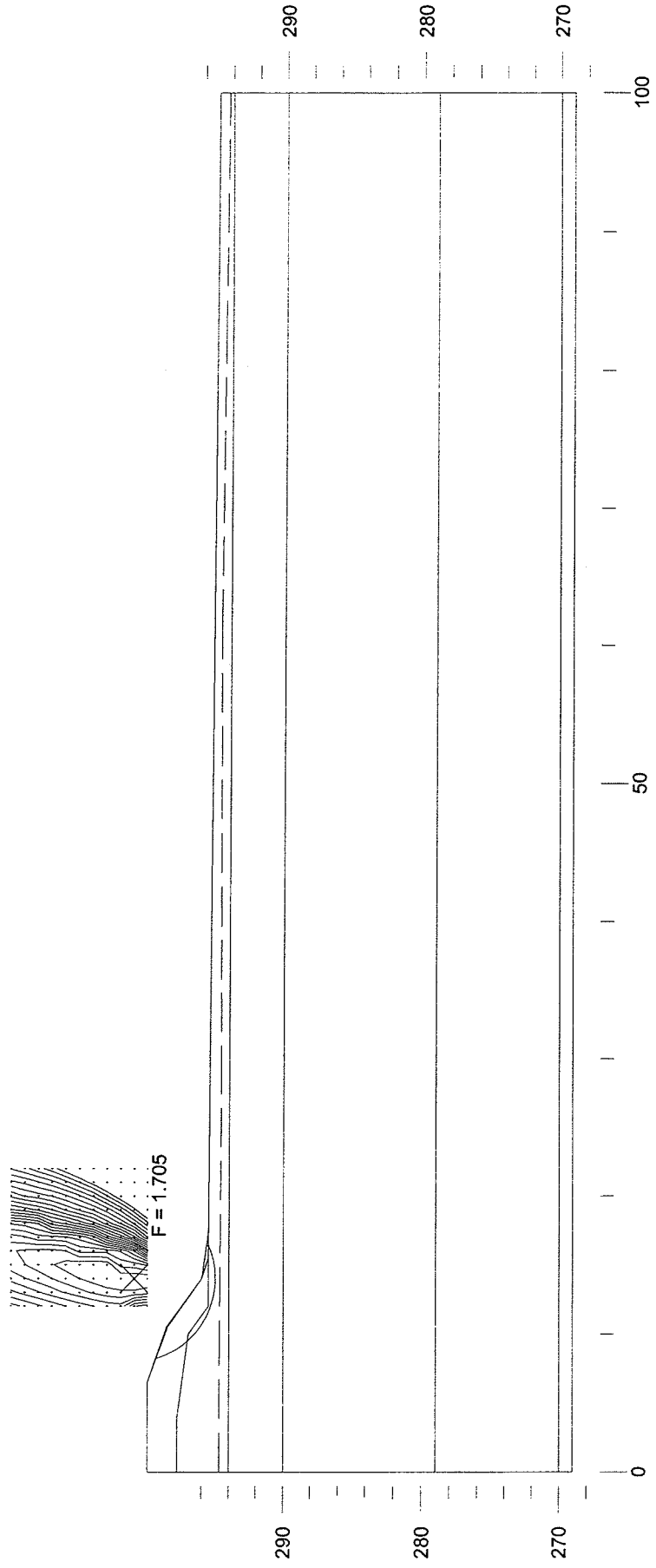


Figure E1

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL South Approach
 Rock Fill as on x-section but 1.25:1, 100 year, 0.08 Seismic

	Gamma C kN/m ³	Phi deg	Min c/p	Piezo Surf.
Water	10	0	0	1
New Fill	20	0	0	1
Earth Fill	22	0	0	1
Silt	20	0	0	1
Clay	19	0	0	1
Clayey Silt	19	0	0	1
Silt/Sand	21	0	0	1

Seismic coefficient = 0.08

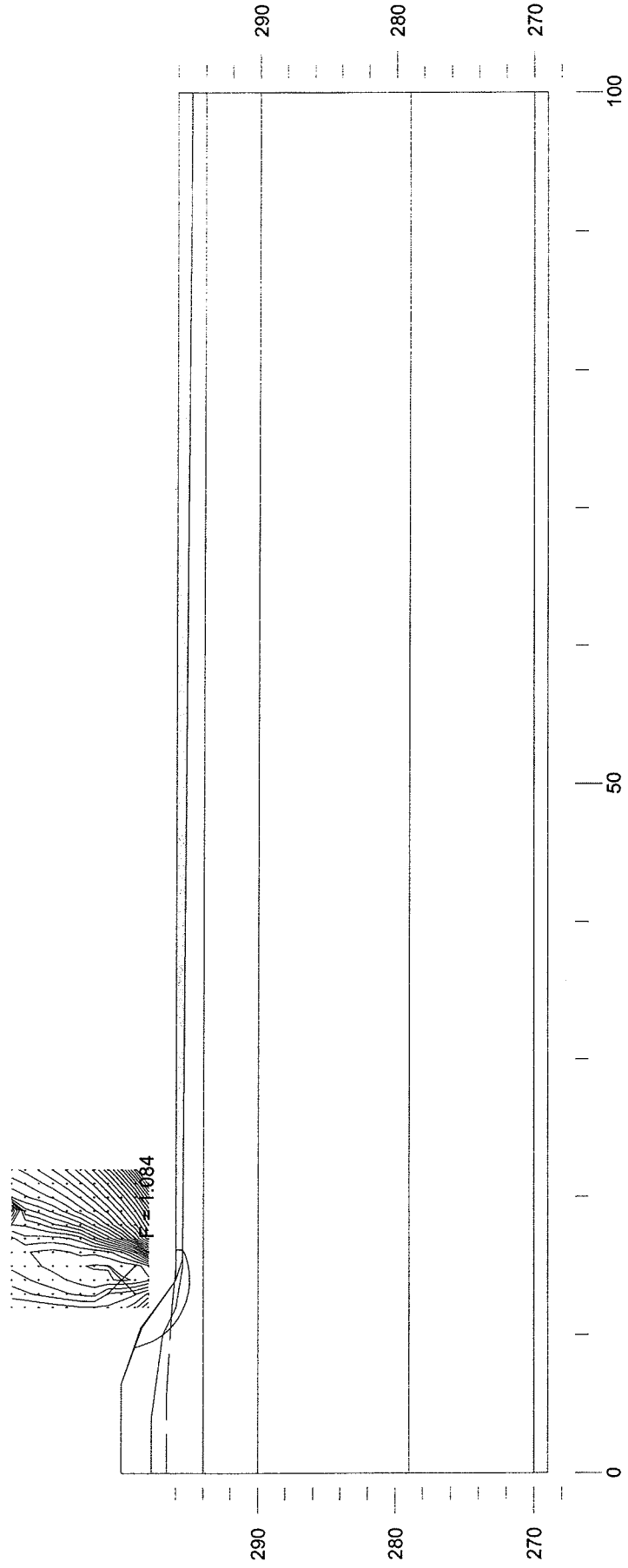


Figure E2

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL South Approach
 Earth Fill 2:1, as on xsection

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
New Fill	22	0	0	1
Earth Fill	22	0	0	1
Silt	20	0	0	1
Clay	19	0	0	1
Clayey Silt	19	0	0	1
Silt/Sand	21	0	0	1

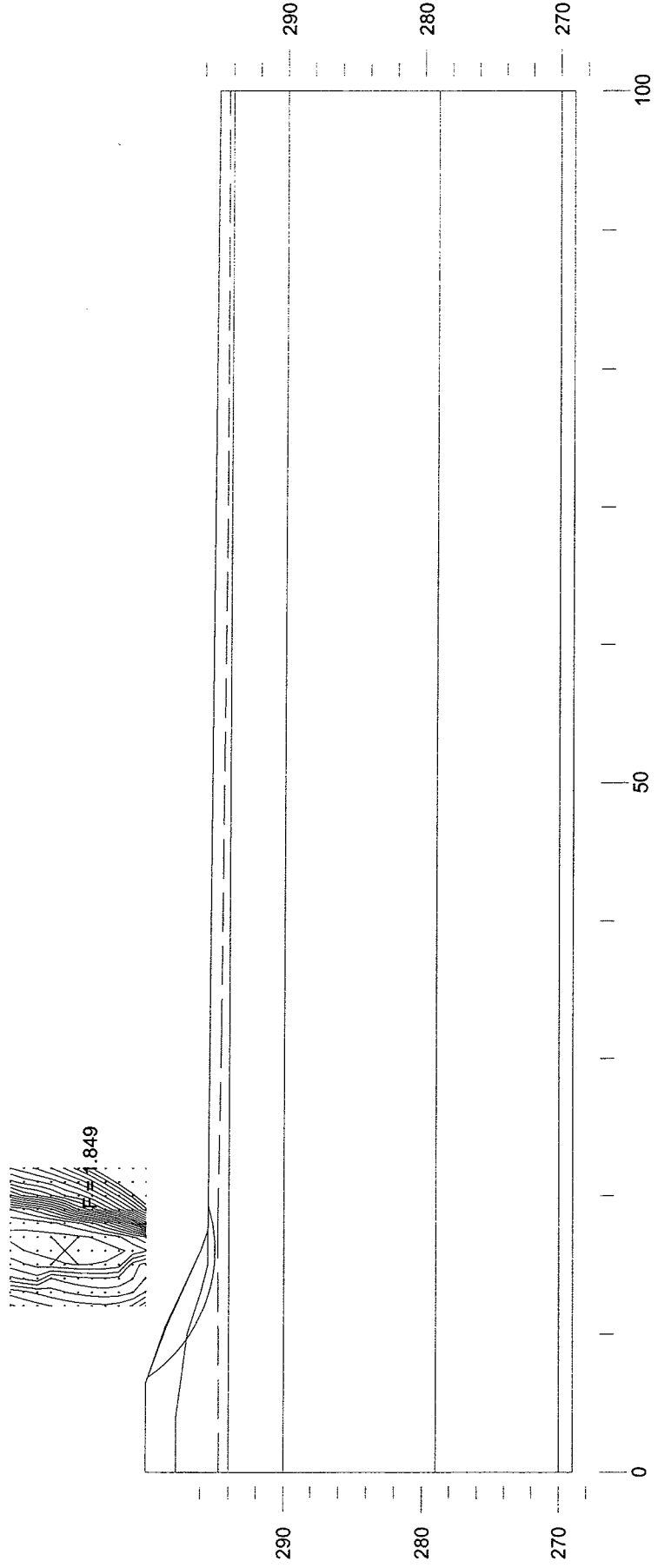


Figure E3

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 December 20, 2004
 Magnetawan South SBL South Approach
 Earth Fill 2:1. as on xsection, 100 year, 0.08 Seismic

	Gamma C kN/m ³	Phi deg	Piezo Surf.
Water	10	0	1
New Fill	22	33	1
Earth Fill	22	33	1
Silt	20	30	1
Clay	19	26	1
Clayey Silt	19	28	1
Silt/Sand	21	30	1

Seismic coefficient = 0.08

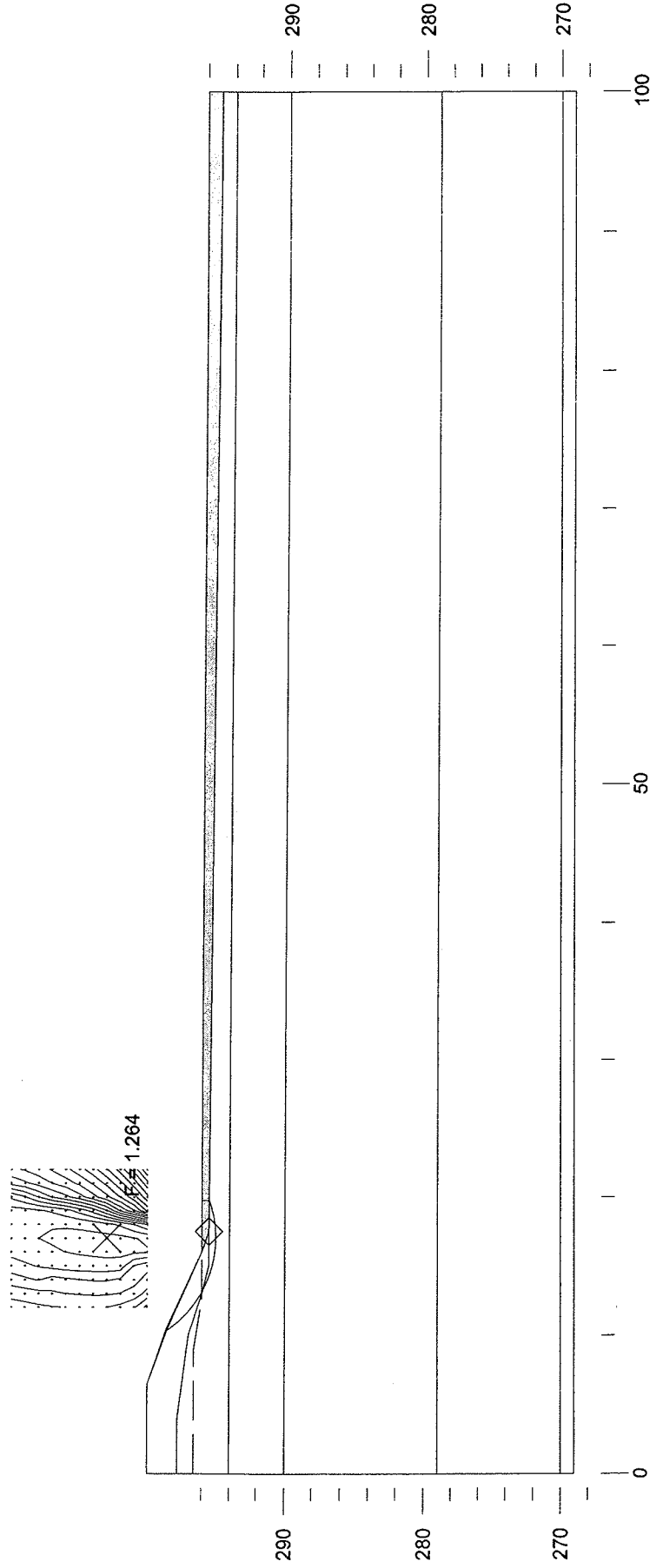


Figure E4

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 December 20, 2004
 Magnetawan South SBL North Approach
 Rock Fill 1.25:1, Normal W.L., No Seismic

	Gamma C kN/m3	Phi deg	Piezo Surf.
Water	10	0	1
Rock Fill	20	42	1
Existing Fill	22	33	1
Clay	19	26	1
Clayey Silt	19	28	1
Silt/Sand	21	30	1

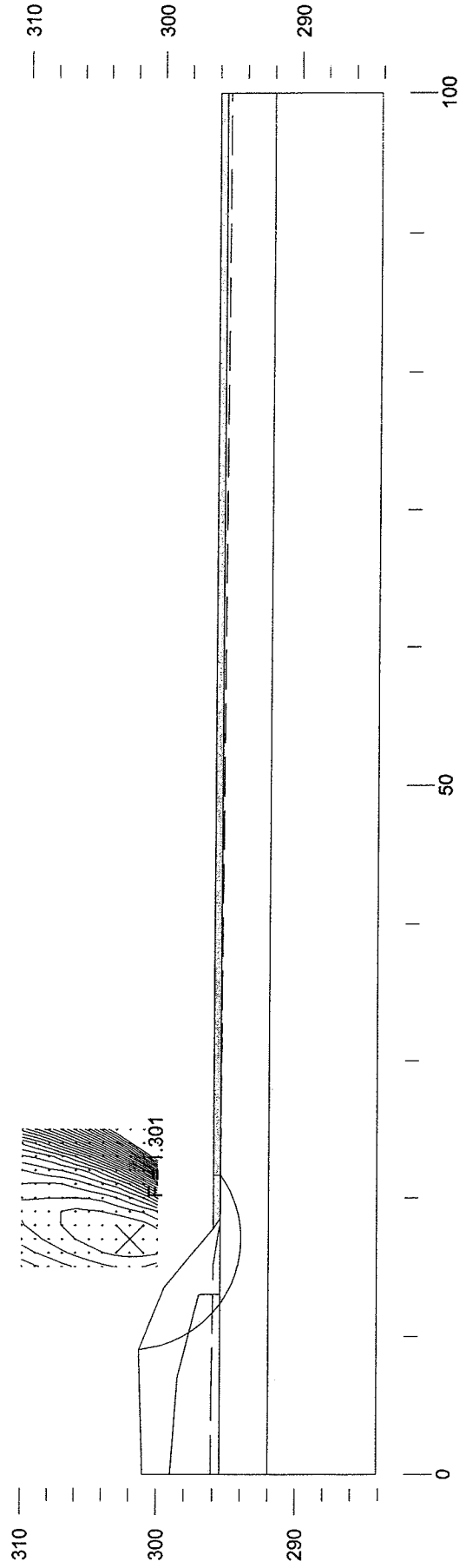


Figure E5

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 December 20, 2004
 Magnetawan South SBL North Approach
 Rock Fill 1.25:1, 100 year, 0.08 Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Rock Fill	20	42	1
Existing Fill	22	33	1
Clay	19	26	1
Clayey Silt	19	28	1
Silt/Sand	21	30	1

Seismic coefficient = 0.08

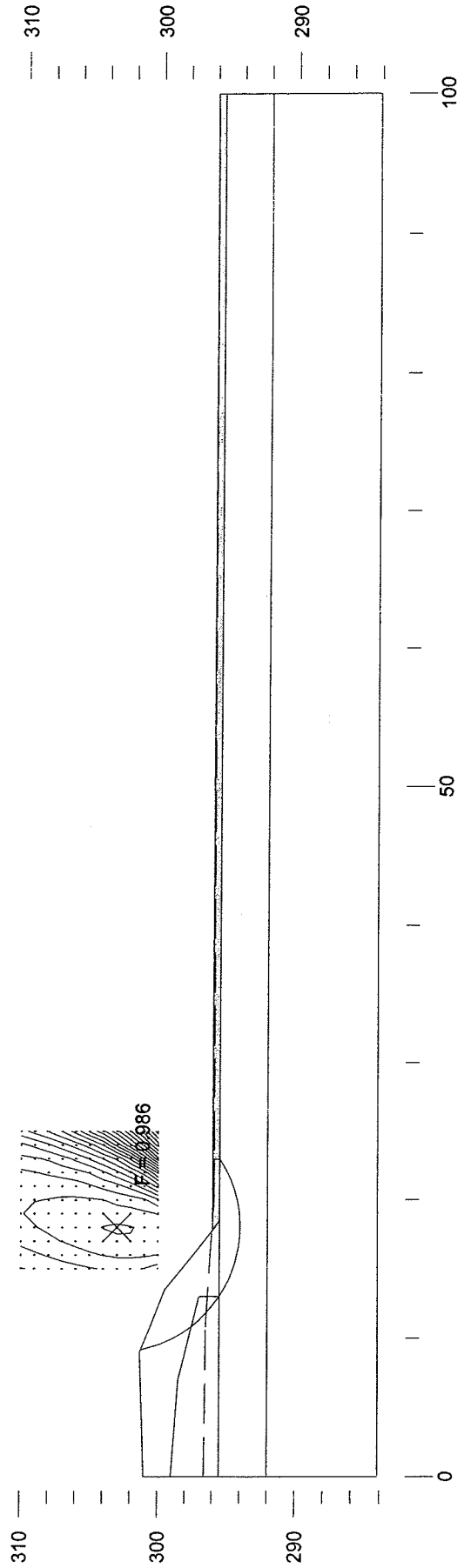


Figure E6

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 December 20, 2004
 Magnetawan South SBL North Approach
 Earth Fill 2:1, as on x-section

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Earth Fill	22	33	1
Existing Fill	22	33	1
Clay	19	26	1
Clayey Silt	19	28	1
Silt/Sand	21	30	1

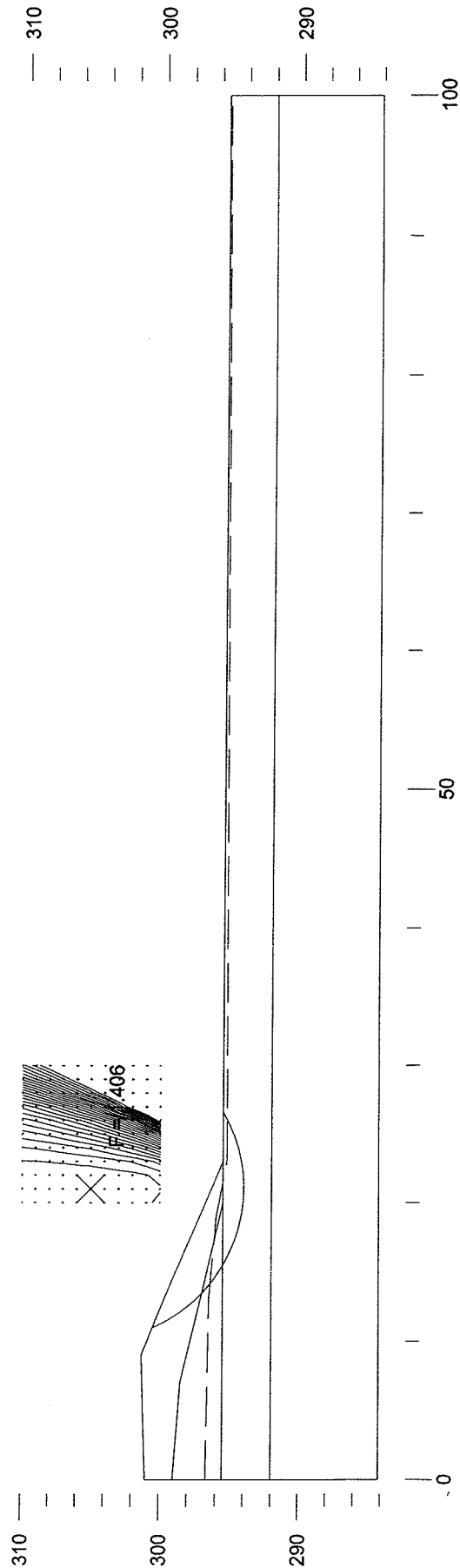


Figure E7

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 December 20, 2004
 Magnetawan South SBL North Approach
 Earth Fill 2:1, as on x-section, 100 year, 0.08 Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Earth Fill	22	33	1
Existing Fill	22	33	1
Clay	19	26	1
Clayey Silt	19	28	1
Silt/Sand	21	30	1

Seismic coefficient = 0.08

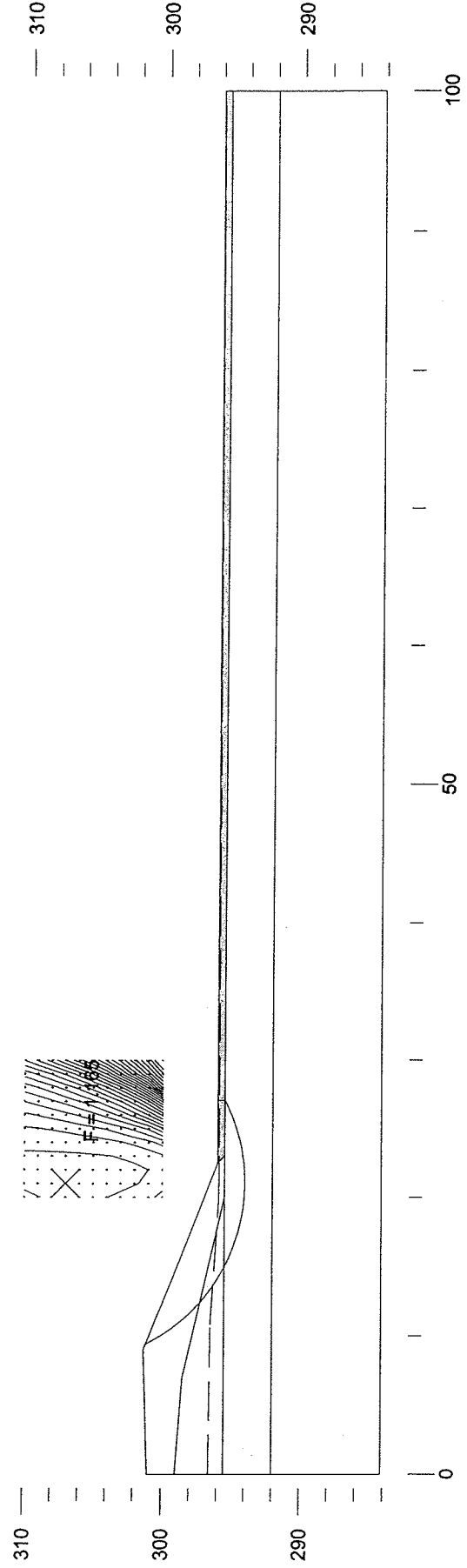


Figure E8

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL South Approach
 Rock Fill - construction

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
New Fill	20	0	0	1
Earth Fill	22	0	0	2
Silt	20	0	0	3
Clay	19	0	0	4
Clayey Silt	19	0	0	5
Silt/Sand	21	0	0	6

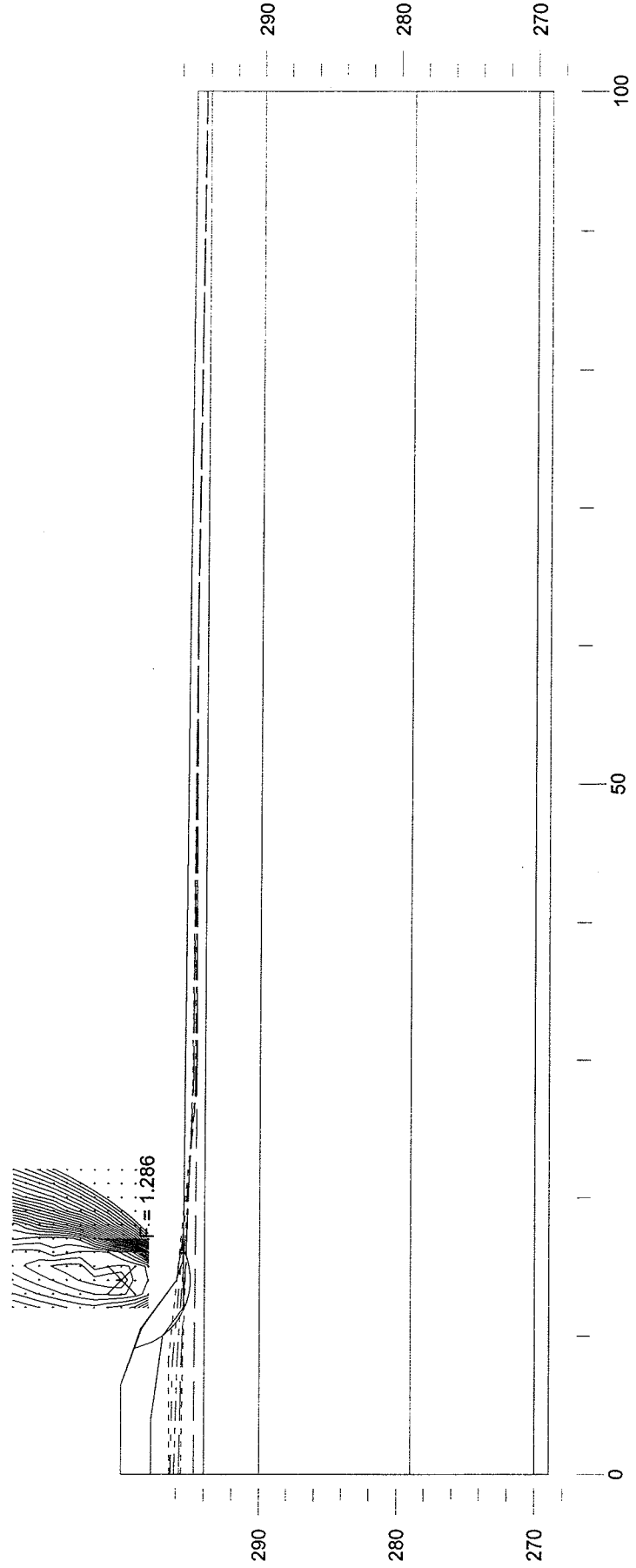


Figure E9

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL South Approach
 Earth Fill 2:1. construction

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
New Fill	22	0	0	1
Earth Fill	22	0	0	2
Silt	20	0	0	3
Clay	19	50	0	4
Clayey Silt	19	0	0	5
Silt/Sand	21	0	0	6

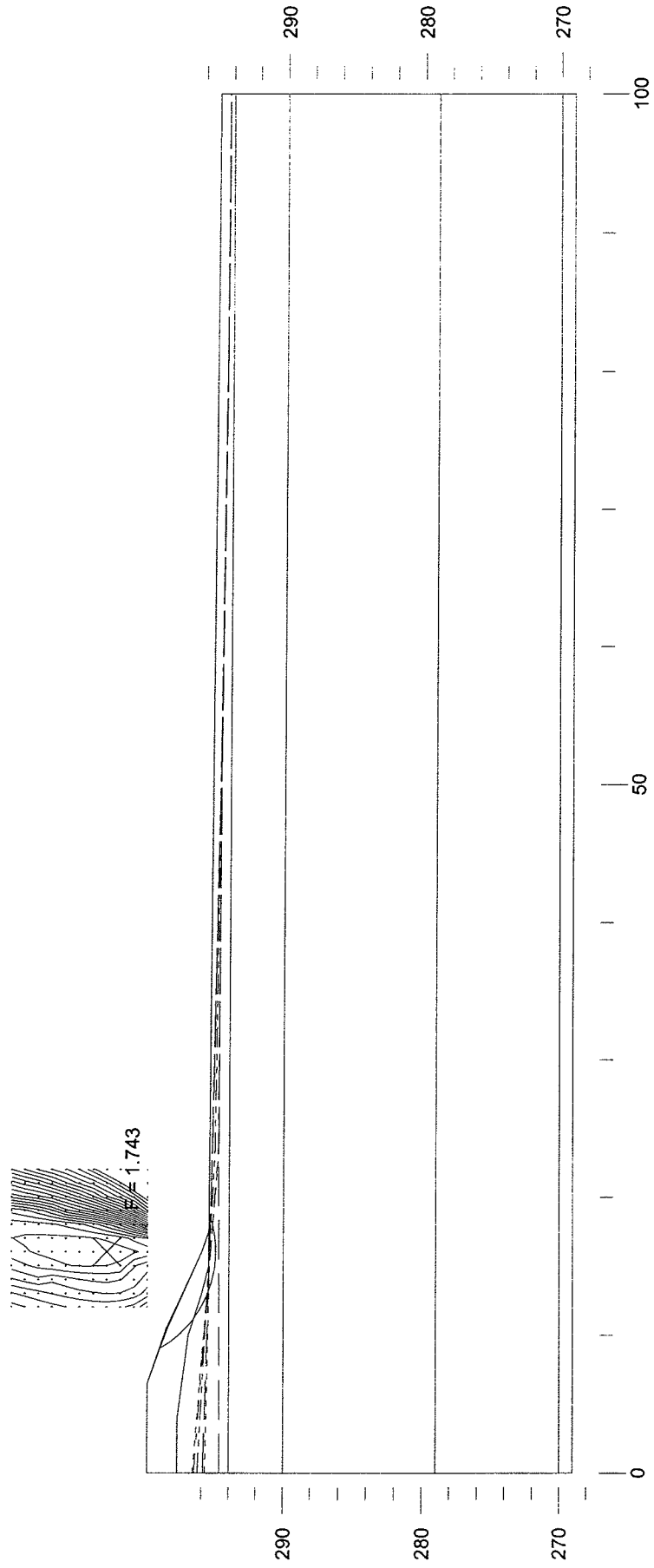


Figure E10

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11, Katrine
 September 13, 2005
 Magnetawan South SBL North Approach
 Rock Fill 1.25:1 - construction

	Gamma C kN/m3	Phi deg	Min c/p	Piezo Surf.
Water	10	0	0	1
Rock Fill	20	0	0	1
Existing Fill	22	0	0	2
Clay	19	50	0	3
Clayey Silt	19	0	0	4
Silt/Sand	21	0	30	5

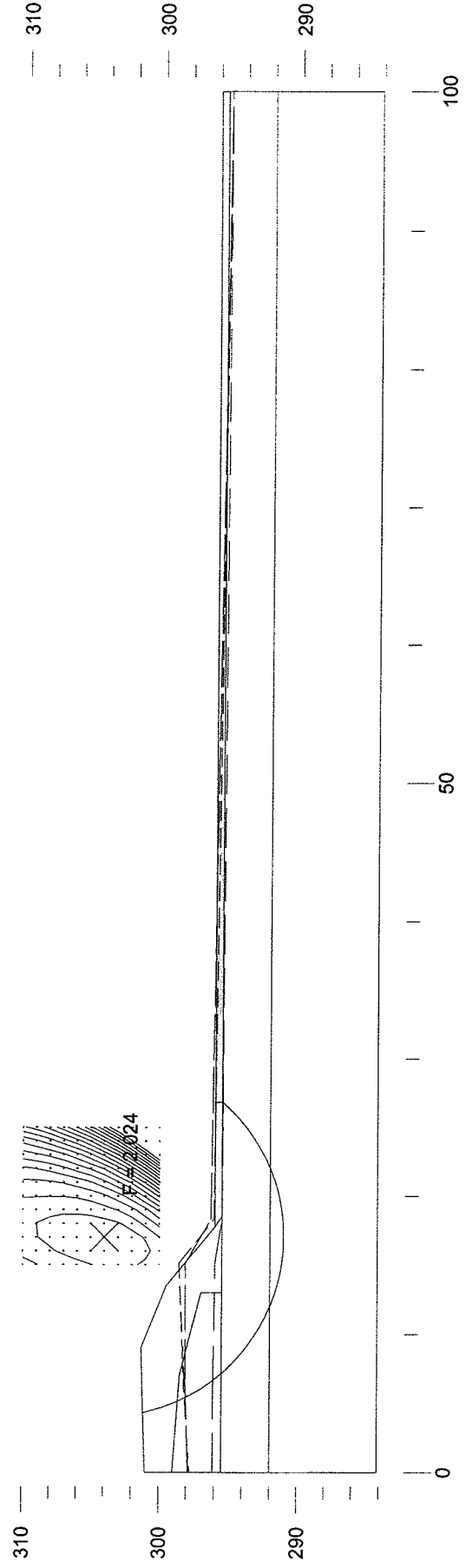


Figure E11

Thurber Engineering Ltd. - Toronto				
19-1423-16				
Hwy 11, Katrine				
September 13, 2005				
Magnetawan South SBL North Approach				
Earth Fill 2:1 - construction				
	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
Earth Fill	22	0	33	1
Existing Fill	22	0	33	2
Clay	19	50	0	3
Clayey Silt	19	0	28	4
Silt/Sand	21	0	30	5

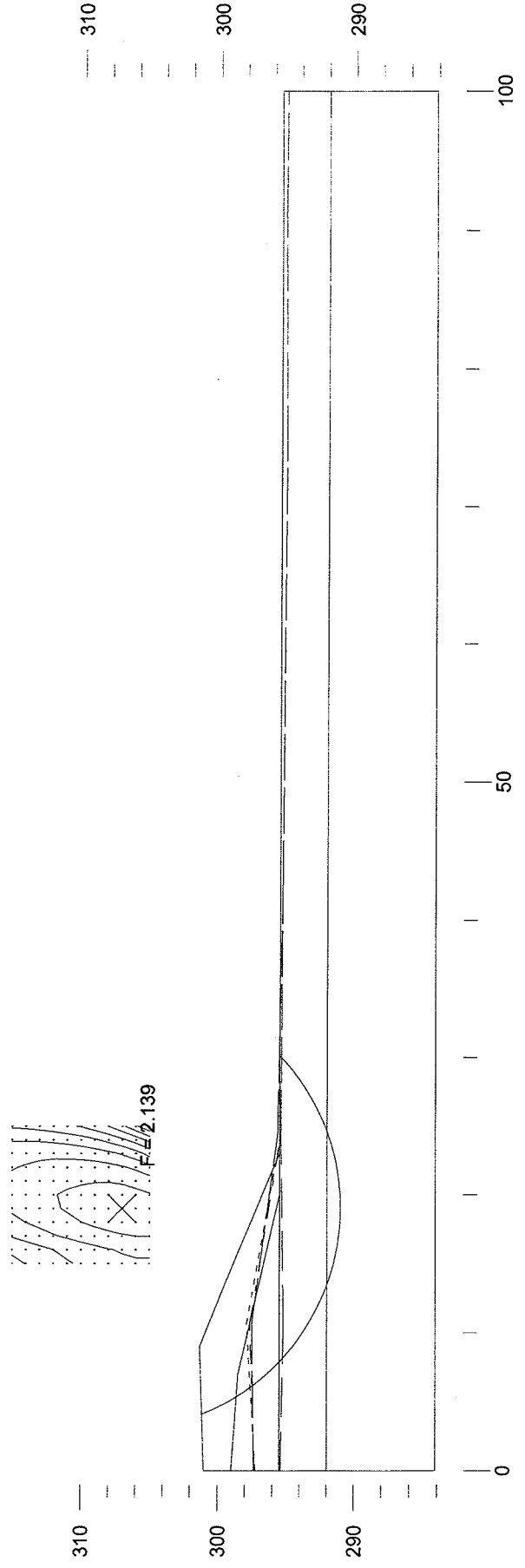


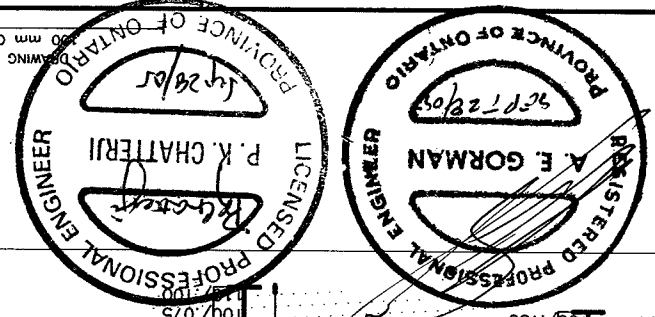
Figure E12

Highway 11 Southbound Lanes Over Magnetawan River South Crossing

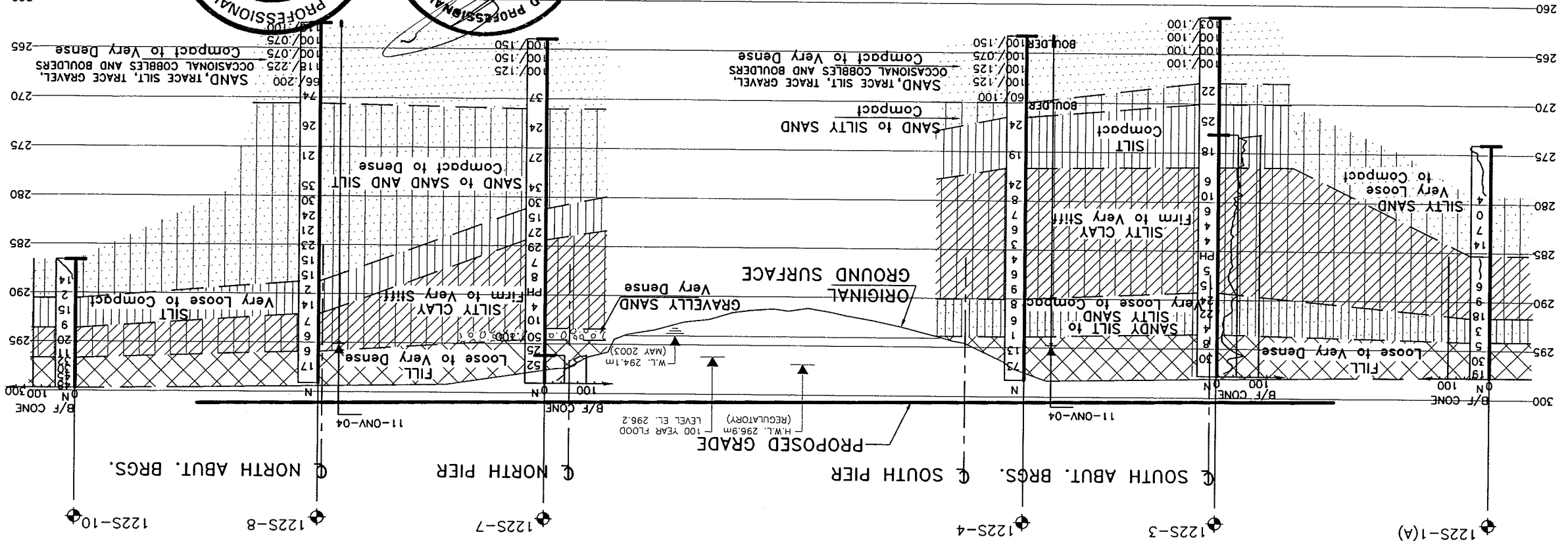
Appendix F

Drawings

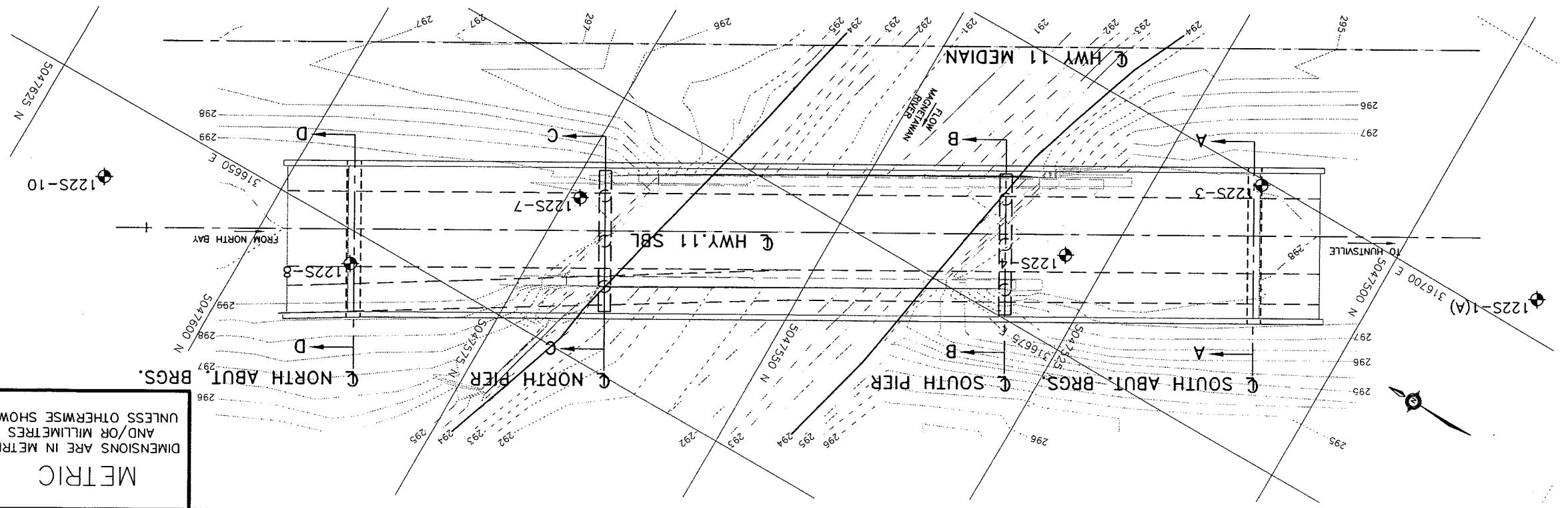
VCP: HCP No. 112
EL. 298.289
19mm x 1.52m IRON BAR
2.3 LT 2.94m N OF HWY 518
24.9 LT STA. 11+172.031



PROFILE @ HWY 11 SBL



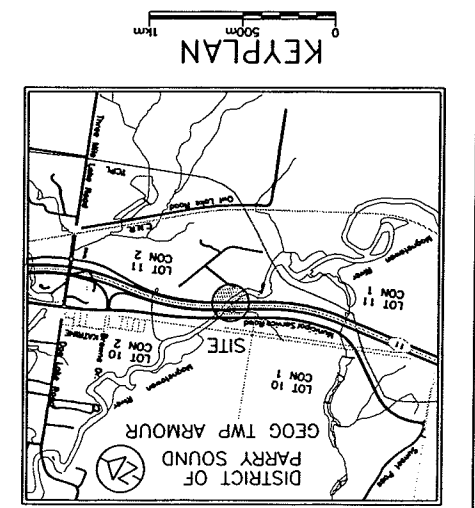
PLAN



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

NO	ELEVATION	NORTHING	EASTING
122S-1	297.8	5047485.6	316703.6
122S-3	297.8	5047515.4	316699.4
122S-4	298.3	5047529.0	316683.5
122S-7	299.0	5047573.8	316664.2
122S-8	299.1	5047590.4	316647.0
122S-10	299.7	5047615.9	316642.3

Dynamic Cone Penetration Test (cone)	N	Blows / 0.5m (Std Pen Test, 475J/blow)	PH	Pressure, Hydraulic	WL	Head Artesian Water	Piezometer	90%	Rock Quality Designation (RQD)
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THURBER ENGINEERING LTD.

PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

Marshall Macklin

BOREHOLE LOCATIONS AND SOIL STRATA

MAGNETAWAN RIVER
SOUTH CROSSING SBL

HPW 11
CONT No 2006-5148
WP No 361-00-01

SHEET

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

