

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 11 NORTHBOUND LANES OVER  
MAGNETAWAN RIVER SOUTH CROSSING  
HIGHWAY 11, HIGHWAY 518 WEST to HIGHWAY 520  
G.W.P. 480-93-00, W.P. 474-93-01, SITE 44-122N**

**Geocres Number: 31E-226**

**Report to**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed bridge to carry the Northbound Lanes of the widened Highway 11 over the Magnetawan River at a point south of the village of Katrine, Ontario. A previous, preliminary investigation had been carried out at the site by Shaheen & Peaker Limited (S&P) and the factual data from that investigation has been incorporated in the current assignment.

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 through considering a combination of the data from the previous S&P investigation and 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.

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 across the Magnetawan River at a location where the proposed northbound lanes of Highway 11 will cross the river south of the Village of Katrine, Armour Township. The site lies approximately 2.8 km north of Highway 518 West and 1.0 km south of Three Mile lake Road. The centreline of the northbound lanes will be approximately 40 m east of the existing Highway 11 centreline.

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 27 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 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 13 and November 1, 2004. Boreholes were also drilled at the site between March 27 and April 10, 2001, as part of the preliminary investigation by Shaheen & Peaker Limited.

The current site investigation consisted of drilling and sampling a total of six boreholes to depths of approximately 11 m at the approach fills and to depths of 36 to 42 m at the foundation locations. The four boreholes drilled at the foundation locations were supplemented by dynamic cone penetration tests. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

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). Where significant proportions of cobbles and boulders were encountered and soil boring and SPT sampling were not feasible, diamond coring techniques were employed to penetrate the soils.

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	122N-10, M4*
North Abutment	122N-8, M2*, M4*
North Pier	122N-7, M2*
South Pier	122N-4, M1*
South Abutment	122N-3, M1*, M3*
South Approach	122N-1, M3*

\* Boreholes drilled by S&P in 2001

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 each of the four deep boreholes drilled at the foundation elements to monitor the groundwater level. Piezometers were also installed in two deep boreholes drilled in the course of the preliminary investigation.

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 122N-3	41.8/254.1	Piezometer with 1.5 m tip installed at 41.8. Sand filter to 39.6, bentonite seal to 38.4, grout to 1.5 and bentonite seal to the surface.
BH 122N-4	39.6/255.2	Piezometer with 1.5 m tip installed at 39.6. Sand filter to 37.4, bentonite seal to 36.6, grout to 1.5 and bentonite seal to the surface.
BH 122N-7	36.6/260.0	Piezometer with 1.5 m tip installed at 36.6. Sand filter to 34.7, bentonite seal to 33.8, grout to 1.5 and bentonite seal to the surface.
BH 122N-8	32.0/267.0	Piezometer with 1.5 m tip installed at 32.0. Sand filter and sand cave to 22.9, bentonite seal to 22.2, grout 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. A total of five samples were selected for this testing.

One Shelby tube sample was selected from the area of the north abutment of the NBL structure and was subjected to one-dimensional consolidation testing at the Laboratory of Golder Associates. The results of the consolidation test are included 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. This latter material typically mantles the bedrock but 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, a layer of sandy silt, silty clay; silty sand, sand and gravel with cobbles and boulders.

More detailed descriptions of the individual strata are presented below.

## **5.2 Topsoil and Peat**

Topsoil and peat were encountered at the ground surface in several boreholes. The thickness ranged from nil at some boreholes to a maximum of 300 mm. Further variations in thickness may occur between or beyond the boreholes.

## **5.3 Sandy Silt**

A layer of fine-grained non-cohesive soils ranging from sandy silt to silty sand was encountered at the ground surface, or below the topsoil and peat. Interbedded layers of fine sand were also encountered.

Based on SPT values ranging from 1 to 27 blows for 0.3 m of penetration, the deposit is classified as very loose to compact. The results of DCPT adjacent to the boreholes confirmed these conditions.

The measured natural moisture contents range from 1 to 58% and the soil is described as dry to wet.

The layer of silt ranges in thickness from 6.0 m in the south pier to 9.1 m at the north approach. The base of the layer lies between Elevation 290.5 at the north pier to 288.7 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 Figures B1 and B2 in Appendix B.

## **5.4 Silty Clay**

A layer of silty clay was encountered on the north side of the river and extending to a point between the south pier and the south abutment. Based on the recorded SPT values ranging from 2 to 28 blows for 0.3 m of penetration, the clay is be classified as soft to very stiff. However, the vane shear strengths measured in the deposit range from 50 to greater than 100 kPa, indicating that the lower bound of the clay strengths is in fact stiff. Variations in the assessed strength are due in part to variations in the silt content and the presence of thin

seams of silt. For design purposes, the undrained strength of the clay should be considered to lie in the range of 50 to 80 kPa.

The clay is silty and layered, with the percentage of silt varying between layers. The plasticity of the clay lies in the intermediate range, as shown in Figure B9 in Appendix B.

The recorded natural moisture contents in the clay ranged from 30 to 55% and the soil is described as moist.

The thickness of the clay layer ranges from 4.0 m at the north pier and 4.8 m at the north approach to 10.1 m at the south pier and 13.2 m at the north abutment. The base of the clay layer lies at Elevation 286.5 at the north pier to Elevation 277.3 at the north abutment.

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

### **5.5 Silt**

A layer of silt was encountered below the silty clay and below the upper sandy silt where the clay was absent. This soil is predominantly silt-sized, with trace sand sizes and trace to some clay-sized particles. Based on SPT values generally ranging from 12 to 40 blows for 0.3 m of penetration, the silt is classified as compact to dense.

The measured natural moisture contents ranged from 24 to 37% and the soil is described as moist.

The thickness of the silt layer ranged from 2.1 m at the north pier to 5.2 m at the south abutment. The base of the silt layer lay between Elevation 284.4 at the north pier and Elevation 275.0 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 B5 in Appendix B.

### **5.6 Sand**

The silt layer is underlain by a layer of sandy silt to silty fine sand that forms a substantial thickness under the south approach and abutment but is less pronounced to the north of the river. Based on SPT values ranging generally from 4 to greater than 100 blows for 0.3 m of penetration, this sand is classified as loose to very dense. However, the low SPT values are considered to be due to sample disturbance and the high blows are considered to be due to isolated pockets of coarse sand or gravel. The deposit should, therefore, be treated as compact to dense.

The measured natural moisture contents ranged from 18 to 32% and the soil is described as wet.

The thickness of this soil layer varied from 5.7 m at the north abutment to 17.2 m at the south abutment. The underside of the sand layer ranged from Elevation 271.5 at the north approach to 266.5 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 Figures B6 and B7 in Appendix B.

### 5.7 Sand With Cobbles and Boulders

Below the sandy silt to sand described in the previous paragraph, the boreholes encountered a layer described as sand and gravel, trace silt with 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 some samples, indicating compact to dense conditions.

Where they could be measured, natural moisture contents ranged from 2 to 22% and the deposit is described as wet.

This deposit was not fully penetrated by any borehole but the borehole at the south abutment penetrated 10.9 m into the deposit, to elevation 254.1.

The grain size distributions of selected samples of this soil are plotted on the Record of Borehole sheets and shown in Figure B8 in Appendix B.

### 5.8 Depths to Refusal

The depths at which effective refusal was encountered, defined as an SPT value exceeding 100 blows for 0.3 m of penetration or layers of cobbles and boulders, are shown in Table 5.1.

**Table 5.1 – Refusal Depths (Elevations)**

Location	Borehole	Refusal Elevation (m)	Material
North Abutment	122N-8	268.5	Very dense sand and gravel with cobbles and boulders
North Pier	122N-7	264.5	
South Pier	122N-4	262.0	
South Abutment	122N-3	258.0	

### 5.9 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 abutment		South Pier		North Pier		North Abutment	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
Completion	0.3	295.6	0.3	294.5	4.3	292.3	--	--
Nov 10/04	0.6	295.3	0.3	294.5	--	--	--	--
Jan. 11/05	0.7	295.2	0.4	294.4	4.2	292.4	4.3	294.7

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.

A temporary artesian flow was recorded at the south pier when the borehole reached a depth of 19.8 m or Elevation 275.0. The groundwater level, however, soon stabilized below the ground surface.

## 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.

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, 99 m long, CPCI girder structure is proposed at this site and integral abutments are under consideration. A span configuration of 27:45:27 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 300.1 and the original ground surface is at Elevation 295.9, resulting in a 4.2 m high embankment.

The finished grade at the north abutment will lie at Elevation 301.8 and the original ground surface at this location is at Elevation 299.0, giving a total embankment height of 2.8 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 30 m of generally very loose to compact sandy silt and sand and firm to very stiff clay overlying very dense sand and gravel with cobbles and boulders.

Initial consideration was given to the following foundation types:

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

Appendix D 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 native soils lying near the surface 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 structure foundations on 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 sand containing cobbles and boulders below Elevation 270 at the north pier and abutment and below Elevation 265 at the south pier and abutment.

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	122N-8	268.5
North Pier	122N-7	264.5
South Pier	122N-4	262.0
South Abutment	122N-3	258.0

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 8.2.4 Pile Driving.

### 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 pier pile caps will penetrate into sandy silt overlying silty clay. It is desirable that the liners described below be sealed into the silty clay below Elevation 287.5. 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. Also, care must be taken not to drive the liners too deep or they will interfere with the driving of batter piles.

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 310 X 110	3,600 kN
HP 310 X 125	3,600 kN
HP360 X 132	4,200 kN

#### 8.2.5 Downdrag

A layer of silty clay underlies the site and long-term 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**

	South Abutment			South Pier		
Pile Type	HP 310x110	HP 310x125	HP 360x132	HP 310x110	HP 310x125	HP 360x132
Factored downdrag force (f = 1.25)	N/A	N/A	N/A	440 kN	440 kN	500 kN
	North Abutment			North Pier		
Pile Type	HP 310x110	HP 310x125	HP 360x132	HP 310x110	HP 310x125	HP 360x132
Factored downdrag force (f = 1.25)	600 kN	600 kN	700 kN	250 kN	250 kN	300 kN

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

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)

$\gamma$  = 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/m<sup>3</sup>),  $D$  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 <sup>3</sup> )	$K_p$	$S_u$ kPa	Unit Weight* (kN/m <sup>3</sup> )	Soil Conditions
South Abutment	295.9 to 280.0	1,200	3.0	-	10	Sand/Silt, loose to compact
	280.0 to 264.5	2,000	3.0	-	9	Sand, dense to very dense
	264.5 to 258.0	3,000	3.0	-	10	Sand and gravel, dense to very dense
	Below 258.0	8,000	4.0	-	11	Sand and gravel with cobbles and boulders, very dense
North Abutment	299.0 to 290.4	2,000	3.0	-	10	Sandy silt, loose to compact
	290.4 to 284.0	-	2.8	50	8	Silty clay, firm to stiff
	284.0 to 277.3	-	2.8	100	8	Silty clay, very stiff
	277.3 to 271.5	5,000	3.5	-	11	Sand , very dense
	Below 271.5	8,000	4.0		11	Sand and gravel, cobbles and 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 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 very loose sandy silt that, 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 and the possibility of the CSP being dragged down by the pile.

Backfill sand should meet the gradation shown in Table 8.6.

**Table 8.6 – Integral Abutment Sand Grading**

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

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

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

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

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

$h$  = full height of the abutment wall (metres)

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

## **8.6 Frost Protection**

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

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

## **9 EXCAVATION**

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

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

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

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**

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 F.

The stability analysis has been based on the existing configuration of the river banks. It is necessary that adequate erosion protection be provided to ensure that the river does not erode material in front of the abutments. Design of this erosion protection is not included in the foundations design and must be completed by others.

### **11.1 Long Term Stability**

#### **11.1.1 South Approach**

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

The analysis showed that a rock fill approach constructed with side slopes of 1.25H:1V have a factor of safety against slope failure of 1.6 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 1.1 was obtained. The theoretical failure plane extended to the shoulder rounding but did not intersect the road platform. These results indicate that the slope will approach a state of incipient failure under the combination of a 100 year flood and an earthquake, considered to be a low probability occurrence. Either condition occurring separately yielded a factor of safety higher than 1.1.

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 1.0 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 issued 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**

The soil conditions governing stability of the north approach embankment consist of the approach fill overlying a layer of sand and silt overlying clay at a depth of approximately 8.5 m. The proposed embankment requires the placement of comparatively minor amounts of fill at the section analysed.

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 2.0 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 1.5 was obtained.

The analyses were repeated for an earth fill approach embankment using the design side slopes. The resulting factors of safety are 2.4 under normal circumstances and 1.8 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**

<b>Location / Material</b>	<b>Condition</b>	<b>Factor of Safety</b>	<b>Figure</b>
<b>South Approach</b>			
Rock Fill	Normal W.L. No Seismic	1.6	F1
Rock Fill	100 yr flood, 0.08 Seismic	1.1	F2
Earth Fill	Normal W.L. No Seismic	1.4	F3
Earth Fill	100 yr flood, 0.08 Seismic	1.0	F4
<b>North Approach</b>			
Rock Fill	Normal W.L. No Seismic	2.0	F5
Rock Fill	100 yr flood, 0.08 Seismic	1.5	F6
Earth Fill	Normal W.L. No Seismic	2.4	F7
Earth Fill	100 yr flood, 0.08 Seismic	1.8	F8

## **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 sandy silt
- That the entire embankment was placed at once.

The result of analysis gave a factor of safety of 1.4 for rock fill and for earth fill. This is an acceptable factor 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 sandy 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 1.7 for rock fill and 1.6 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 Settlement

The 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	Not applicable	
North Abutment	170 mm	12 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.

### 11.4 Seismic Considerations

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

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

### 11.5 Forward Slopes

It is recommended that the forward slopes 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.

### 11.6 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.

### 11.7 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 <sub>f</sub> = 280 kPa SLS = 180 kPa	ULS <sub>f</sub> = 360 SLS = 240
Coefficient of sliding resistance	0.5	0.5
Estimated settlement	120 mm	80 mm

The near surface foundation soils, particularly at the south abutment, consist of very loose to compact sandy silts. 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 294.0 at the south abutment and Elevation 298.5 at the north abutment. Lower founding elevation may be required to accommodate the required thickness of engineered fill.

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.

### **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)

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

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

$q$  = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

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

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

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

**Table 14.1 – Earth Pressure Coefficient (K)**

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

\* For wing walls.

## 15 SEISMIC CONSIDERATIONS

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

### 15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

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

The Soil Profile Type at this site has been classified as Type 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) method<sup>1</sup>.

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

## 15.3 Retaining Wall Dynamic Earth Pressures

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

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

$\phi$	= 35° for OPSS Granular A or Granular B Type II
$\phi$	= 32° for OPSS Granular B Type I
$\phi$	= 42° for rock fill
$\delta$	= 50% of $\phi$

Where  $\phi$  = the angle of internal friction of the backfill and  $\delta$  = 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.

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

## 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

**Table 15.1**  
**Earth pressure Coefficients for Seismic Design**

Earth Pressure Coefficient (K) for Earthquake Loading						
Condition	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)
	0.28	0.46	0.31	0.58	0.21	0.30
	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

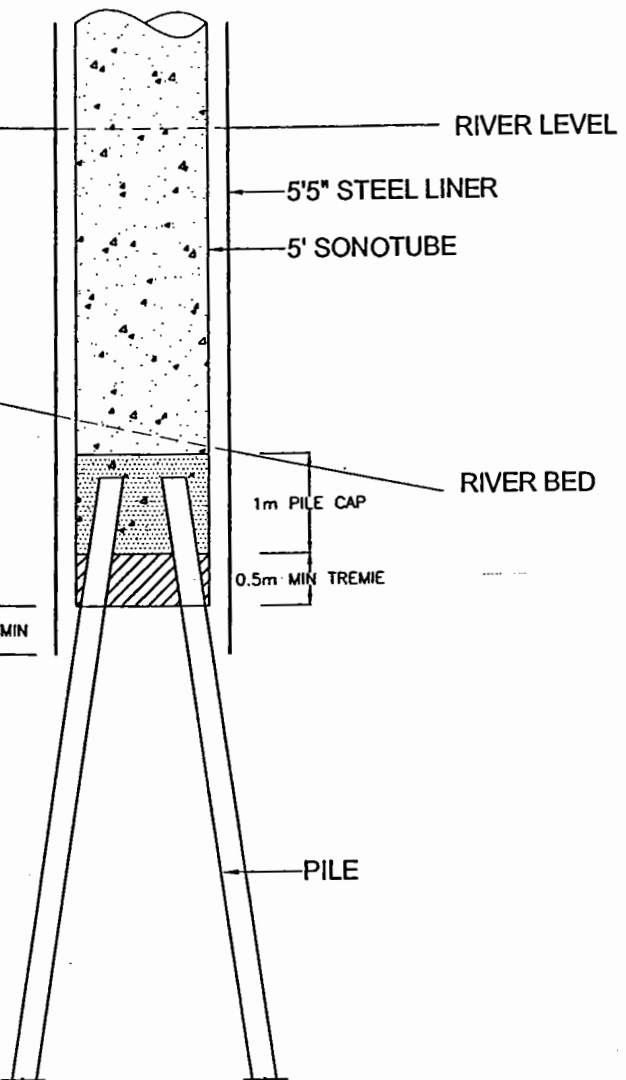
N.B.

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

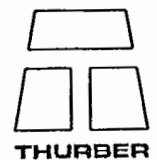


## RISKS:

- 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)



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 <sup>(1)</sup> 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

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



### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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






 Water Level  
 Shear Strength Determination by Pocket Penetrometer

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

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS			
Fresh (FR)	No visible signs of weathering.				
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE		
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE		
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE		
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL		
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)		
DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
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 122N-1

1 OF 2

METRIC

W.P. 474-93-01 LOCATION N 5047474.4 E 316758.4 Magnetawan South, NBL, 122N-1 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/HS  
 DATUM Geodetic DATE 21.10.04 - 21.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
294.9								20 40 60 80 100				
0.0	PEAT, fibrous, trace rootlets Dark Brown		1	SS	2			20 40 60 80 100				47
0.2	Sandy SILT, occasional iron oxide staining Very Loose to Compact Brown Wet		2	SS	6		294	20 40 60 80 100				
	Becoming Grey		3	SS	11		293	20 40 60 80 100				0 3 89 8
			4	SS	10			20 40 60 80 100				
291.9							292	20 40 60 80 100				
3.0	SAND, fine to medium grained, trace gravel Compact Grey Wet		5	SS	15		291	20 40 60 80 100				
			6	SS	16		290	20 40 60 80 100				
								20 40 60 80 100				
288.8							289	20 40 60 80 100				
6.1	Silty SAND, fine to coarse grained Compact Grey Wet		7	SS	20		288	20 40 60 80 100				
								20 40 60 80 100				
287.3								20 40 60 80 100				
7.6	Sandy SILT Compact Grey Wet		8	SS	15		287	20 40 60 80 100				0 26 68 6
								20 40 60 80 100				
			9	SS	21		286	20 40 60 80 100				
								20 40 60 80 100				

Continued Next Page

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

# RECORD OF BOREHOLE No 122N-1

2 OF 2

METRIC

W.P. 474-93-01 LOCATION N 5047474.4 E 316758.4 Magnetawan South, NBL, 122N-1 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/HS  
 DATUM Geodetic DATE 21.10.04 - 21.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
281.8			10	SS	20		284						1 33 63 3
283							283						
282							282						
13.1	END OF BOREHOLE AT 13.11 m. BOREHOLE OPEN TO 10.67 m AND WATER LEVEL AT 3.66 m UPON COMPLETION. BOREHOLE GROUTED TO SURFACE.												

# RECORD OF BOREHOLE No 122N-3

1 OF 5

METRIC

W.P. 474-93-01 LOCATION N 5047496.8 E 316752.2 Magnetawan South, NBL, 122N-3 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 13.10.04 - 14.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
295.9 0.0	Sandy SILT, interbedded with layers of silty sand Loose to Very Loose Brown Dry		1	SS	6		295						0 50 47 3
			2	SS	5								
			3	SS	4								
			4	SS	2								
			5	SS	2								
			6	SS	2								
			7	SS	5								
288.9 7.0	SILT, trace clay, trace sand Compact Grey Wet		8	SS	20		288						0 91 9 (SI+CL)
			9	SS	14								
	occasional sand seams												

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+ 3 . X 3 : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-3

2 OF 5

METRIC

W.P. 474-93-01 LOCATION N 5047496.8 E 316752.2 Magnetawan South, NBL, 122N-3 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 13.10.04 - 14.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
283.7			10	SS	12		285							0 2 91 7
12.2	SAND, fine grained, trace silt Loose to Compact Grey Wet		11	SS	8		284							
			12	S	7		283							
			13	SS	11		282							
			14	SS	6		281							
			15	SS	23		280							
			16	SS	50		279							
							278							0 86 14 (SI+CL)
							277							
							276							

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

# RECORD OF BOREHOLE No 122N-3

3 OF 5

METRIC

W.P. 474-93-01 LOCATION N 5047496.8 E 316752.2 Magnetawan South, NBL, 122N-3 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 13.10.04 - 14.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	becoming silty		17	SS	10		275							0 78 22 (SI+CL)
							274							
							273							
			18	SS	23		272							
							271							
							270							
			19	SS	16		269							
							268							
							267							
266.5							266							
29.4	SAND and GRAVEL, trace silt, occasional cobbles Compact to Very Dense													

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+ 3, x 3 : Numbers refer to  
Sensitivity

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(%) STRAIN AT FAILURE





# RECORD OF BOREHOLE No 122N-3

5 OF 5

METRIC

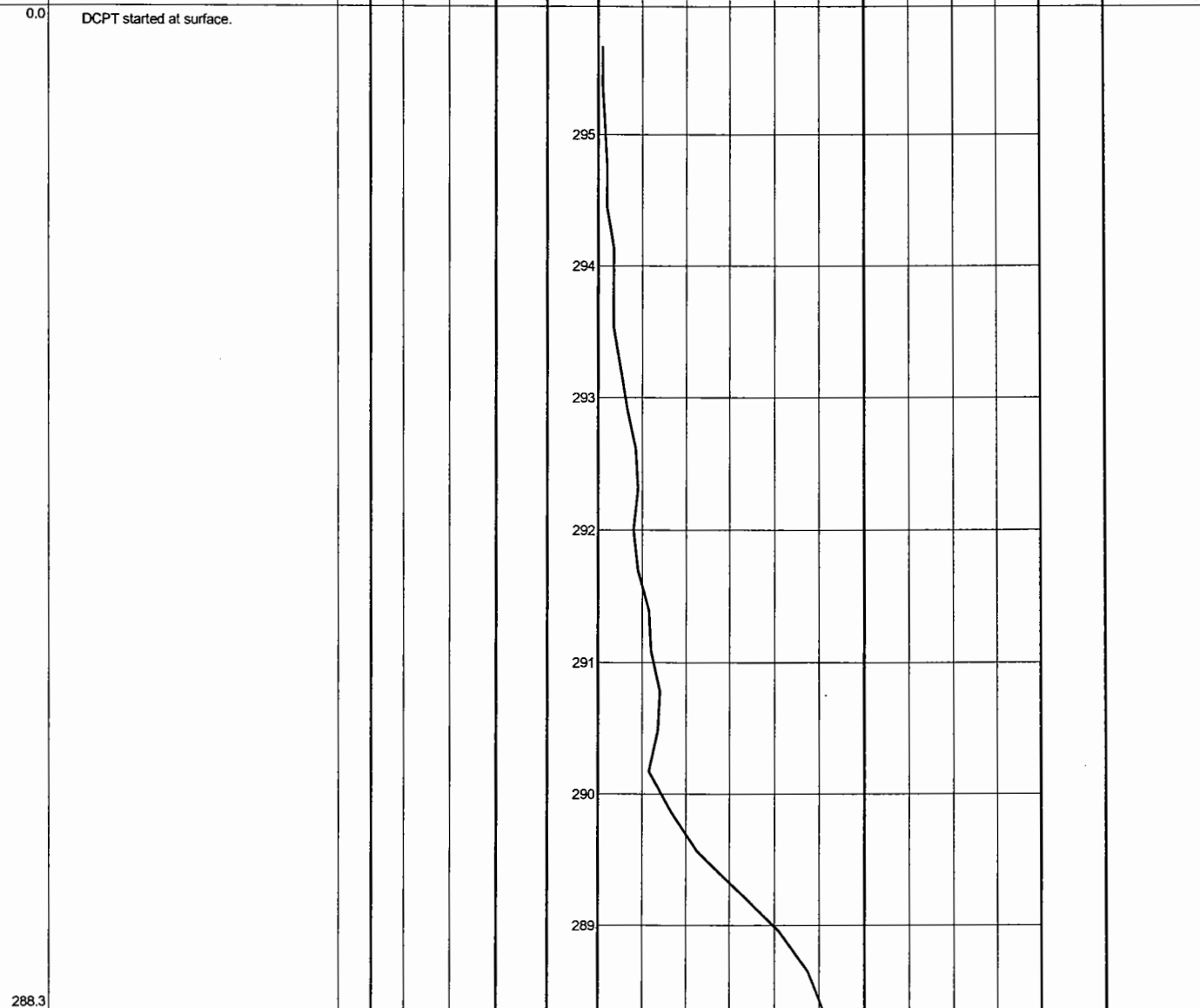
W.P. 474-93-01 LOCATION N 5047496.8 E 316752.2 Magnetawan South, NBL, 122N-3 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 13.10.04 - 14.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
254.1			3	RUN													
			4	RUN													
41.8	END OF BOREHOLE AT 41.76 m. BOREHOLE OPEN TO 41.76 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen.  WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) Completion 0.3 295.6 10-NOV-04 0.6 295.3 11-JAN-05 0.7 295.2																

ONTMT4S 122N.GPJ 12/01/05

## METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES		W <sub>p</sub>	W			W <sub>L</sub>
295.9							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100				20 40 60		

[illegible]

# RECORD OF BOREHOLE No 122N-4

1 OF 5

METRIC

W.P. 474-93-01 LOCATION N 5047511.2 E 316731.3 Magnetawan South, NBL, 122N-4 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 19.10.04 - 21.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
294.8	TOPSOIL (50 mm)													
0.0 0.1	Sandy SILT, interbedded with layers of silty sand Very Loose Brown Wet		1	SS	2									
			2	SS	2		294							
			3	SS	2		293						0 34 62 4	
	Becoming Grey		4	SS	2		292						0 68 30 3	
			5	SS	2		291							
			6	SS	2		290							
							289							
288.7							288							
6.1	Silty CLAY, trace sand Soft to Firm Grey Wet		7	SS	2		287						0 3 62 35	
			8	SS	2		286						0 1 41 59	
			9	SS	2		285							

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

20  
15  
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5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-4

2 OF 5

METRIC

W.P. 474-93-01 LOCATION N 5047511.2 E 316731.3 Magnetawan South, NBL, 122N-4 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 19.10.04 - 21.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
			1	TW	PH									
			10	SS	2		284							
			11	SS	6		283							
							282							
			12	SS	8									
							281							
	Becoming Very Stiff, Varved		13	SS	19		280							
278.7							279							
16.2	SILT, trace sand Compact to Dense Grey Wet		14	SS	24		278							
							277							
			15	SS	40		276							
							275							
275.0			16	SS	38									
19.8	Sandy SILT													

ONTMT4S 122N.GPJ 12/01/05

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

**METRIC**

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60	W <sub>p</sub> W W <sub>L</sub>			
	Loose to Compact Grey Wet Artesian pressure condition from 19.8 m		17	SS	4		274					0 27 67 6	
							273						
							272						
			18	SS	23		271					0 10 87 3	
							270						
							269						
268.0							268						
26.8	SAND, fine grained, trace silt Dense to Compact Grey Wet		19	SS	40		267						
							266						
							265						

+ 3, x 3: Numbers refer to Sensitivity

**METRIC**[illegible]

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

## METRIC



SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES		W <sub>P</sub>	W		
							SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE 20   40   60   80   100				20   40   60	

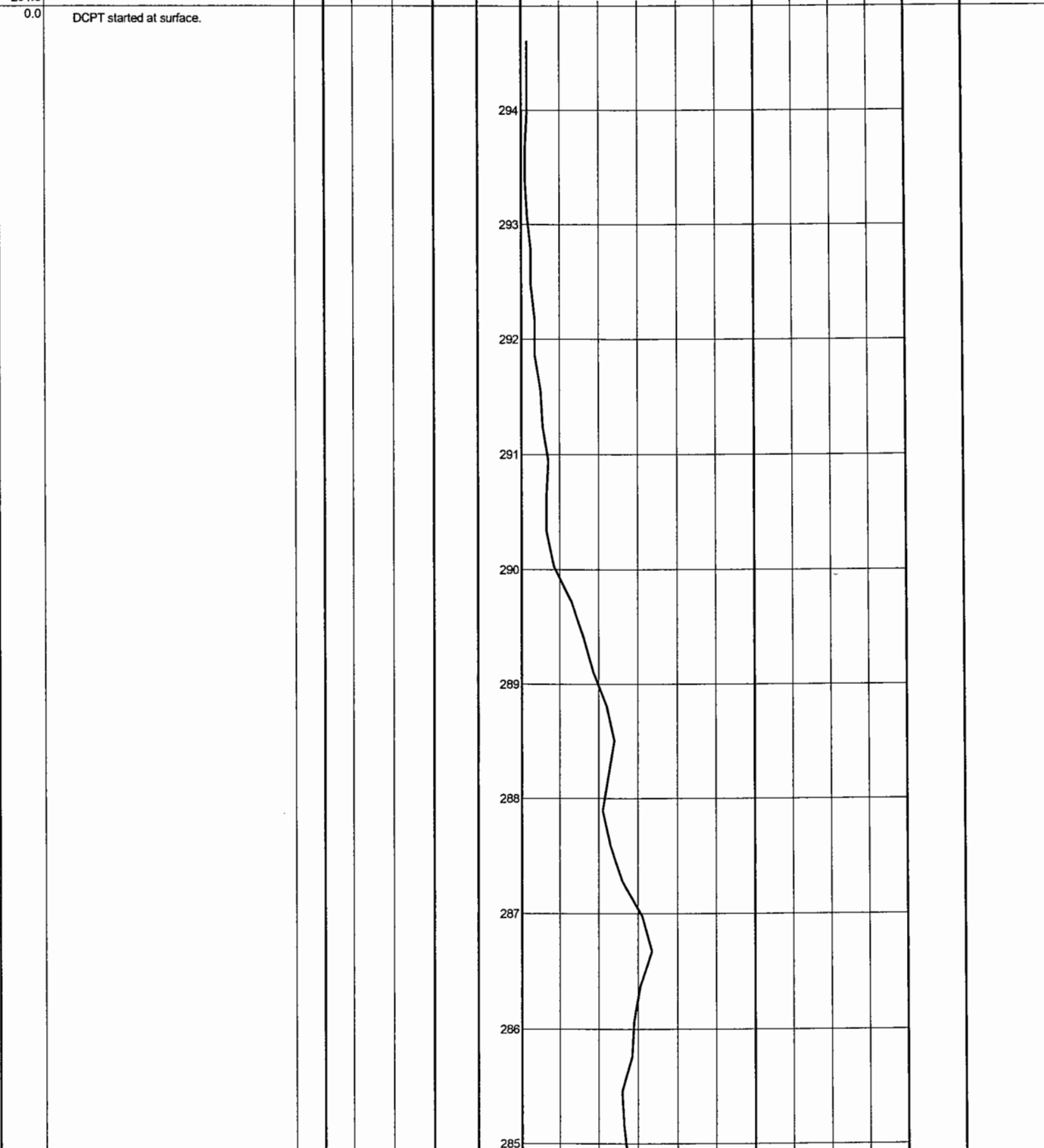
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ONTMT4S 122N.GPJ 12/01/05



**METRIC**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100						
294.8								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60					



Continued Next Page

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

(%) STRAIN AT FAILURE

## METRIC

SOIL PROFILE						SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60 80 100										
280.8 14.0	END OF DCPT AT 14.02 m.								284 283 282 281								

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

# RECORD OF BOREHOLE No 122N-7

1 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047562.0 E 316715.1 Magnetawan South, NBL, 122N-7 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
296.6							20 40 60 80 100							
0.0	Sandy SILT, interbedded with layers of silty sand, occasional iron oxide staining, trace rootlets at surface Loose to Compact Dark Brown to Brown Moist to Wet		1	SS	11									
			2	SS	18									4 91 5 (SI+CL)
			3	SS	11									
			4	SS	9									
			5	SS	9									0 1 91 8
	becoming grey below 5m		6	SS	15									
290.5														
6.1	Silty CLAY Very Stiff to Soft Grey		7	SS	16									0 1 67 32
			8	SS	4									
			1	TW	PH									

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-7

2 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047562.0 E 316715.1 Magnetawan South, NBL, 122N-7 ORIGINATED BY GA  
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
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		
286.5 10.1	SILT, occasional sand seams Compact Grey Wet		9	SS	15		286						
284.4 12.2	Silty SAND, fine grained, trace silt Loose to Compact Grey Wet		10	SS	5		284						0 62 36 2
			11	SS	26		283						
			12	SS	15		282						
			13	SS	40		281						0 37 60 3
	Becoming Dense						280						
278.3 18.3	SAND, medium grained, trace gravel Dense to Very Dense Grey Wet		14	SS	30		278						
							277						

Continued Next Page

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

# RECORD OF BOREHOLE No 122N-7

3 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047562.0 E 316715.1 Magnetawan South, NBL, 122N-7 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	20 40 60		
			15	SS	24								
						276							
						275							
						274							
			16	SS	100/								
					.075	273							
						272							
						271							
			17	SS	35	270							
269.5						269							
27.1	SAND and GRAVEL, trace silt. occasional cobbles and boulders Very Dense Grey Wet					268							
	frequent cobbles and boulders below 28.3 m					267							
			18	SS	100/								
					.0								
			19	SS	43								

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4S 122N.GPJ 12/01/05

# RECORD OF BOREHOLE No 122N-7

4 OF 4



METRIC

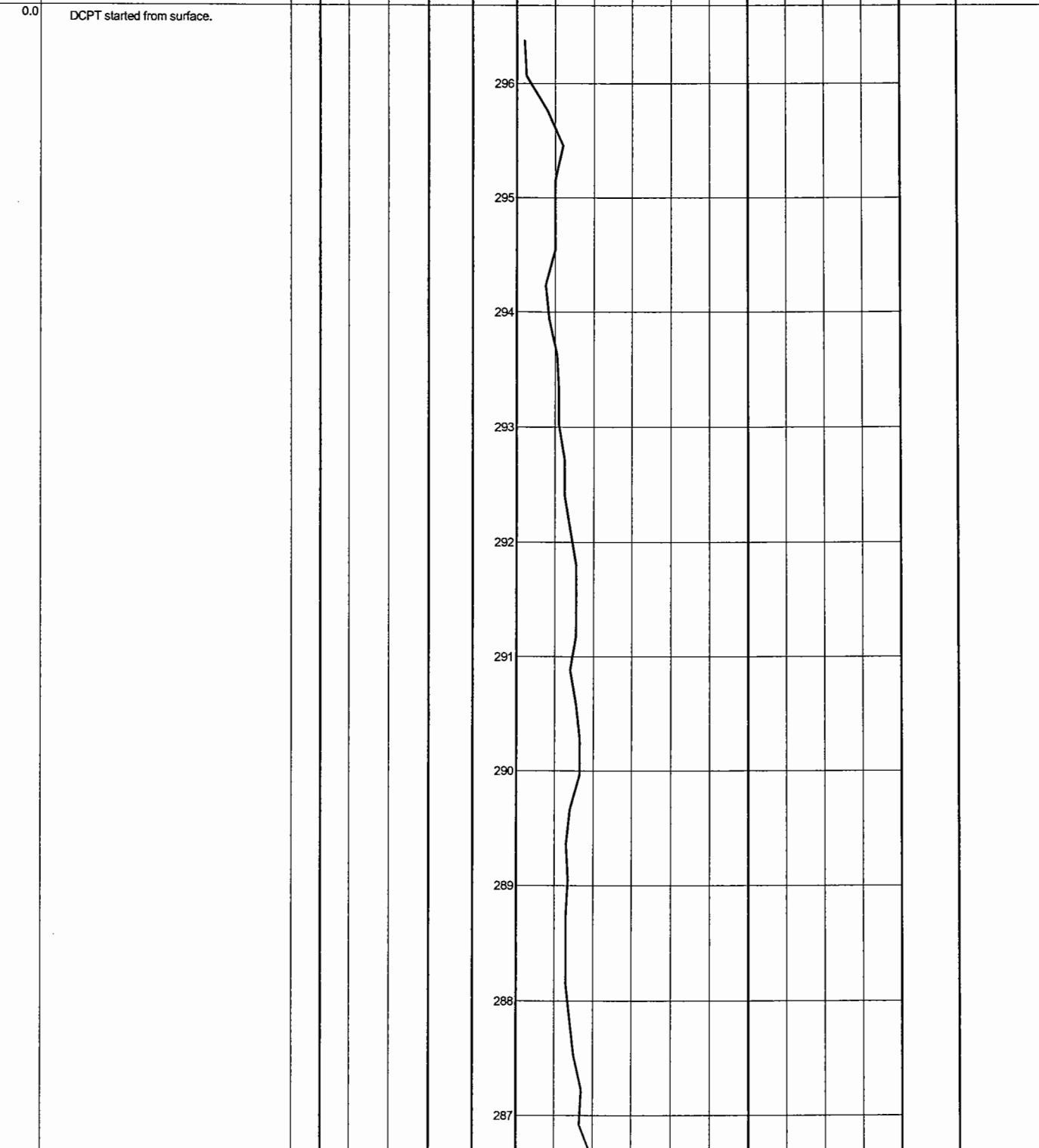
W.P. 474-93-01 LOCATION N 5047562.0 E 316715.1 Magnetawan South, NBL, 122N-7 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 25.10.04 - 28.10.04 CHECKED BY AEG

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (SI+CL)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
			20	SS	100/ .150											
			21	SS	100/ .200											
			22	SS	100/ .150											
259.9			23	SS	100/ .150											
36.7	END OF BOREHOLE AT 36.73 m. BOREHOLE OPEN TO 36.58 m IN CASING AND WATER LEVEL AT 4.7 m UPON COMPLETION. Piezometer installation consist of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen.  WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) Completion 4.3 292.3 11-JAN-05 4.2 292.4															

ONTMT4S 122N.GPJ 1201/05

## METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT 	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES						
296.6							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60		GR SA SI	



(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-7A

2 OF 2

METRIC

W.P. 474-93-01 LOCATION N 5047562.0 E 316715.1 Magnetawan South, NBL, 122N-7A ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/HS  
 DATUM Geodetic DATE 29.10.04 - 29.10.04 CHECKED BY AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
279.5												
17.1	END OF DCPT AT 17.07 m.											

ONTMT4S 122N.GPJ 12/01/05



# RECORD OF BOREHOLE No 122N-8

1 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047577.0 E 316696.9 Magnetawan South, NBL, 122N-8 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 29.10.04 - 31.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
299.0														
0.0	Sandy SILT, interbedded with layers of silty sand, trace rootlets and wood fibers at surface Compact to Loose Dark Brown to Brown Moist		1	SS	12		299							
			2	SS	9		298							0 61 37 2
			3	SS	13		297							
			4	SS	8		296							0 2 93 5
			5	SS	11		295							
			6	SS	12		294							
	becoming grey		7	SS	13		293							
			8	SS	19		292							
			9	SS	12		291							0 0 86 14
290.4							290							
8.5	Silty CLAY Stiff to Firm Grey Wet													

ONTMT4S 122N.GPJ 12/01/05

Continued Next Page

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

20  
15  
10  
5  
0  
5  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-8

2 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047577.0 E 316696.9 Magnetawan South, NBL, 122N-8 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 29.10.04 - 31.10.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W P W W L	20 40 60	GR SA SI CL						
	becoming soft		10	SS	4		289										
							288										
							287										
	silty seams throughout stratum		1	TW	PH		286										
							285										
			11	SS	2		284										
	becoming varved						283										
			12	SS	2		282										
							281										
							280										
	becoming stiff		13	SS	10												
			14	SS	16												

# RECORD OF BOREHOLE No 122N-8

3 OF 4

METRIC

W.P. 474-93-01 LOCATION N 5047577.0 E 316696.9 Magnetawan South, NBL, 122N-8 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing COMPILED BY WM/HS  
 DATUM Geodetic DATE 29.10.04 - 31.10.04 CHECKED BY 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	20 40 60		
277.3	SAND, trace to some silt Very Dense Grey Wet		15	SS	90		279							GR SA SI CL 0 0 83 16
278														
277														
276														
275														
274														
273														
272														
271.5	SAND and GRAVEL, trace silt, occasional boulders and cobbles		16	SS	85/ .200		276							
275														
274														
273														
272														
271														
270														
270														

Continued Next Page

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

**METRIC**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV. DEPTH	DESCRIPTION	STRAT.PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100			W <sub>p</sub>	W	W <sub>L</sub>
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			WATER CONTENT (%)						
								20	40	60	80	100					

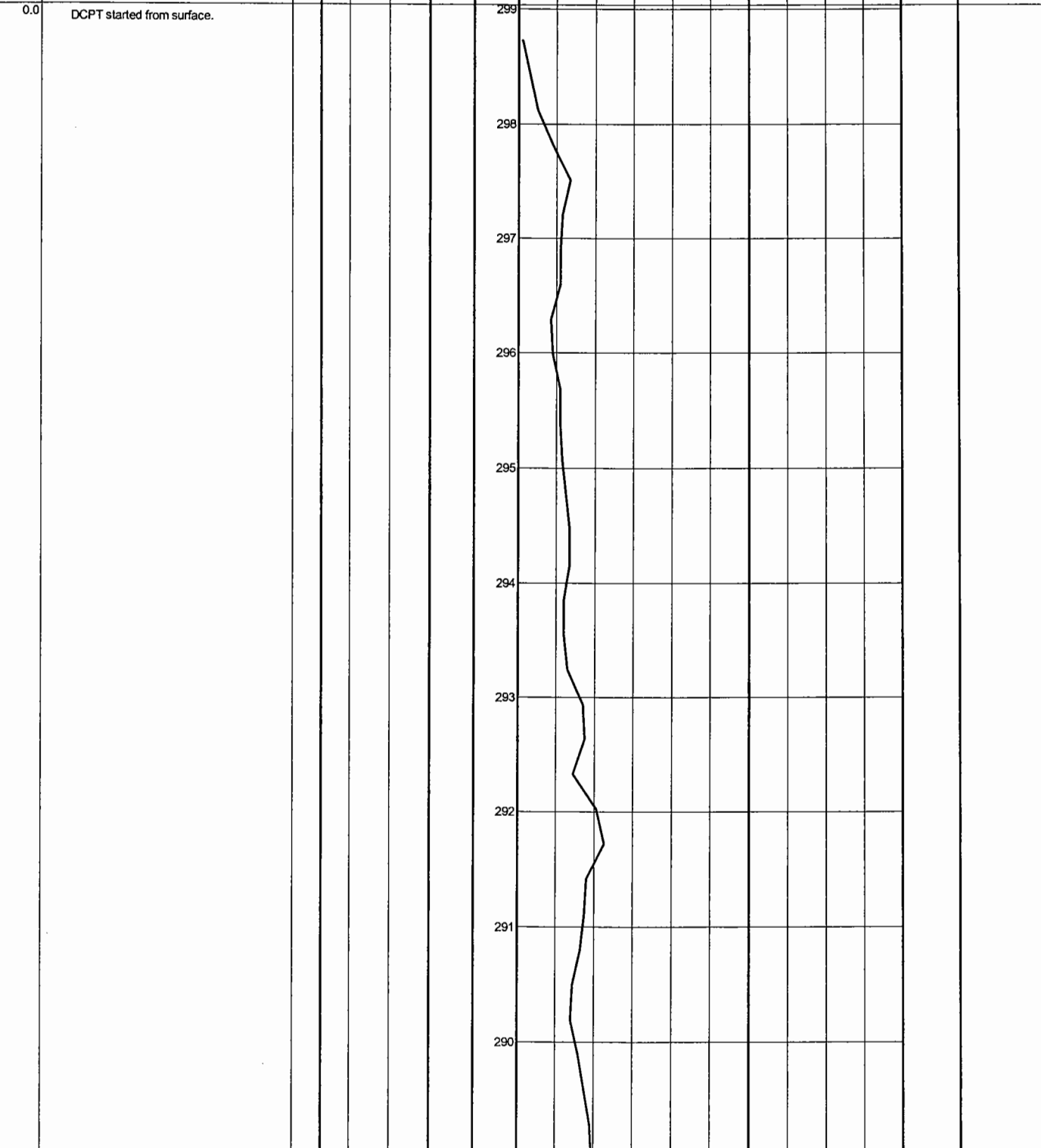
[illegible]

ONTMT4S 122N.GPJ 12/01/05

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

## METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80			100
299.0														
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%) 20 40 60					
							20	40	60	80	100			



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

## METRIC

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

# RECORD OF BOREHOLE No 122N-10

1 OF 2

METRIC

W.P. 474-93-01 LOCATION N 5047594.0 E 316692.2 Magnetawan South, NBL, 122N-10 ORIGINATED BY GA  
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / Dynamic Cone Penetration Test (DCPT) COMPILED BY WM/HS  
 DATUM Geodetic DATE 01.11.04 - 01.11.04 CHECKED BY 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								○ UNCONFINED   + FIELD VANE	● QUICK TRIAXIAL   × LAB VANE	W <sub>P</sub>	W	W <sub>L</sub>			
299.4						20	40	60	80	100	20	40	60		
0.0	Sandy SILT, interbedded with layers of silty sand Compact to Loose Brown Moist to Wet		1	SS	20							○			
			2	SS	9							○			0   4   88   8
			3	SS	8							○			
			4	SS	20							○			
			5	SS	6							○			
	Becoming Grey		6	SS	13							○			
			7	SS	16							○			0   1   87   12
			8	SS	23							○			

Continued Next Page

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

20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

ONTM4S 122N.GPJ 12/01/05

**METRIC**

+ 3, x 3: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No 122N-10A

1 OF 2

METRIC

W.P. 480-93-00 LOCATION Magnetawan South, NBL, 122N-10A ORIGINATED BY WRW  
 HWY 11 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 06.04.05 - 06.04.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W P	W	W L	20 40 60			
299.4	Refer to BH 122N-10 Augered to 11.28 m.														
299															
298															
297															
296															
295															
294															
293															
292															
291															
290															

Continued Next Page

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

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 122N-10A

2 OF 2

METRIC

W.P. 480-93-00 LOCATION Magnetawan South, NBL, 122N-10A ORIGINATED BY WRW  
 HWY 11 BOREHOLE TYPE NW Casing COMPILED BY WM  
 DATUM Geodetic DATE 06.04.05 - 06.04.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
							20 40 60 80 100	20 40 60												
288.1	SAMPLING STARTED AT 11.3m						289													
11.3	Silty CLAY Very Stiff Grey		1	SS	23		288													
							287		3.7											
			1	TW	PH		286													
285.5									1.5											
13.9	SILT, some clay, some sand Dense Grey		2	SS	33		285													
							284													
284.0							283													
15.4	Silty SAND, trace clay Dense to Compact Grey Wet		3	SS	30		282													
			4	SS	28															
281.4																				
18.0	END OF BOREHOLE AT 17.98 m. WATER LEVEL AT 0.30 m UPON COMPLETION OF DRILLING. BOREHOLE GROUTED WITH BENSEAL BENTONITE TO SURFACE.																			

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

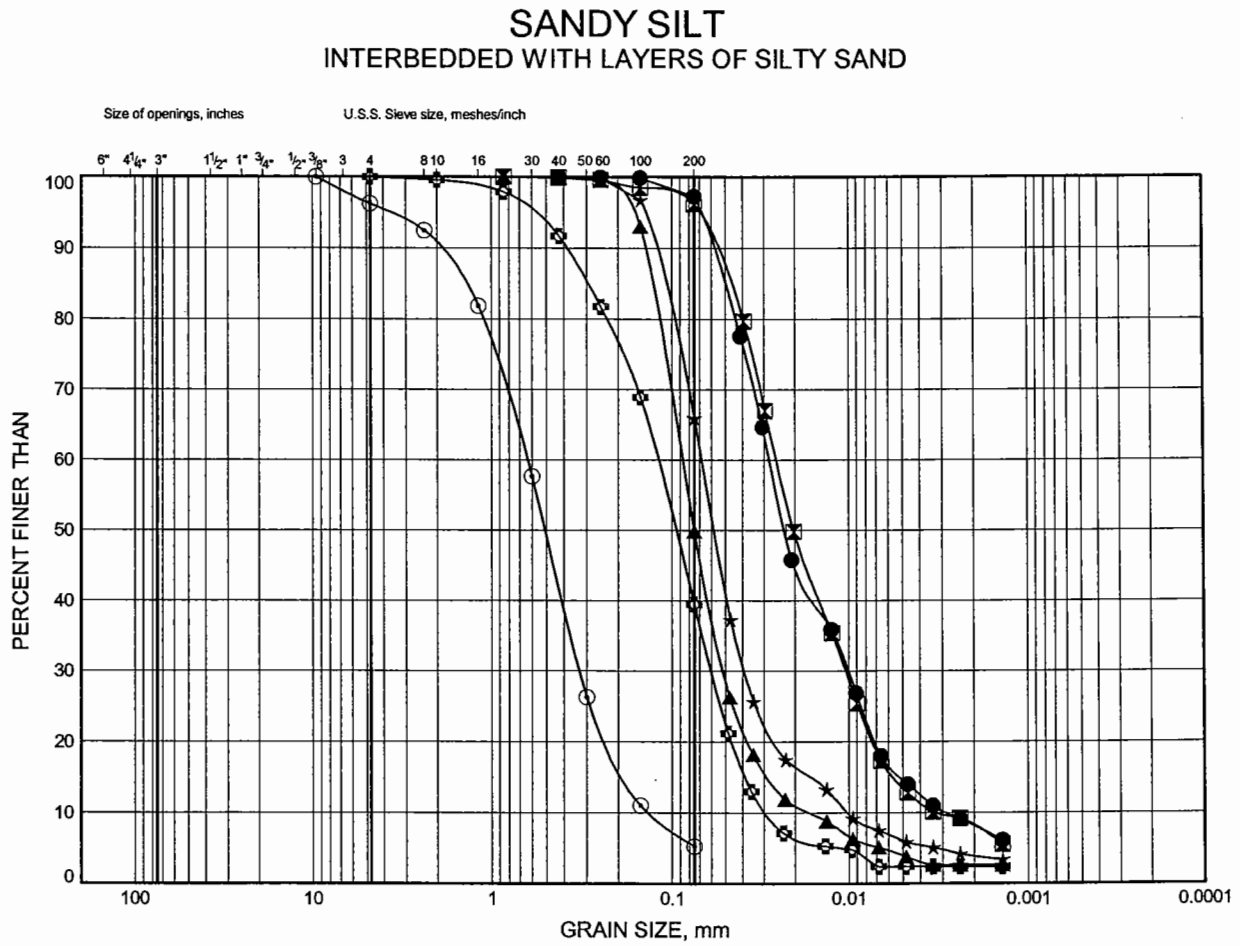
20  
15  
10  
(%) STRAIN AT FAILURE

## **Appendix B**

### **Laboratory Test Results**

# Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

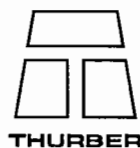
FIGURE B1



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-1	1.83	293.09
⊠	122N-10	1.07	298.36
▲	122N-3	2.59	293.30
★	122N-4	1.83	293.00
⊙	122N-7	1.07	295.53
⊕	122N-8	1.07	297.89

Date January 2005  
Project 474-93-01

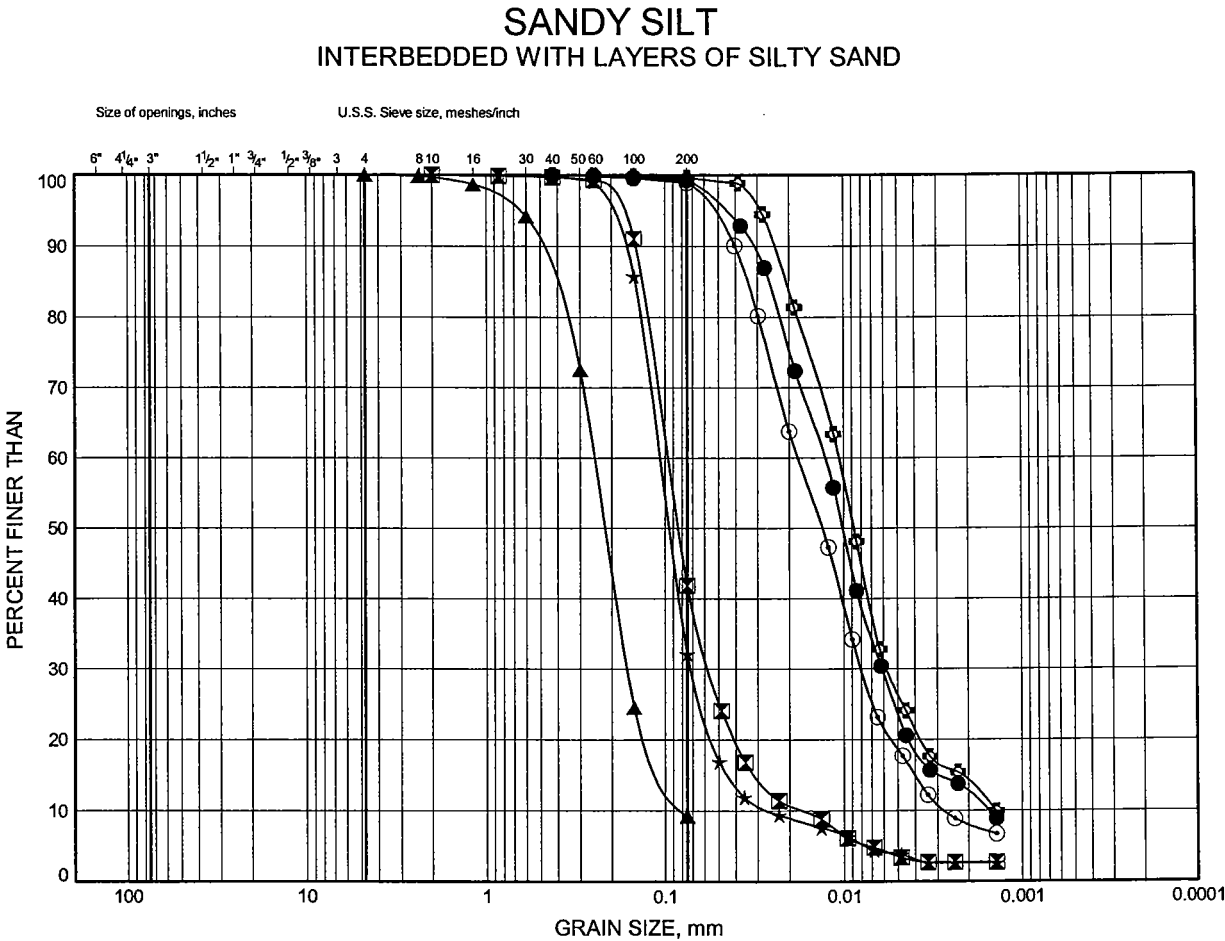


Prep'd HS  
Chkd. AEG

Hwy 11 Katrine

# GRAIN SIZE DISTRIBUTION

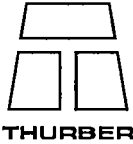
FIGURE B2



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-10	6.32	293.11
⊠	122N-3	3.35	292.54
▲	122N-3	6.32	289.57
★	122N-4	3.35	291.48
⊙	122N-7	3.35	293.25
⊕	122N-8	7.85	291.11

Date January 2005  
 Project 474-93-01

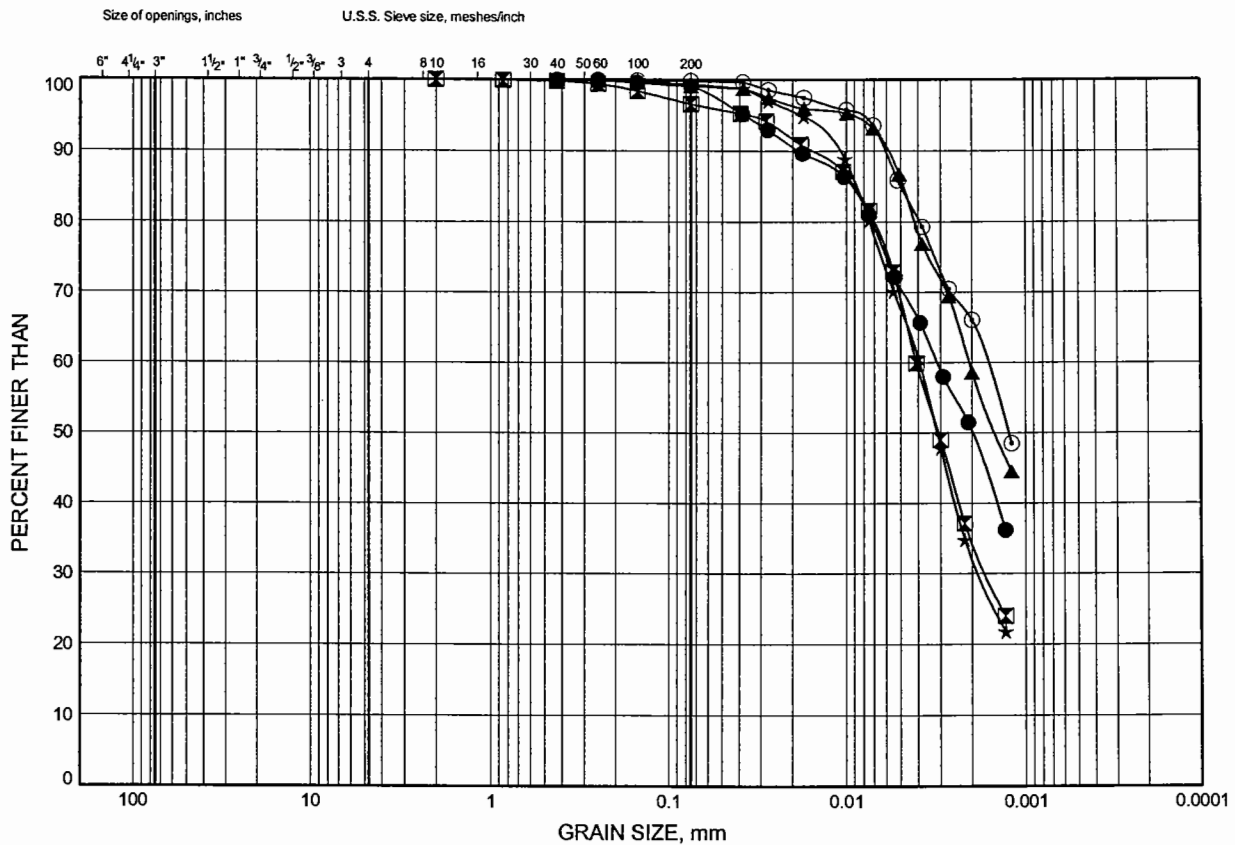


Prep'd HS  
 Chkd. AEG

# Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B3

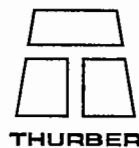
## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-10	10.90	288.53
⊠	122N-4	6.32	288.51
▲	122N-4	7.92	286.91
★	122N-7	6.32	290.28
⊙	122N-8	13.94	285.02

Date January 2005  
Project 474-93-01

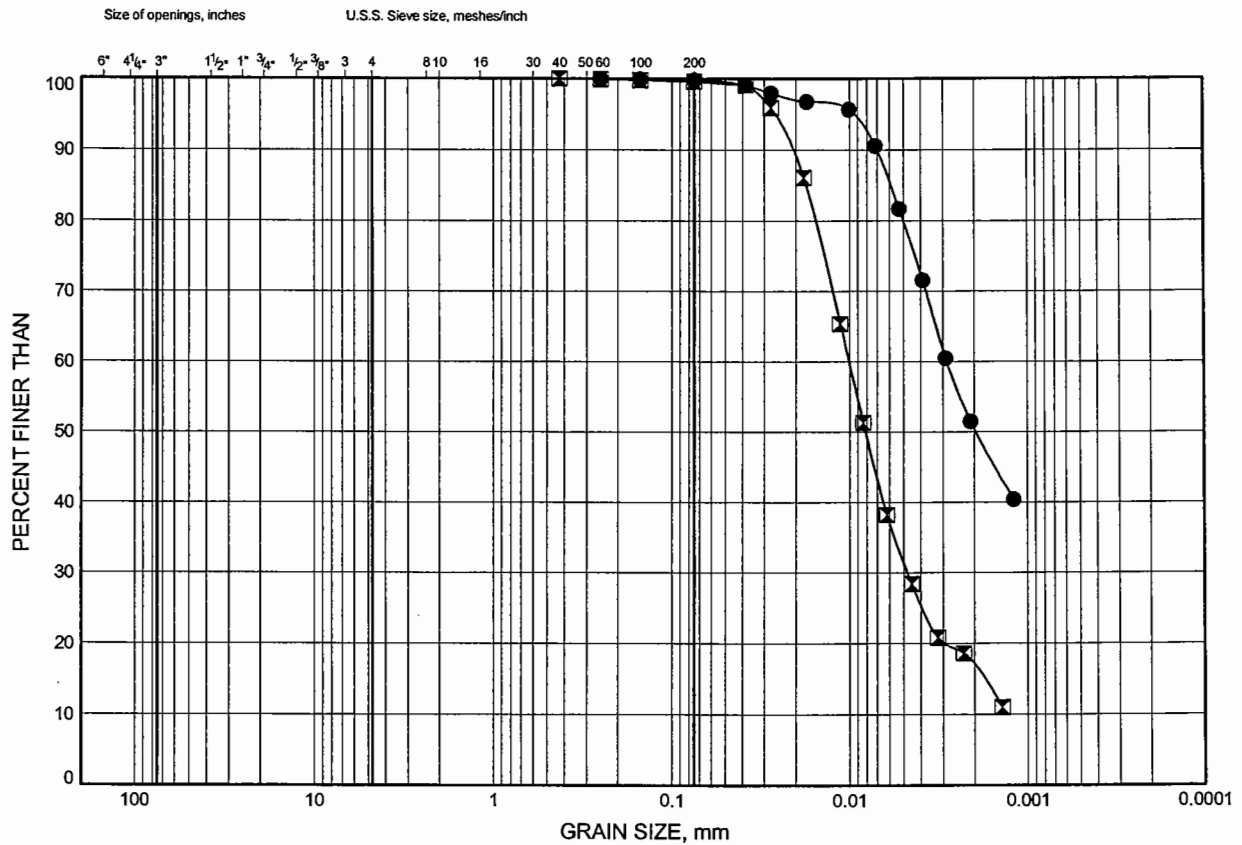


Prep'd HS  
Chkd. AEG

# Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B4

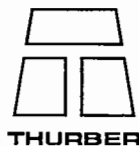
## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-8	15.47	283.49
⊠	122N-8	20.04	278.92

Date January 2005  
Project 474-93-01

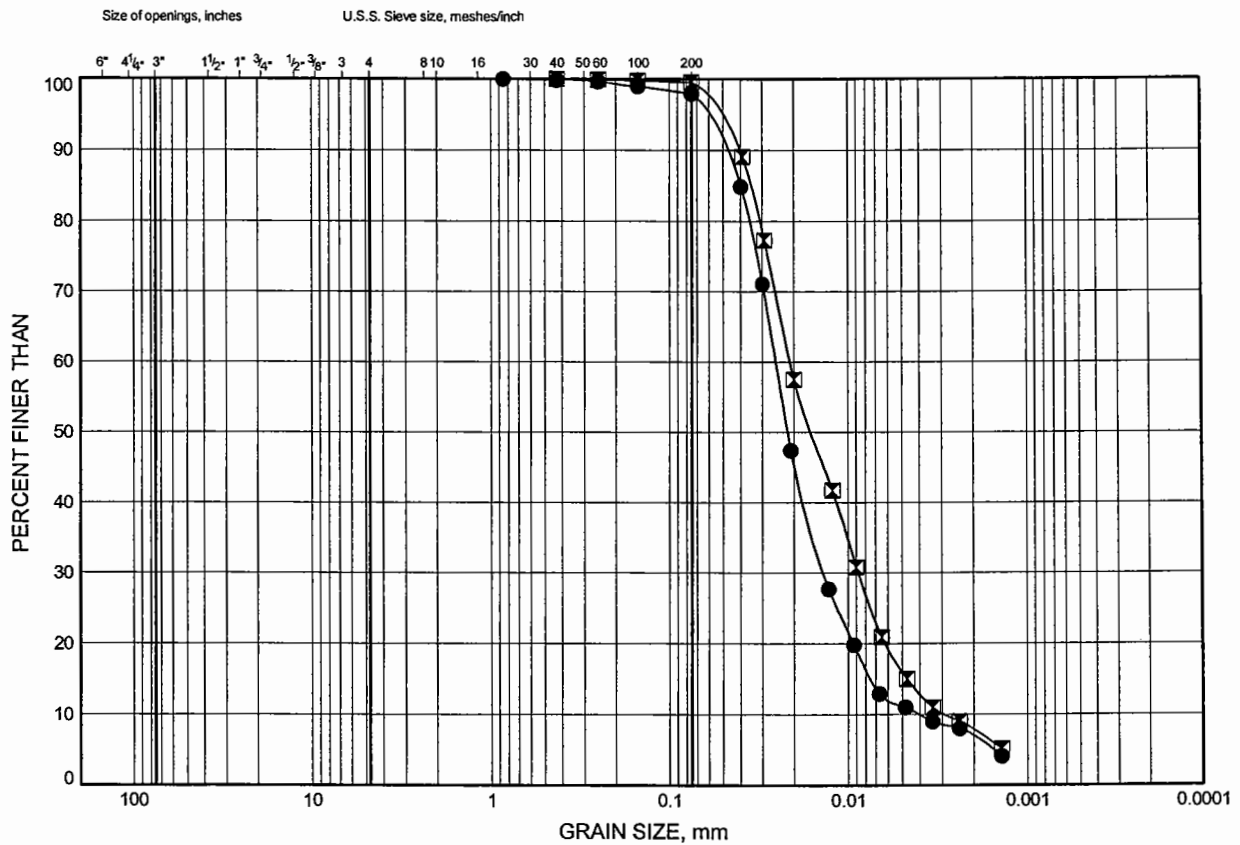


Prep'd HS  
Chkd. AEG

# Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B5

## SILT



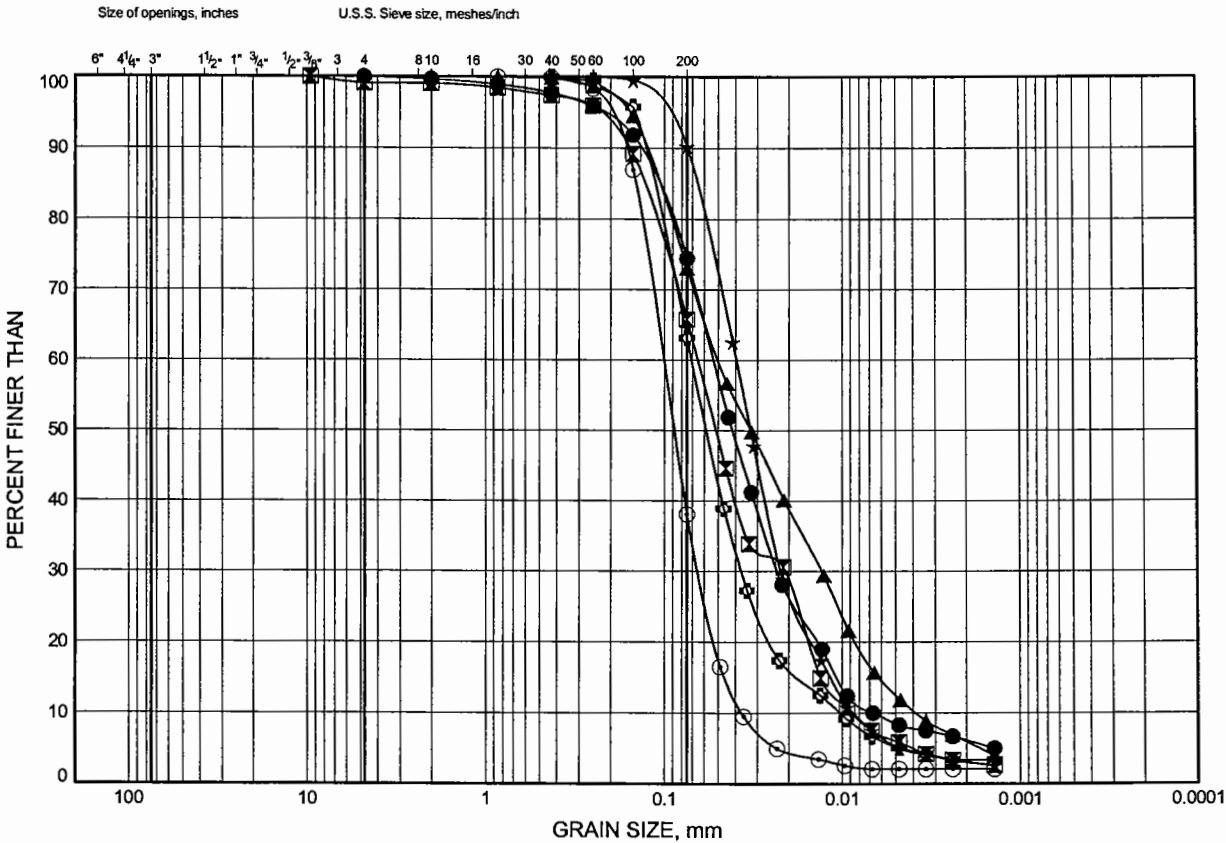


# Hwy 11 Katrine

## GRAIN SIZE DISTRIBUTION

FIGURE B6

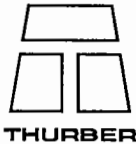
### SANDY SILT TO SAND



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-1	7.85	287.07
⊠	122N-1	10.90	284.02
▲	122N-4	20.96	273.87
★	122N-4	24.00	270.83
⊙	122N-7	12.42	284.18
⊕	122N-7	15.47	281.13

Date January 2005

Project 474-93-01



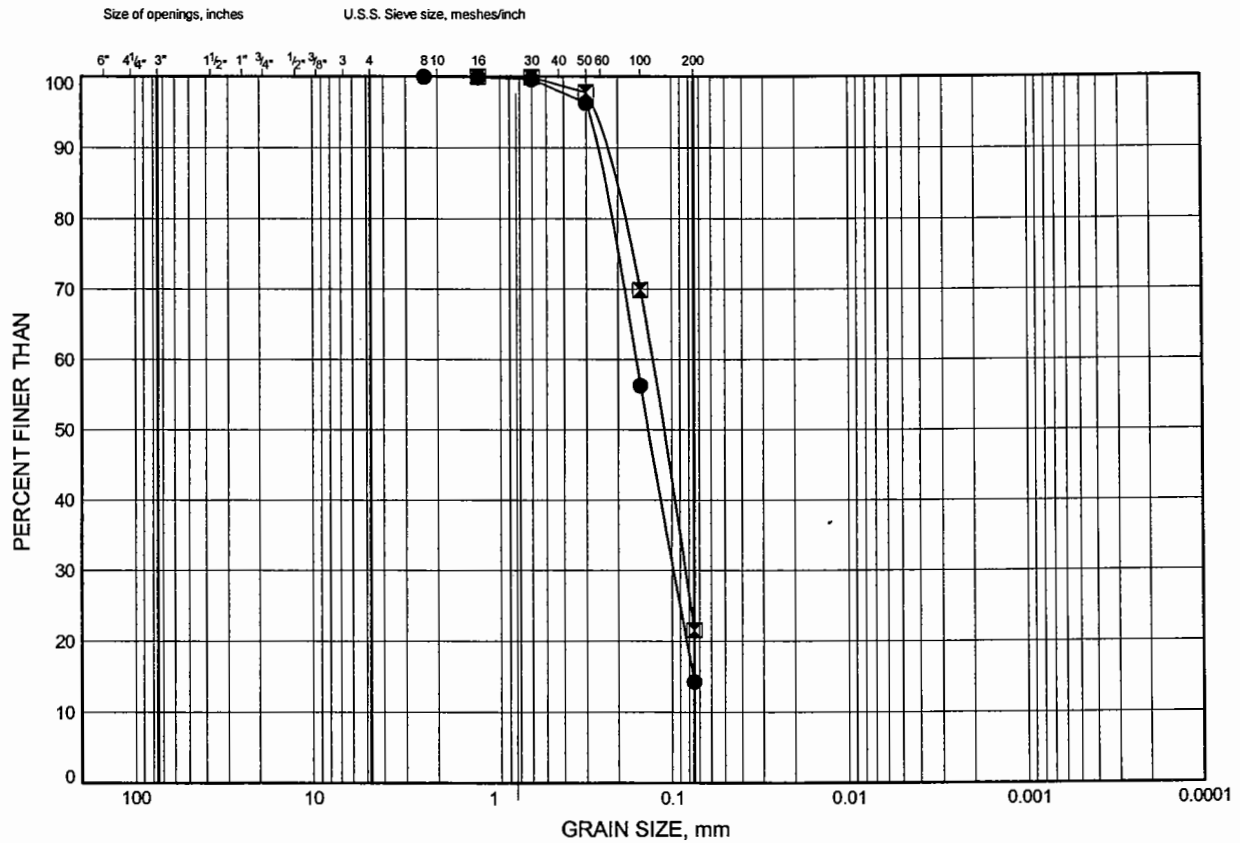
Prep'd HS

Chkd. AEG

# Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B7

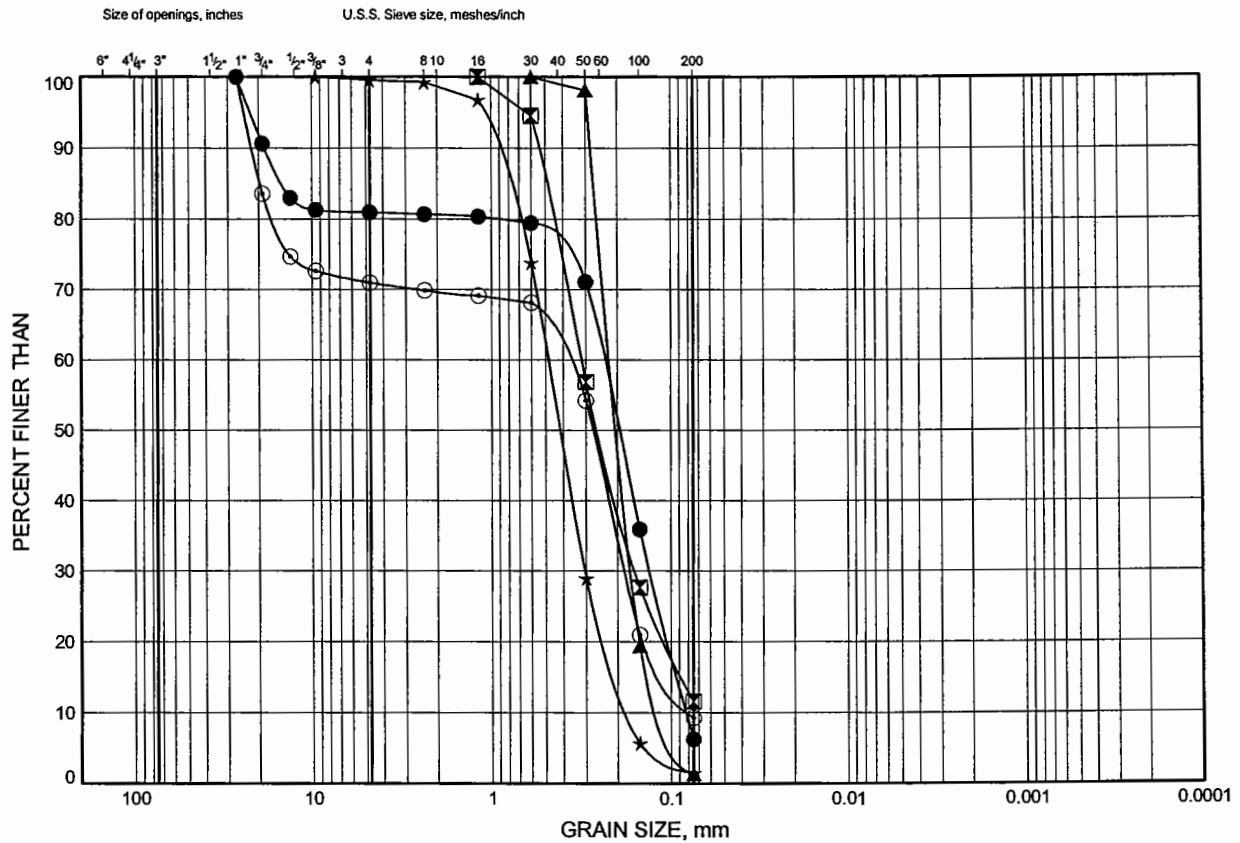
## SANDY SILT TO SAND



# Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B8

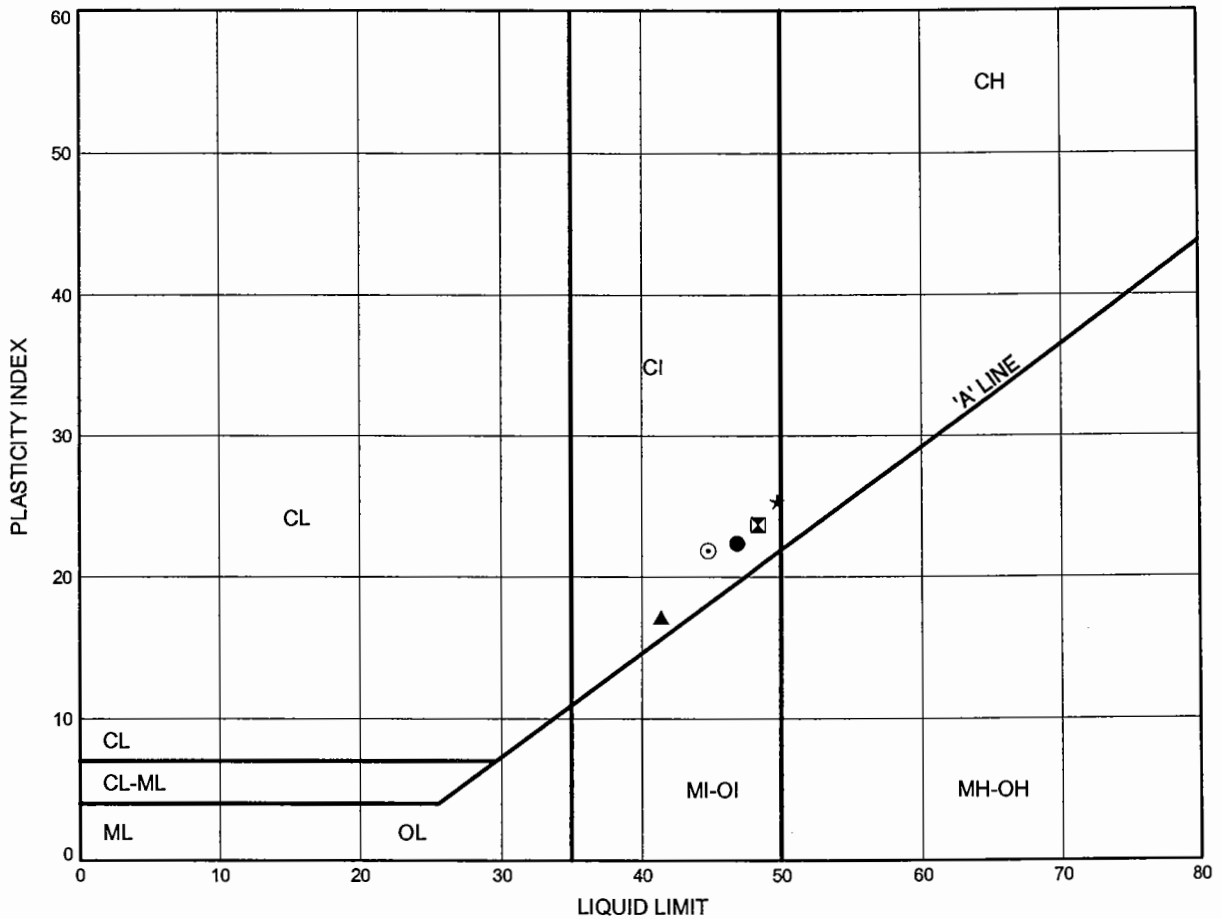
## SAND AND GRAVEL



# Hwy 11 Katrine ATTERBERG LIMITS TEST RESULTS

FIGURE B9

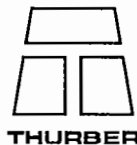
## SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	122N-10	10.90	288.53
⊠	122N-4	7.92	286.91
▲	122N-7	7.85	288.75
★	122N-8	13.94	285.02
⊙	122N-8	15.47	283.49

Date January 2005

Project 474-93-01



Prep'd HS

Chkd. AEG

## OEDOMETER CONSOLIDATION SUMMARY

### SAMPLE IDENTIFICATION

Project Number	04-1116-123	Sample Number	122N-8
Borehole Number	-	Sample Depth, m	12.2-12.9

### TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	12/19/2004		
Date Completed	01/03/2005		

### SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m <sup>3</sup>	17.33
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	11.95
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	60.65	Solids Height, cm	0.848
Water Content, %	45.05	Volume of Solids, cm <sup>3</sup>	26.87
Wet Mass, g	107.18	Volume of Voids, cm <sup>3</sup>	33.78
Dry Mass, g	73.89	Degree of Saturation, %	98.6

### TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.915	1.257	1.915				
4.85	1.912	1.254	1.914	98	7.92E-03	3.23E-04	2.51E-07
9.50	1.907	1.248	1.910	64	1.21E-02	5.61E-04	6.65E-07
19.59	1.898	1.237	1.903	124	6.19E-03	4.66E-04	2.82E-07
39.07	1.883	1.219	1.891	184	4.12E-03	4.02E-04	1.62E-07
77.88	1.859	1.191	1.871	80	9.28E-03	3.23E-04	2.94E-07
155.39	1.823	1.149	1.841	89	8.07E-03	2.43E-04	1.92E-07
311.43	1.758	1.072	1.791	103	6.60E-03	2.18E-04	1.41E-07
622.27	1.627	0.918	1.693	197	3.08E-03	2.20E-04	6.65E-08
1243.06	1.521	0.793	1.574	124	4.24E-03	8.92E-05	3.70E-08
2484.87	1.423	0.677	1.472	108	4.25E-03	4.12E-05	1.72E-08
1243.06	1.434	0.690	1.429				
311.43	1.467	0.729	1.451				
77.88	1.508	0.777	1.488				
19.59	1.560	0.839	1.534				
4.85	1.595	0.880	1.578				

Notes:

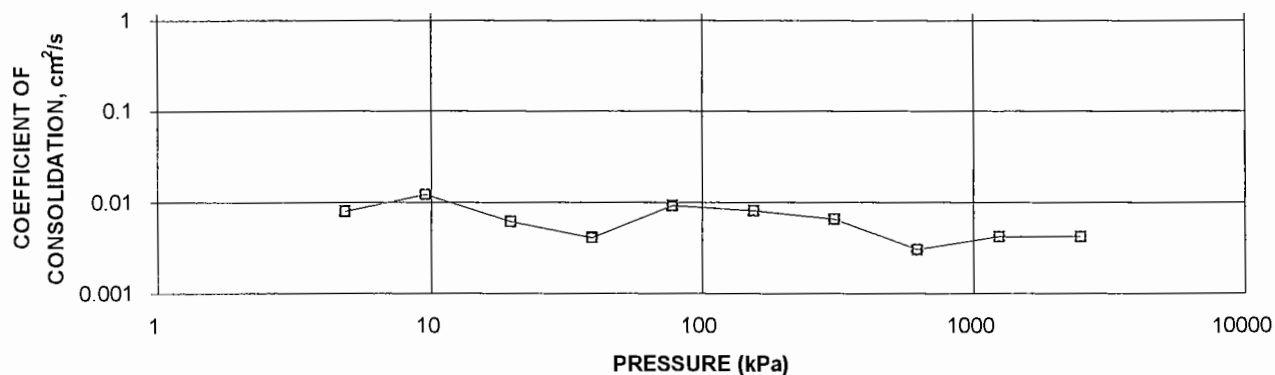
k calculated using cv based on t<sub>90</sub> values.

### SAMPLE DIMENSIONS AND PROPERTIES - FINAL

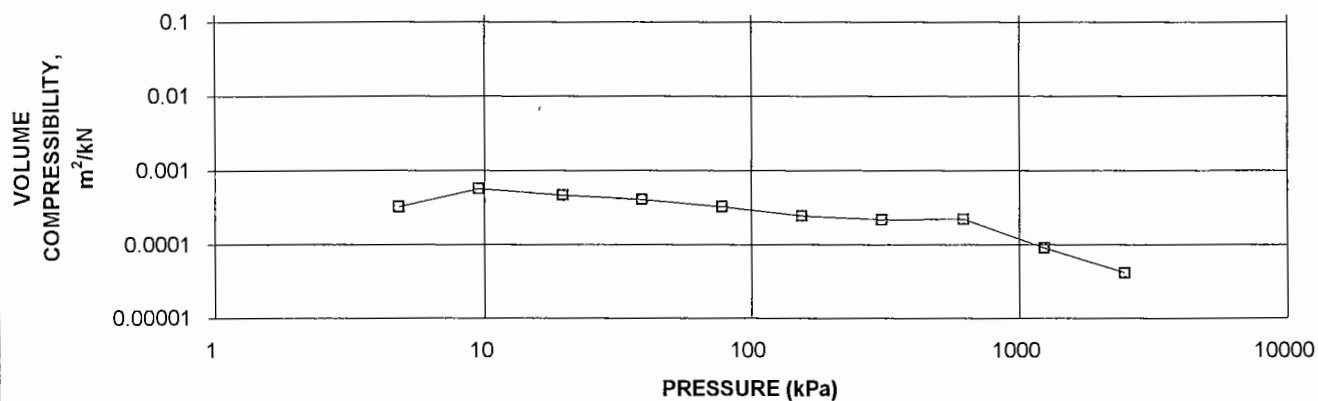
Sample Height, cm	1.60	Unit Weight, kN/m <sup>3</sup>	19.53
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.35
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	50.51	Solids Height, cm	0.848
Water Content, %	36.11	Volume of Solids, cm <sup>3</sup>	26.87
Wet Mass, g	100.57	Volume of Voids, cm <sup>3</sup>	23.64
Dry Mass, g	73.89		

# OEDOMETER CONSOLIDATION SUMMARY

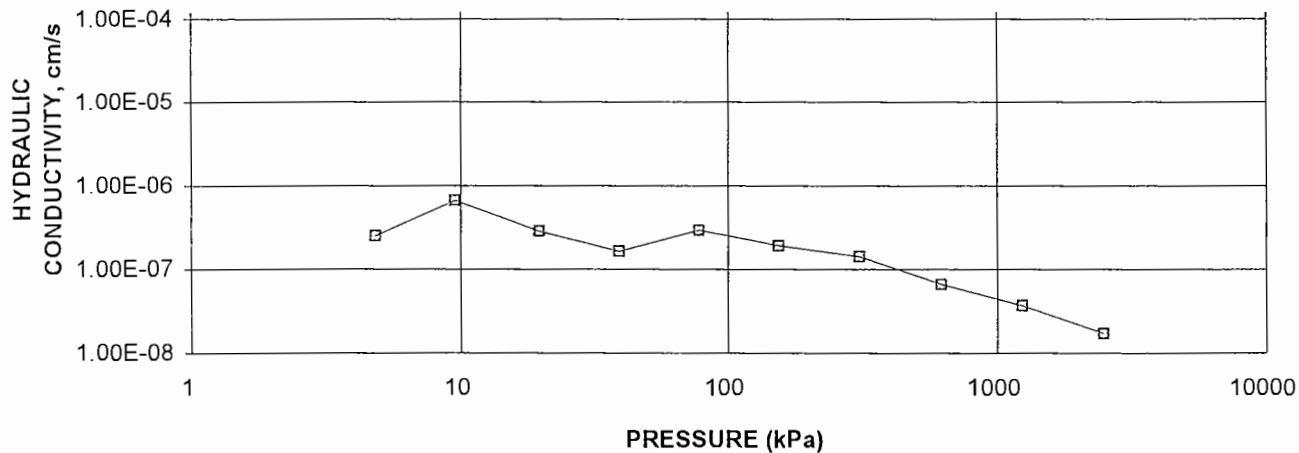
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
SA 122N-8



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
SA 122N-8



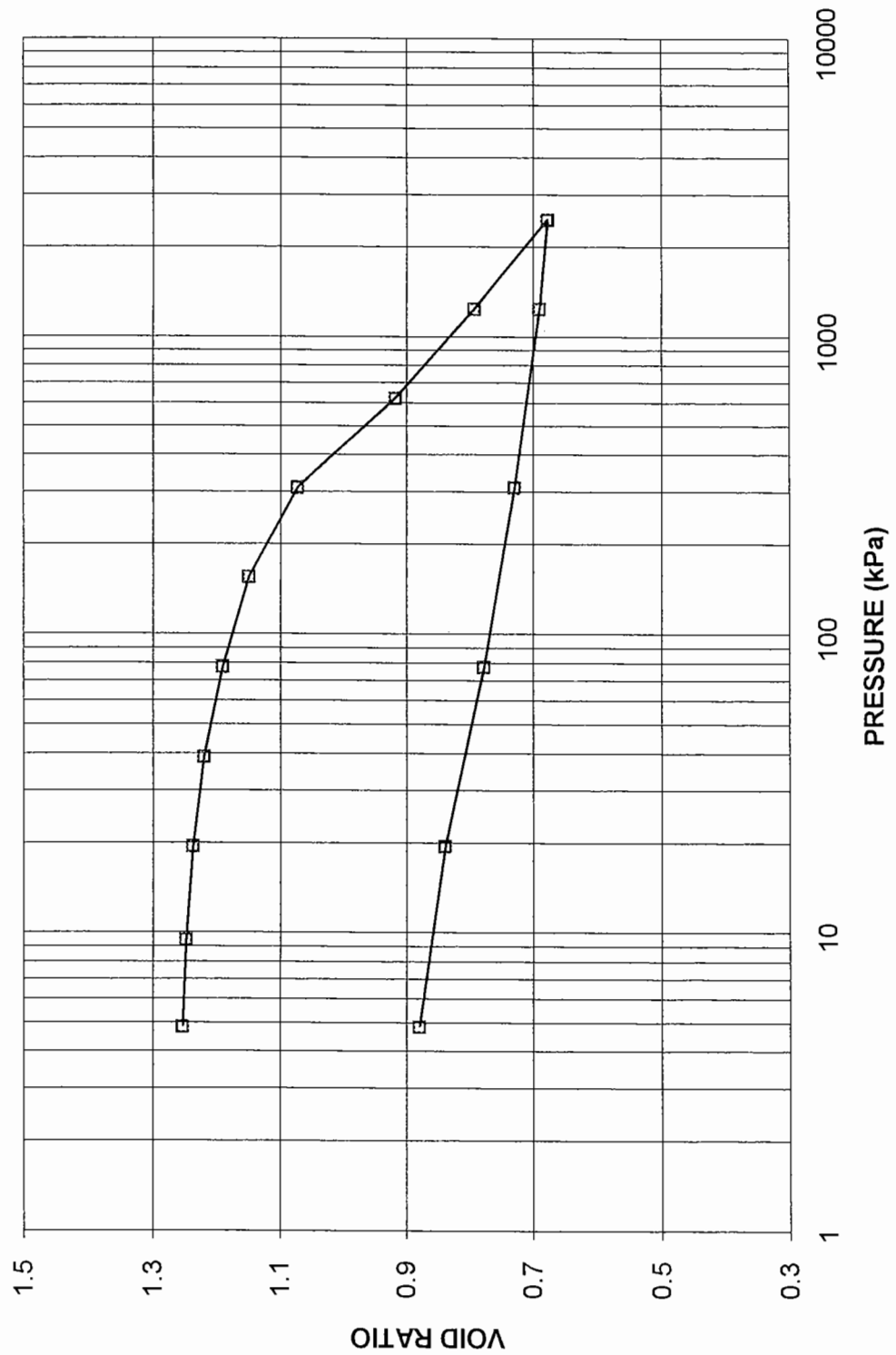
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
SA 122N-8



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE

CONSOLIDATION TEST  
VOID RATIO vs. PRESSURE  
SA 122N-8



## **Appendix C**

### **Data From Previous Investigation**



# RECORD OF BOREHOLE No M1

1 OF 4

METRIC

W.P. 314-99-00 LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 506.7; E 316 747.4 ORIGINATED BY G.I.  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem and Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.  
DATUM Geodetic DATE 27.03.01 to 30.03.01 CHECKED BY LSR

ELEV. DEPTH	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
295.2	Ground Surface													
0.0	150 mm Topsoil		1	SS	2		295							
	SILT some fine sand, very loose, brown, laminated with organic matter	moist --- wet	2	SS	2		294							0 35 (65)
293.9			3	SS	2		293							
1.3		brown ----- grey	4	SS	3		292							SST Augering
	FINE SAND trace silt with organic impurities (rootlets, wood fragments, etc.) laminated very loose, wet		5	SS	2		291							
			6	SS	3		290							
			7	SS	3		289							
			8	SS	1		288							0 93 7 0
			9	SS	3		287							
288.5			10	SS	6		286							
6.7	SILTY CLAY trace fine sand, laminated		11	TW	PH		285							0 10 75 (15)
	with silt layers grey wet	firm ----- very stiff	12	SS	17		284							0 10 81 9
285.1			13	SS	10		283							
10.1	SILT trace fine sand, compact to loose, grey wet		14	SS	9		282							0 17 (83)
282.0			15	SS	6*		281							*SS15 Low N-value probably due to hydrostatic uplift
13.2	SILTY FINE SAND compact, grey wet													
280.2														

15.0

Continued Next Page

+ 3 X 3

Numbers refer to Sensitivity

20  
15  
10

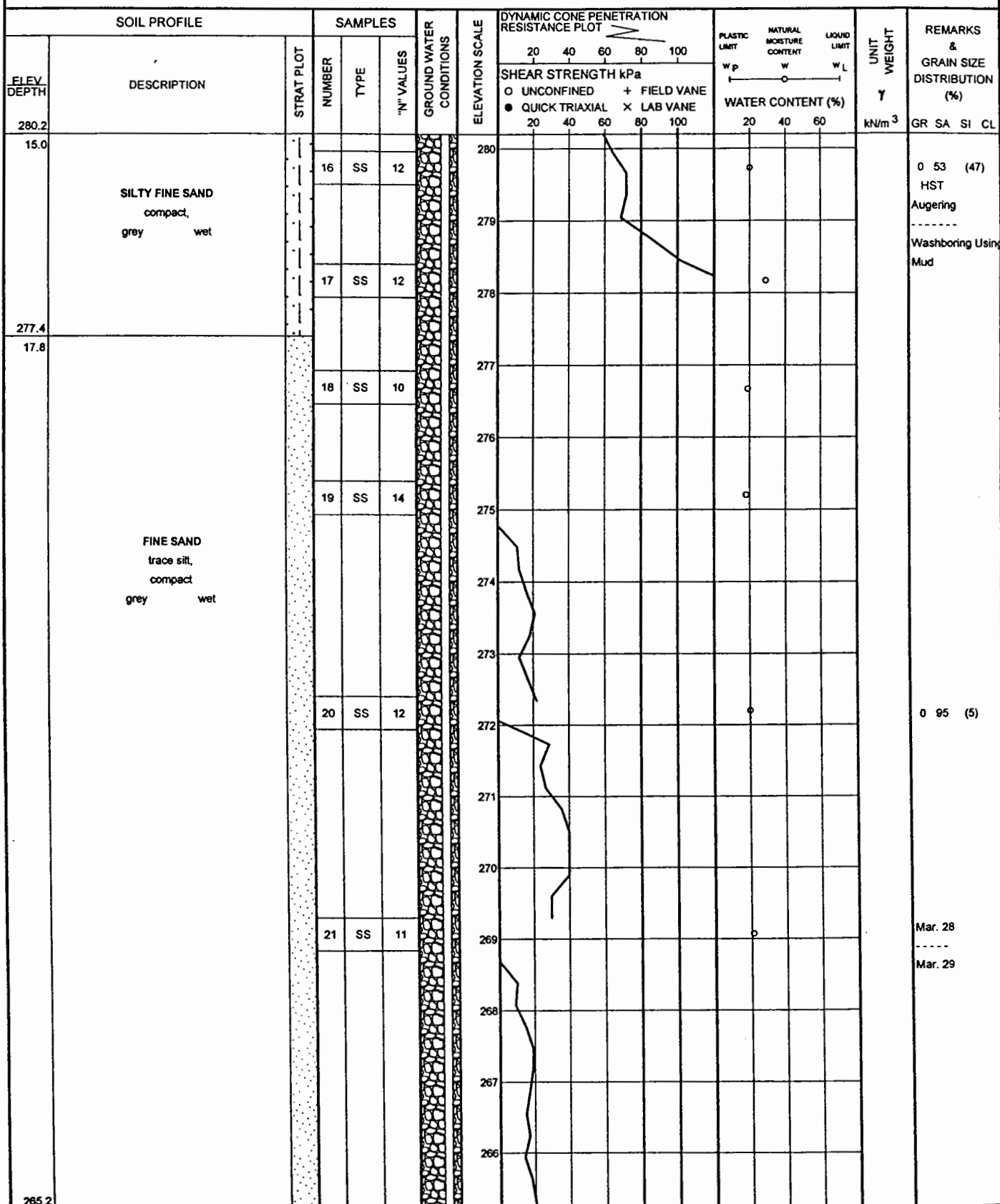
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No M1

2 OF 4

METRIC

W.P. 314-99-00 LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 506.7; E 316 747.4 ORIGINATED BY G.I.  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem and Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.  
DATUM Geodetic DATE 27.03.01 to 30.03.01 CHECKED BY LSR



30.0

Continued Next Page

+ 3 . x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

**METRIC**

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

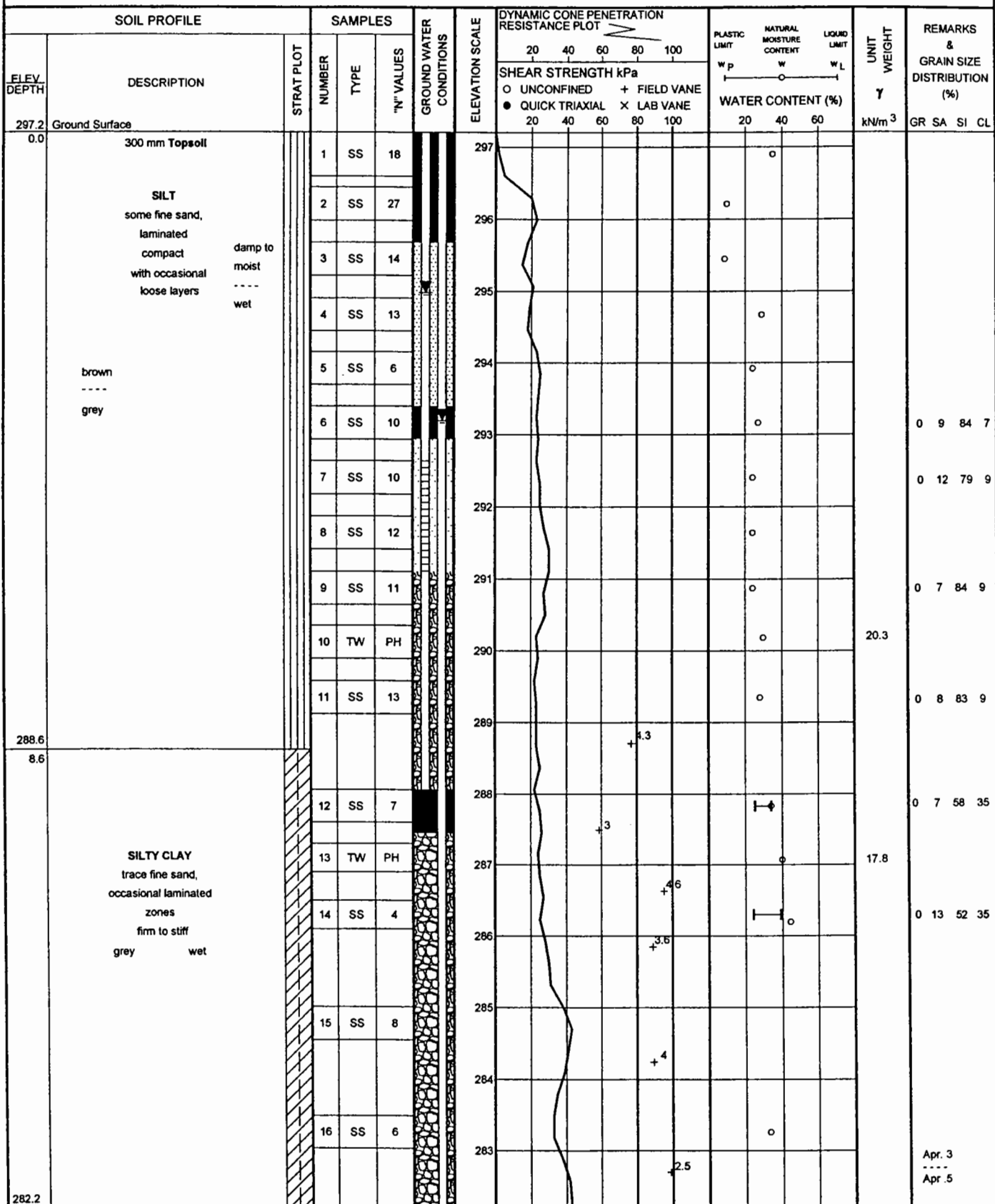
RECORD OF BOREHOLE No M1										4 OF 4		METRIC																					
W.P. 314-99-00		LOCATION Magnetawan River Bridge, NBL South Crossing Co-ords: N 5 047 506.7; E 316 747.4				ORIGINATED BY G.I																											
DIST 52 HWY 11		BOREHOLE TYPE Solid Stem and Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T.				COMPILED BY G.T																											
DATUM Geodetic		DATE 27.03.01 to 30.03.01				CHECKED BY LSR																											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL																		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE				W P	W			W L																	
	Piezometers installed on March 30/2001 Shallow piezometer to 6.1 m Deep piezometer to 39.6 m Ground water readings in piezometers:  <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>DATE</th> <th>SHALLOW</th> <th>DEEP</th> </tr> </thead> <tbody> <tr><td>02.04.01</td><td>1.1 m</td><td>0.65 m</td></tr> <tr><td>04.04.01</td><td>1.0 m</td><td>0.60 m</td></tr> <tr><td>06.04.01</td><td>1.1 m</td><td>0.60 m</td></tr> <tr><td>09.04.01</td><td>0.8 m</td><td>0.60 m</td></tr> <tr><td>11.04.01</td><td>0.65 m</td><td>0.50 m</td></tr> </tbody> </table> D.C.P.T. from 0 to 17.2 m depth performed on 27.03.01, at 2 m W from Borehole M1  D.C.P.T. performed between samples: SS 19 and SS 20 (20.4 to 22.9 m) SS 20 and SS 21 (23.2 to 25.9 m) SS 21 and SS 22 (26.5 to 30.5 m) SS 22 and SS 23 (31.1 to 35.1 m) SS 23 and SS 24 (35.7 to 39.6 m) SS 24 and SS 25 (40.2 to 41.4 m) SS 25 and SS 26 (41.1 to 41.9 m) SS 26 and SS 27 (42.8 to 43.4 m) Stratigraphy inferred only D.C.P.T. performed after SS 27 (44.5 to 44.7 m)	DATE	SHALLOW	DEEP	02.04.01	1.1 m	0.65 m	04.04.01	1.0 m	0.60 m	06.04.01	1.1 m	0.60 m	09.04.01	0.8 m	0.60 m	11.04.01	0.65 m	0.50 m														
DATE	SHALLOW	DEEP																															
02.04.01	1.1 m	0.65 m																															
04.04.01	1.0 m	0.60 m																															
06.04.01	1.1 m	0.60 m																															
09.04.01	0.8 m	0.60 m																															
11.04.01	0.65 m	0.50 m																															

# RECORD OF BOREHOLE No M2

1 OF 3

METRIC

W.P. 314-99-00 LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 563.5; E 316 703.7 ORIGINATED BY G.I.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.  
DATUM Geodetic DATE 03.04.01 to 09.04.01 CHECKED BY Z.O.



15.0

Continued Next Page

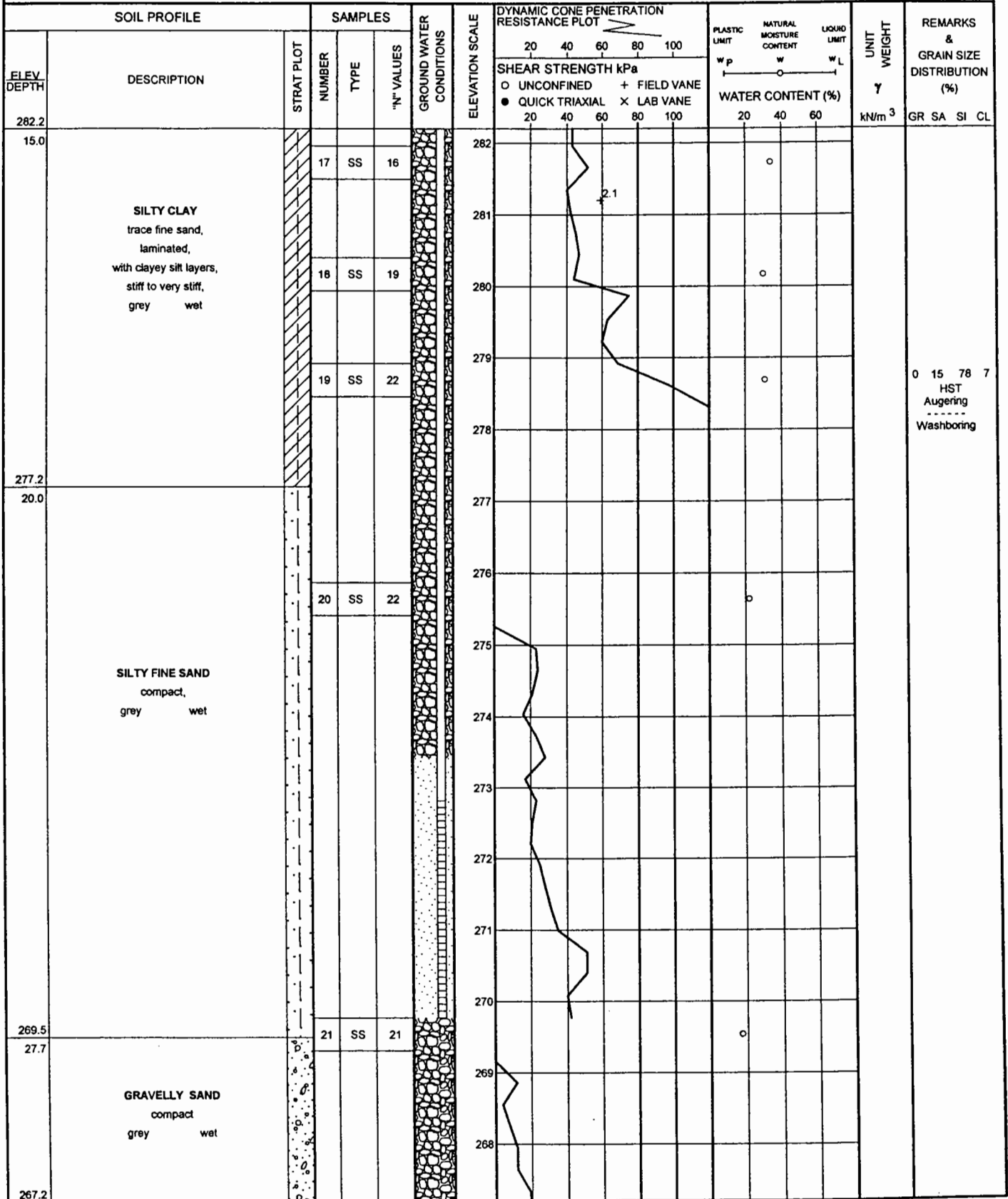
+ 3, x 3: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M2

2 OF 3

METRIC

W.P. 314-99-00 LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 563.5; E 316 703.7 ORIGINATED BY G.I  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T  
DATUM Geodetic DATE 03.04.01 to 09.04.01 CHECKED BY Z.O



30.0

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No M2										3 OF 3	METRIC							
W.P. 314-99-00		LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 563.5; E 316 703.7				ORIGINATED BY G.I												
DIST 52 HWY 11		BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T.				COMPILED BY G.T												
DATUM Geodetic		DATE 03.04.01 to 06.04.01				CHECKED BY Z.O												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	20
267.2																		
30.0	GRAVELLY SAND																	
266.4	compact																	
	grey wet																	
30.8	SANDY GRAVEL		22	NQ	-													
	with BOULDERS		23	SS	FOUL													
	cobbles and boulders																	
	very dense,																	
264.9	grey wet		24	NQ	-													
32.3	End of borehole																	
264.3																		
32.9	End of Dynamic Cone Penetration Test																	
	Piezometers installed on April 6, 2001																	
	Shallow piezometer to 6.1 m																	
	Deep piezometer to 27.4 m																	
	Ground water readings in piezometers:																	
	DATE SHALLOW DEEP																	
	11.04.01 2.2 m 3.9 m																	
	D.C.P.T. from 0 to 19.1 m depth performed on 03.04.01, at 2 m E of Borehole M2																	
	D.C.P.T. performed between samples SS20 and SS21 (21.9 m to 27.4 m) stratigraphy inferred only																	

RECORD OF BOREHOLE No M3										1 OF 1	METRIC		
W.P. 314-99-00		LOCATION Magnetawan River Bridge NBL, South Crossing Co-ords: N 5 047 486.8; E 316 753.0				ORIGINATED BY G.I							
DIST 52 HWY 11		BOREHOLE TYPE Hollow Stem Augering				COMPILED BY G.T							
DATUM Geodetic		DATE 02.04.01				CHECKED BY LSR							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					
294.9	Ground Surface												
0.0	80 mm Topsoil		1	SS	2								
	brown												
	grey		2	SS	1								0 52 48 0
	SILTY FINE SAND												
	very loose to loose		3	SS	6								
	wet												
	traces of organic matter		4	SS	7								
			5	SS	5								0 52 (48)
291.2													
3.7	SILTY CLAY		6	SS	5								0 13 67 20
	trace fine sand,												
	laminated		7	TW	PH								
	firm to stiff												
289.7	grey wet												
5.2			8	SS	8								0 9 84 7
	SILT												
	some fine sand,		9	SS	6								0 16 77 7
	laminated,												
	loose												
	grey wet		10	SS	3*								0 9 88 3
286.3													* SS10 & 11
8.6	SILTY FINE SAND												Low N-value
	loose, grey, wet		11	SS	1*								probably due to
285.3													hydrostatic
9.6	End of borehole												uplift
	Ground water not stabilized												
	on completion of boring.												
	**Ground water level estimated												
	from moisture condition of soil												
	samples												



RECORD OF BOREHOLE No M4

1 OF 1

METRIC

W.P. 314-99-00 LOCATION Magnetawan River Bridge NBL, South Crossing, Co-ords: N 5 047 582.5; E 316 698.6 ORIGINATED BY G.I.  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY G.T.  
DATUM Geodetic DATE 09.04.01 & 10.04.01 CHECKED BY LSR

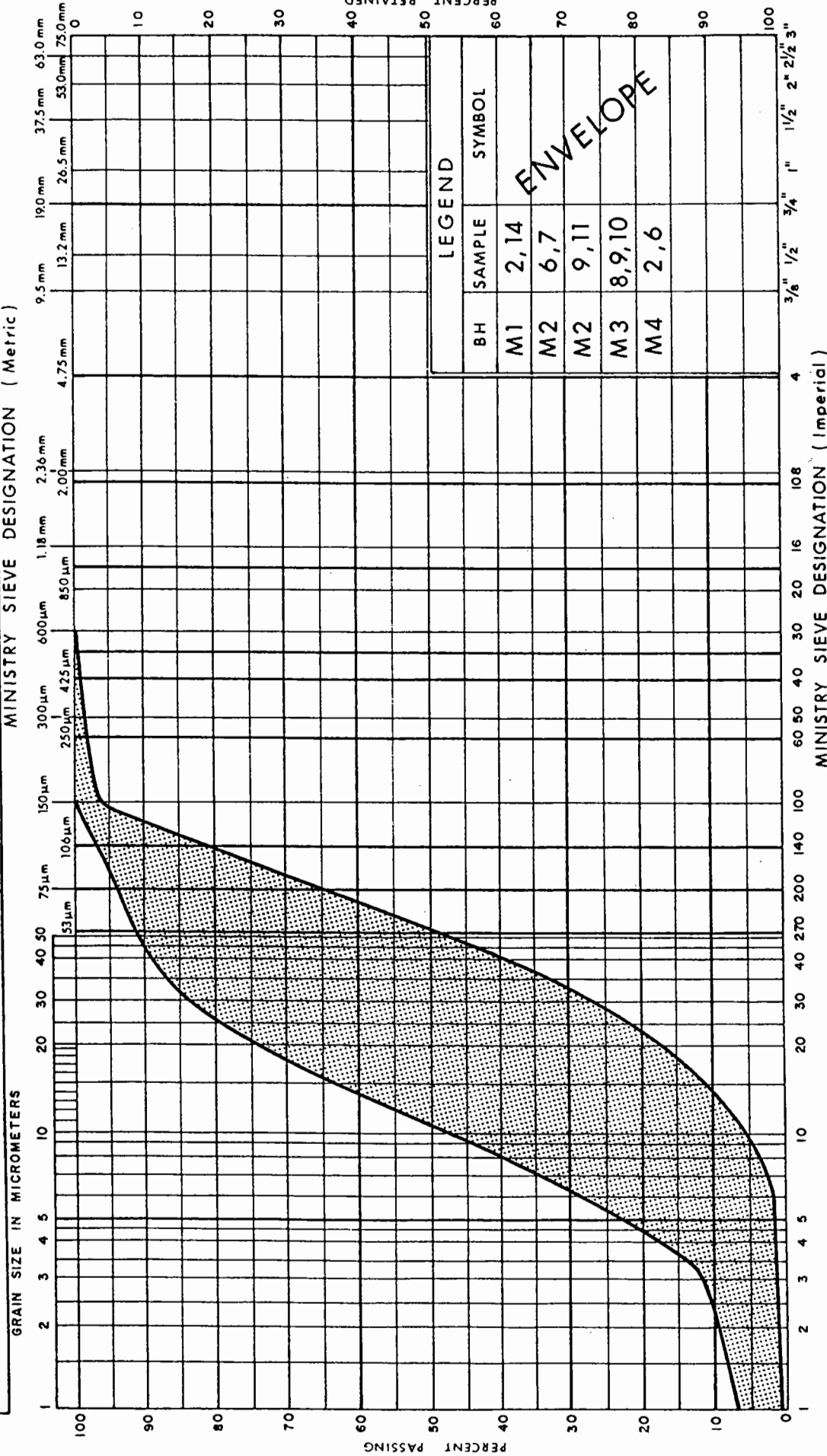
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
299.6	Ground Surface							20 40 60 80 100				
0.0	300 mm Topsoil		1	SS	3		299	○ UNCONFINED + FIELD VANE				
	very loose damp							● QUICK TRIAXIAL X LAB VANE				
	-----							20 40 60 80 100				
	SILT		2	SS	15		298	WATER CONTENT (%)				
	some fine sand, laminated, with fine sand layers							W <sub>p</sub> W W <sub>L</sub>				
	wet							20 40 60				
	compact							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				
	-----							○				
	loose to very loose		3	SS	6		297					
	brown											
	-----											
	grey		4	SS	4		296					
			5	SS	6		295					
							294					
			6	SS	14		293					
							292					
			7	SS	15		291					
							290					
289.6												
10.0												
	SILTY CLAY		8	SS	9		289					
	trace fine sand, with silt layers											
	grey wet											
	stiff											
	-----											
	firm		9	SS	5		288					
			10	TW	PH		287					
			11	SS	5		286					
285.1												
14.5	End of borehole											

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

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS



Ministry of  
Transportation



## GRAIN SIZE DISTRIBUTION

SILT, SOME FINE SAND

FIG No 1

W P 314-99-00

SPT 1010A1

# UNIFIED SOIL CLASSIFICATION SYSTEM

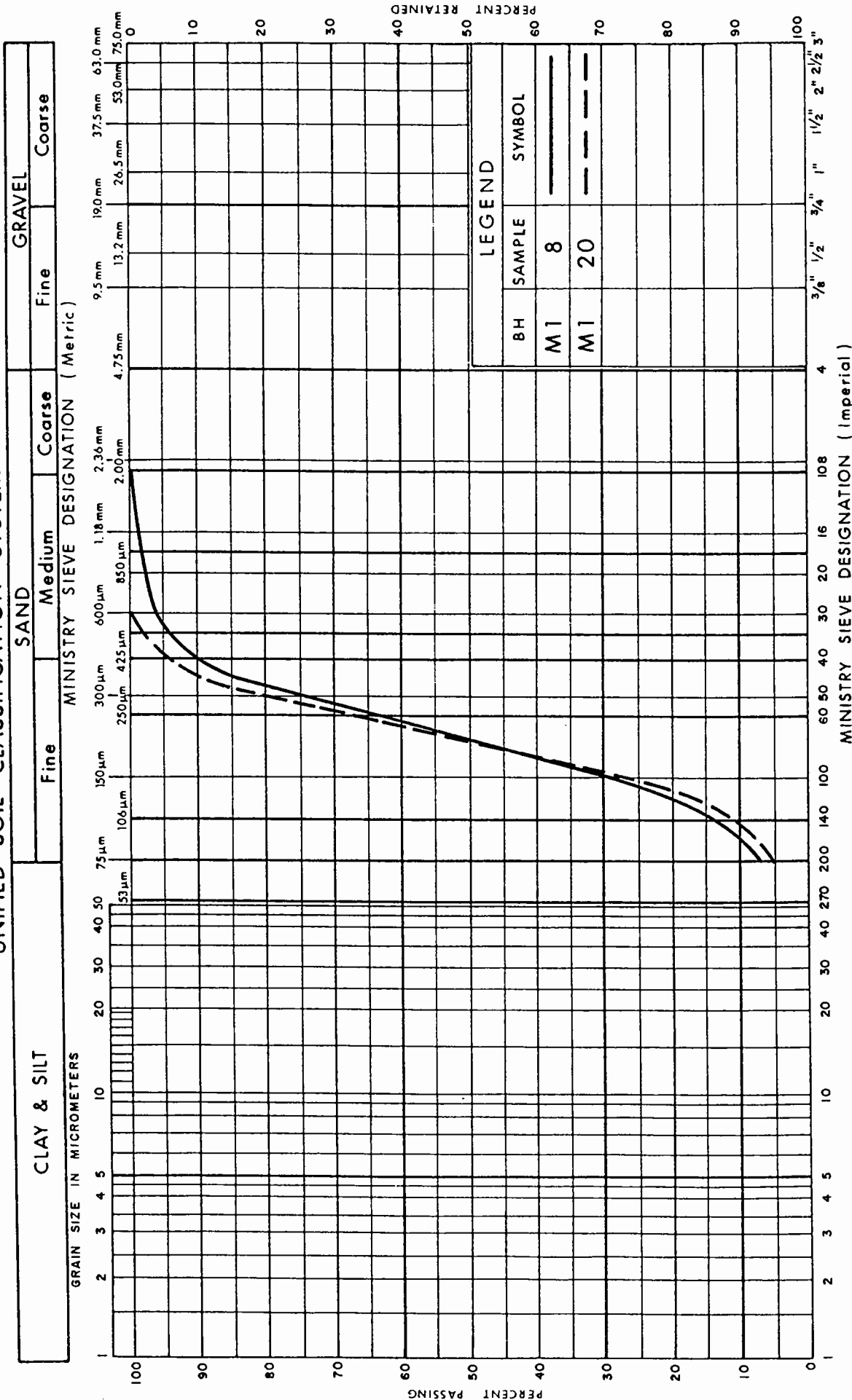


FIG No 2

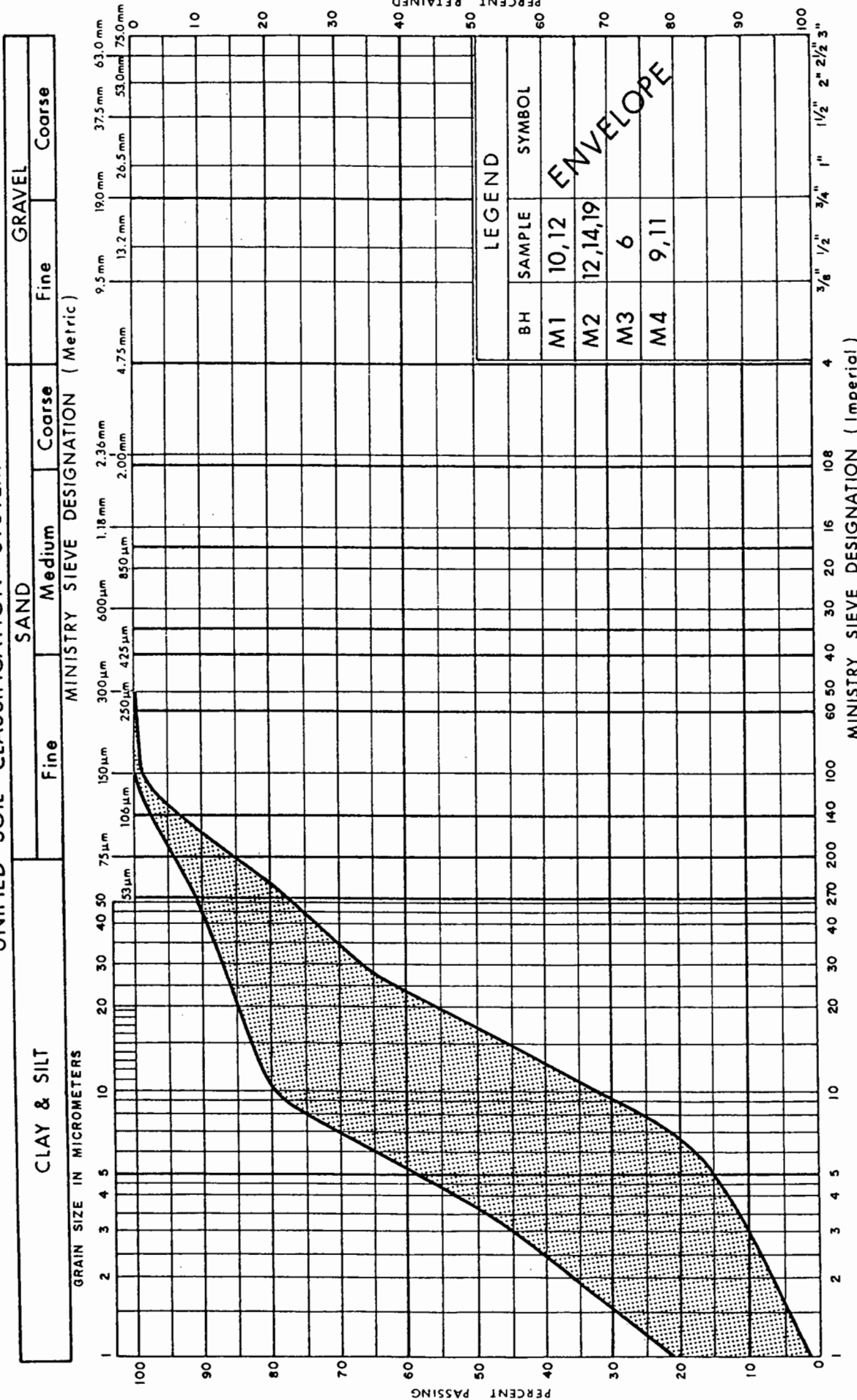
W P 314-99-00

SPT 1010A1

GRAIN SIZE DISTRIBUTION

FINE SAND, TRACE SILT

# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION SILTY CLAY, TRACE FINE SAND

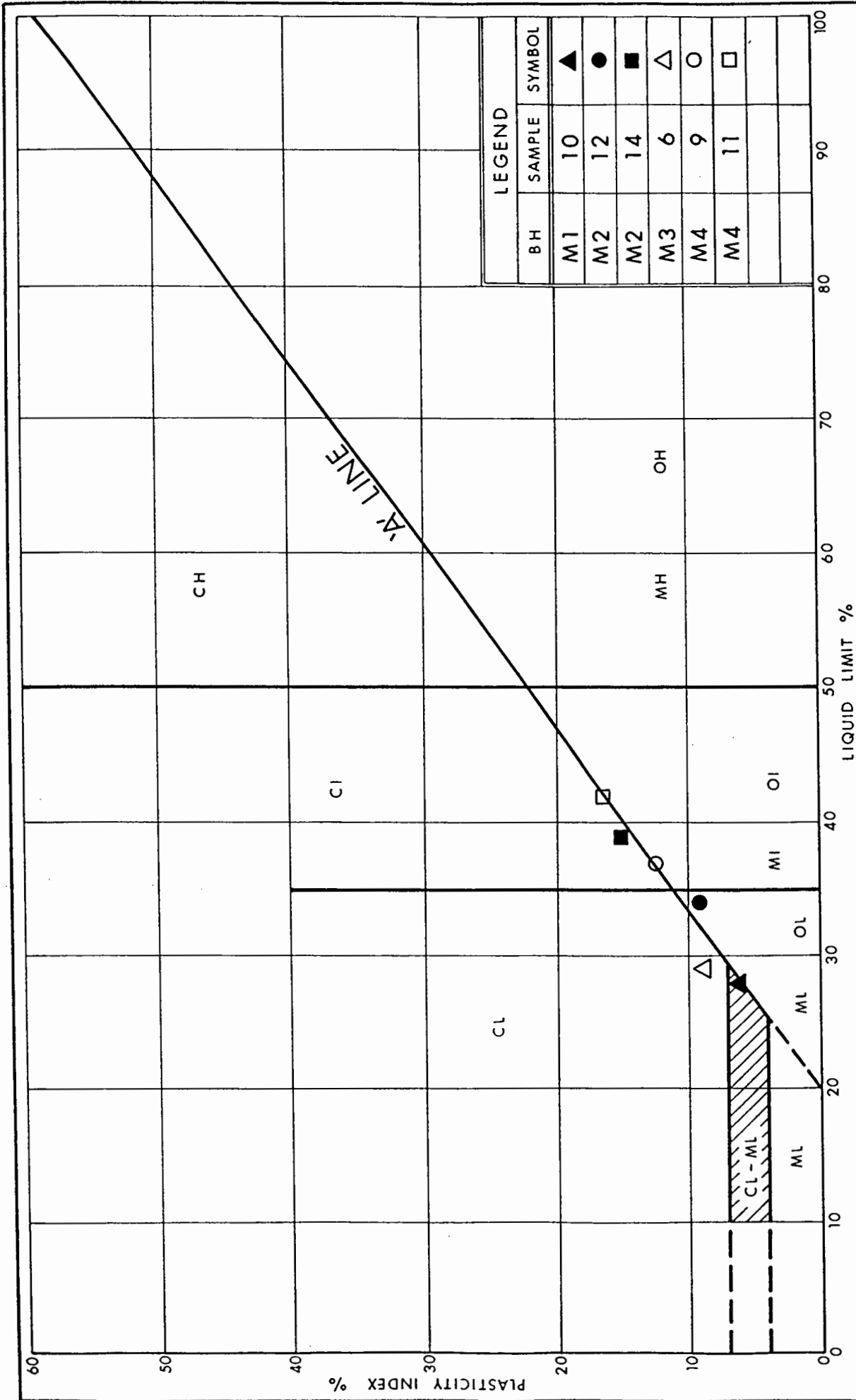
FIG No 3

W P 314-99-00

SPT 1010AI

Ministry of  
Transportation





# PLASTICITY CHART

SILTY CLAY, TRACE FINE SAND

FIG No 4

W P 314 -99-00

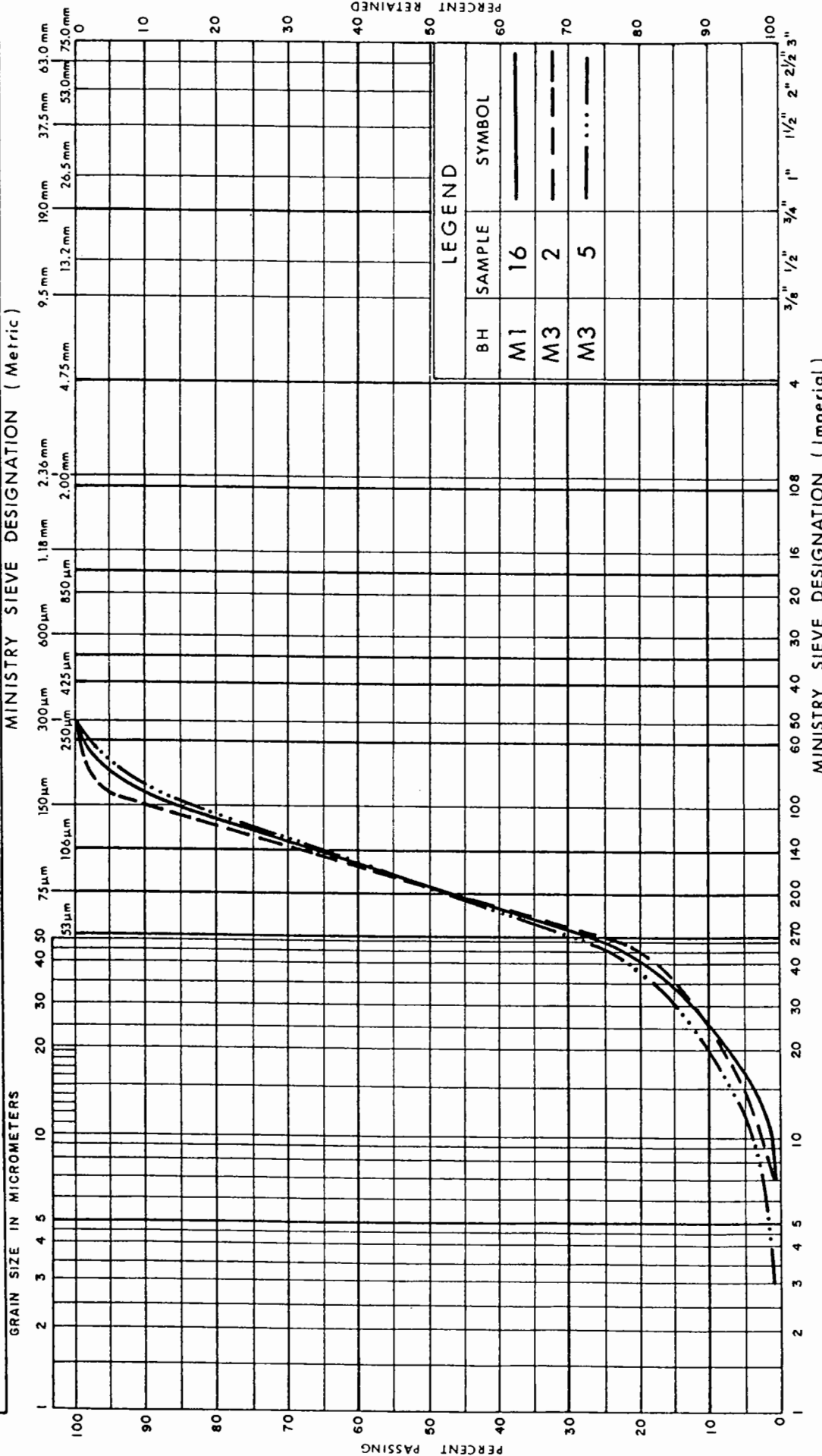
SPT 1010A1

Ministry of  
Transportation



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse
		MINISTRY SIEVE DESIGNATION (Metric)					



## GRAIN SIZE DISTRIBUTION SILTY FINE SAND

FIG No 5

W P 314-99-00

SPT 1010A1

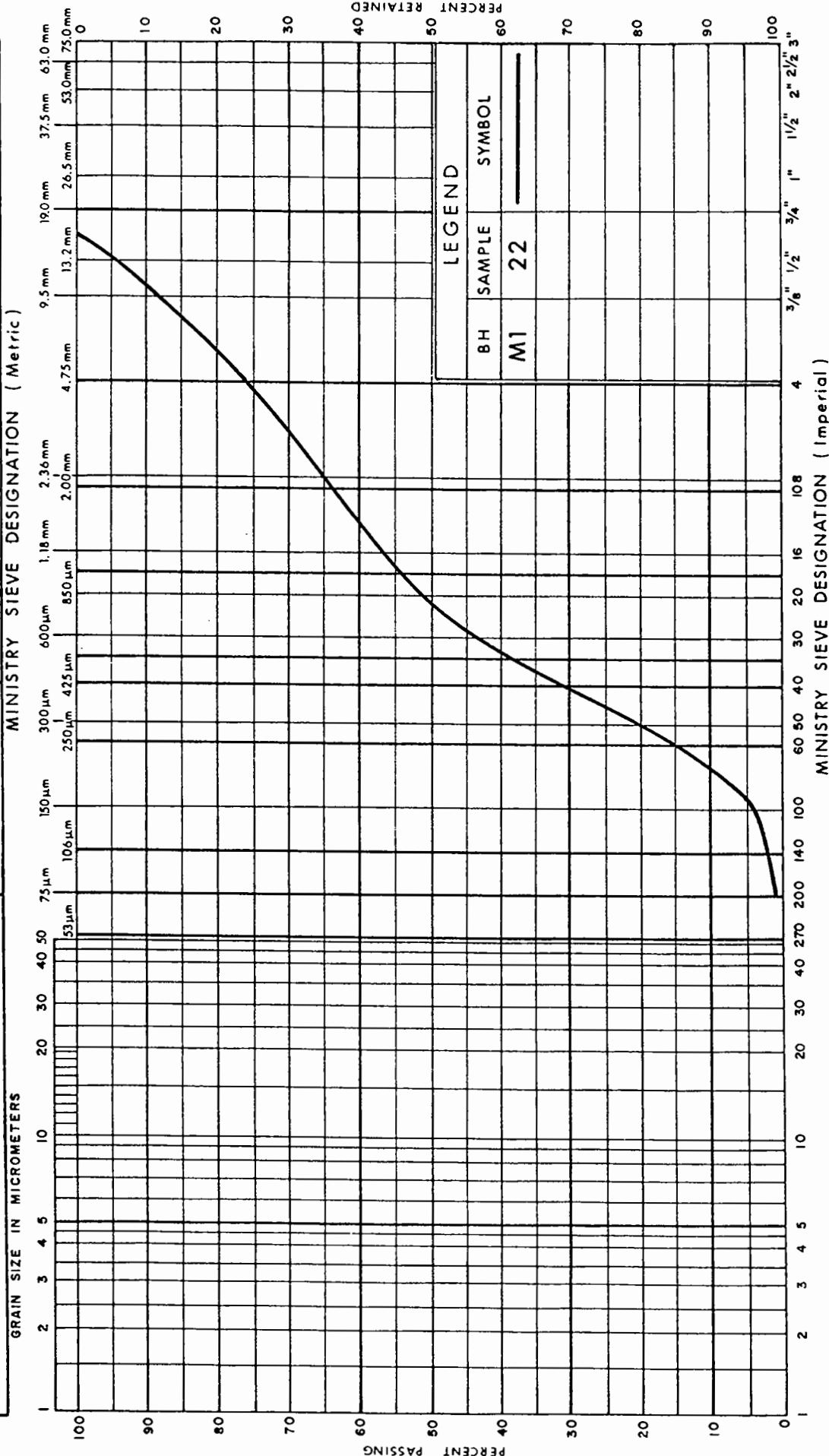
Ministry of  
Transportation



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

MINISTRY SIEVE DESIGNATION (Metric)



## GRAIN SIZE DISTRIBUTION GRAVELLY SAND

FIG No 6

W P 314-99-00

SPT 1010A1

Ministry of  
Transportation



## **Appendix D**

### **Foundation Comparison**



**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

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

DRAFT

## **Appendix E**

### **Special Provisions**

The following Special provisions are referenced in this report:

- Amendment to OPSS 206, December 1993
- 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 F**

### **Selected Slope Stability Output**

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL South Approach  
 Rock Fill 1.25:1 Base

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Rock Fill	20	0	42
Silt & Sand	21	0	30
Gravel	22	0	32

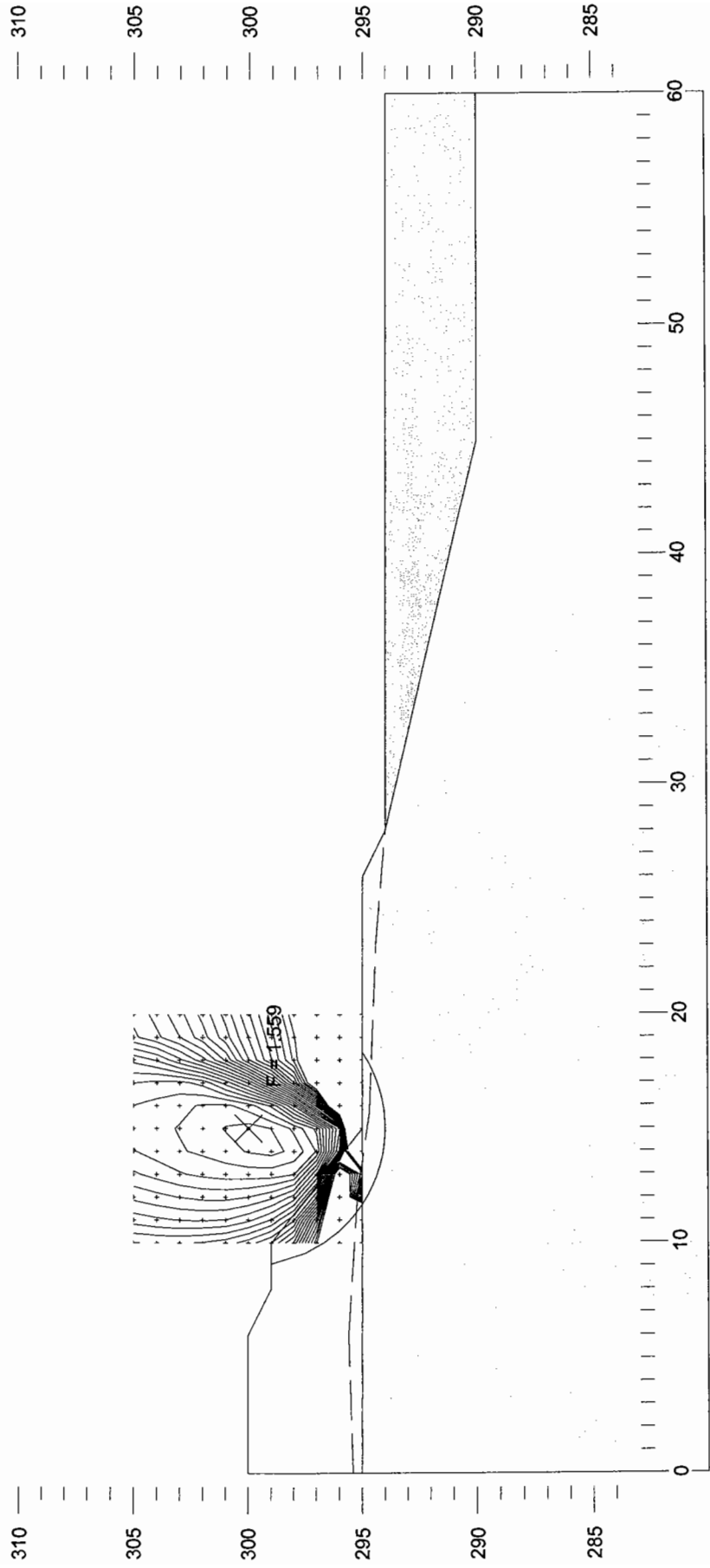


Figure F1

Thurber Engineering Ltd. - Toronto  
19-1423-16  
Hwy 11, Katrine  
January 10, 2005  
Magnetawan South NBL South Approach  
Rock Fill 1.25:1 Flood and Seismic

	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	deg	Surf.
Water	10	0	1
Rock Fill	20	42	1
Silt & Sand	21	30	1
Gravel	22	32	1

Seismic coefficient = 0.08

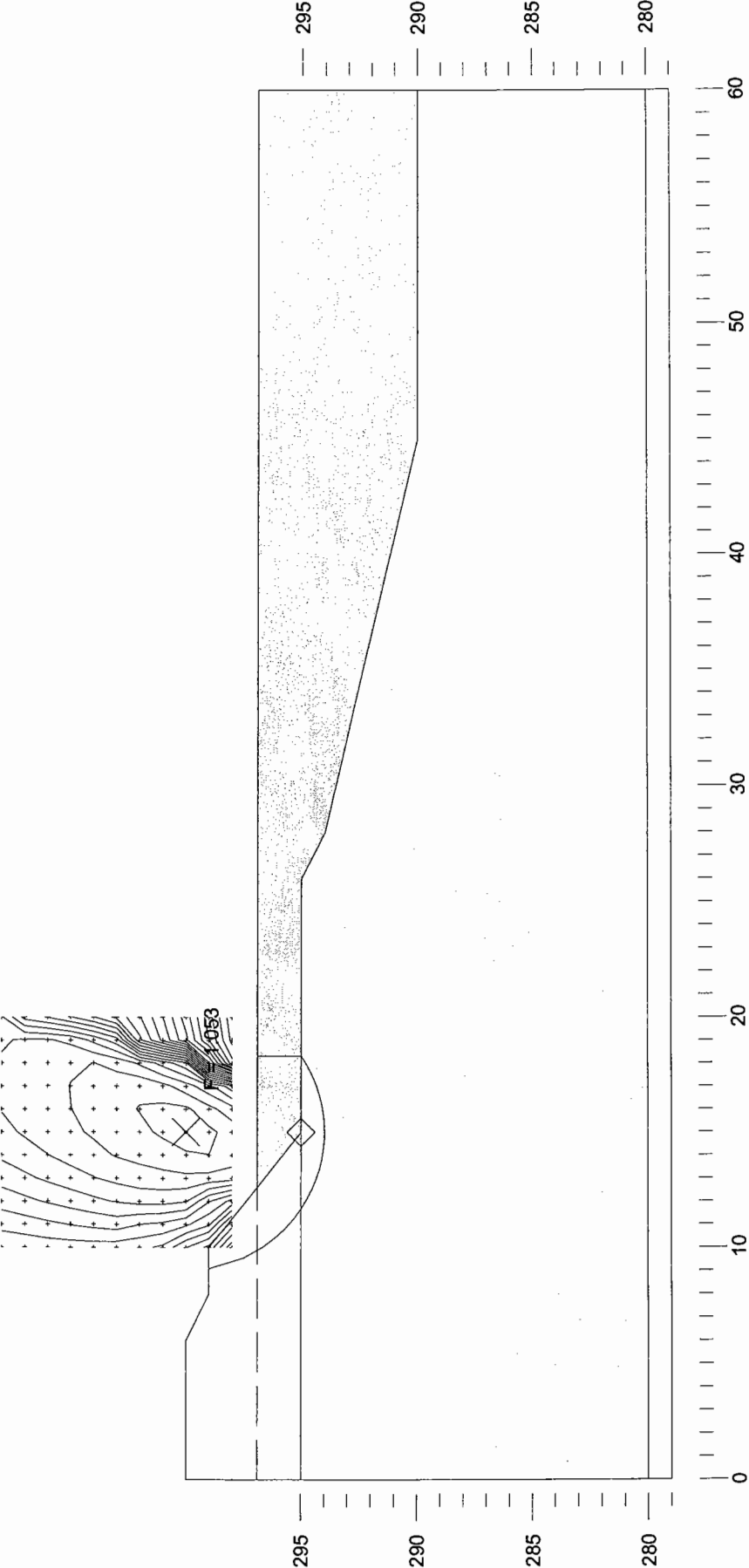


Figure F2

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL South Approach  
 Earth Fill 2:1 Base

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Earth Fill	22	0	1
Silt & Sand	21	0	1
Gravel	22	0	1

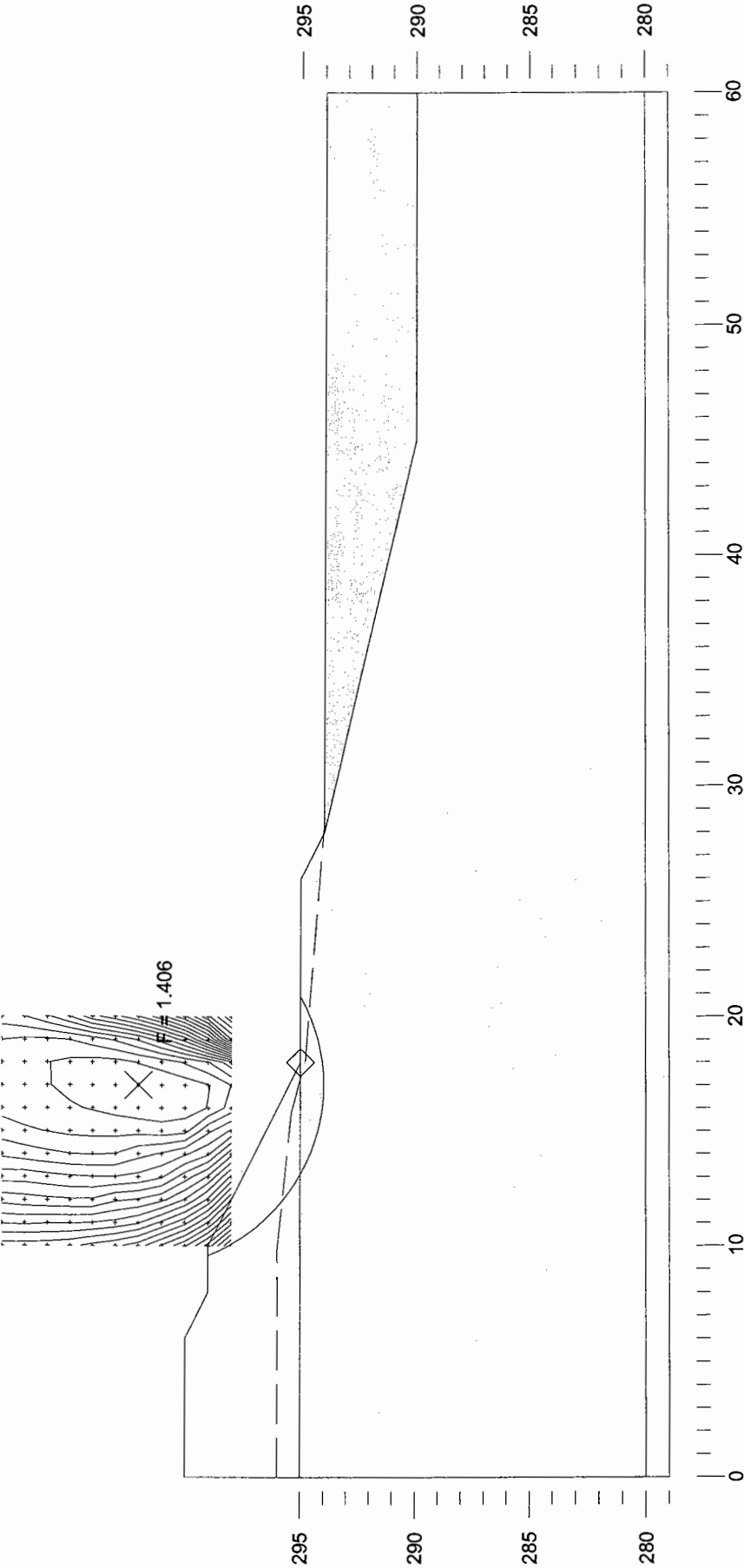


Figure F3

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL South Approach  
 Earth Fill 2:1 Flood and Seismic

	Gamma	C	Phi	Piezo
	kN/m <sup>3</sup>	kPa	deg	Surf.
Water	10	0	0	1
Earth Fill	22	0	30	1
Silt & Sand	21	0	30	1
Gravel	22	0	32	1

Seismic coefficient = 0.08

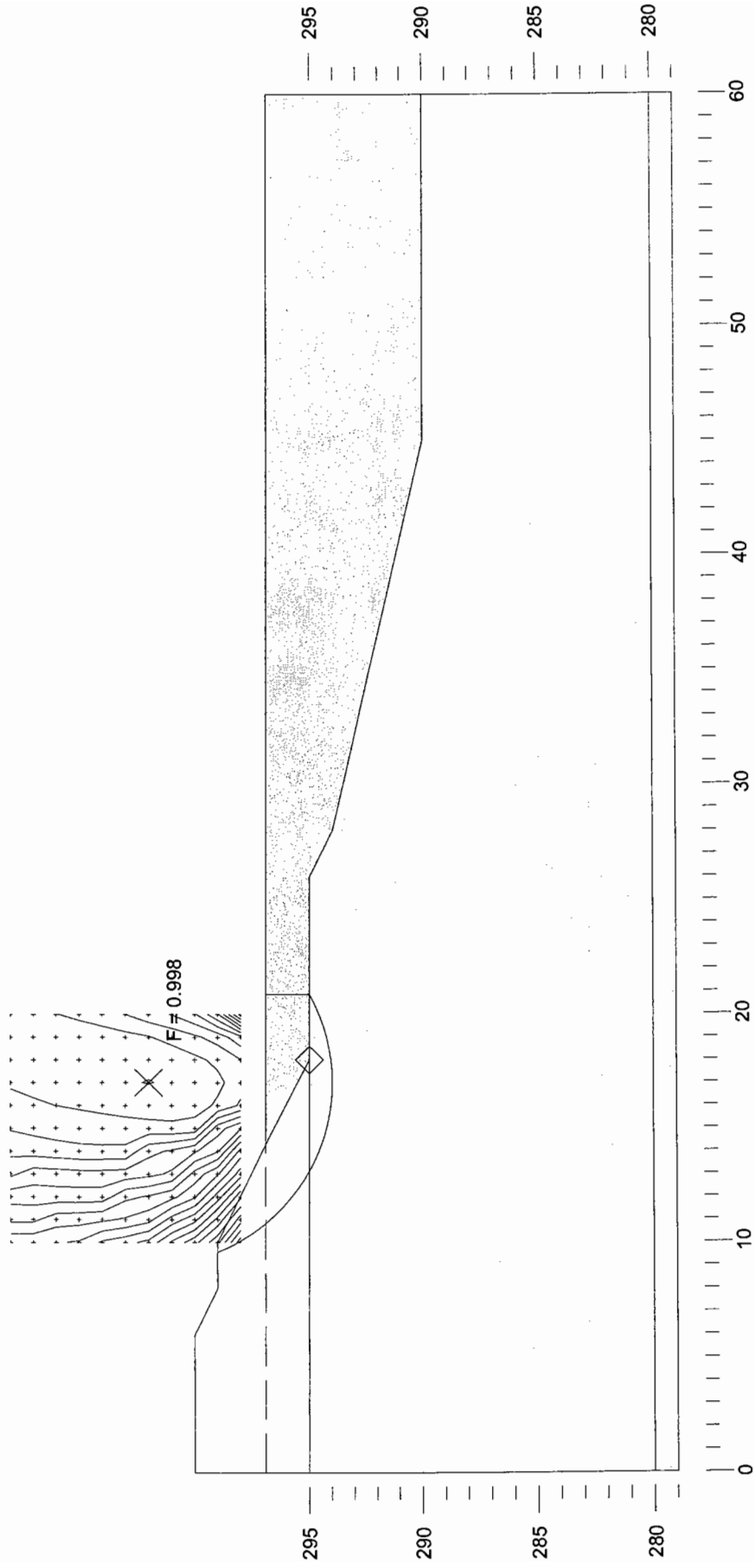


Figure F4



Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL North Approach  
 Earth Fill 2:1 Base

	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	deg	Surf.
Water	10	0	1
Rock fill	20	0	1
Sand and Silt	21	0	1
Clay	20	0	1
Gravel	22	0	1

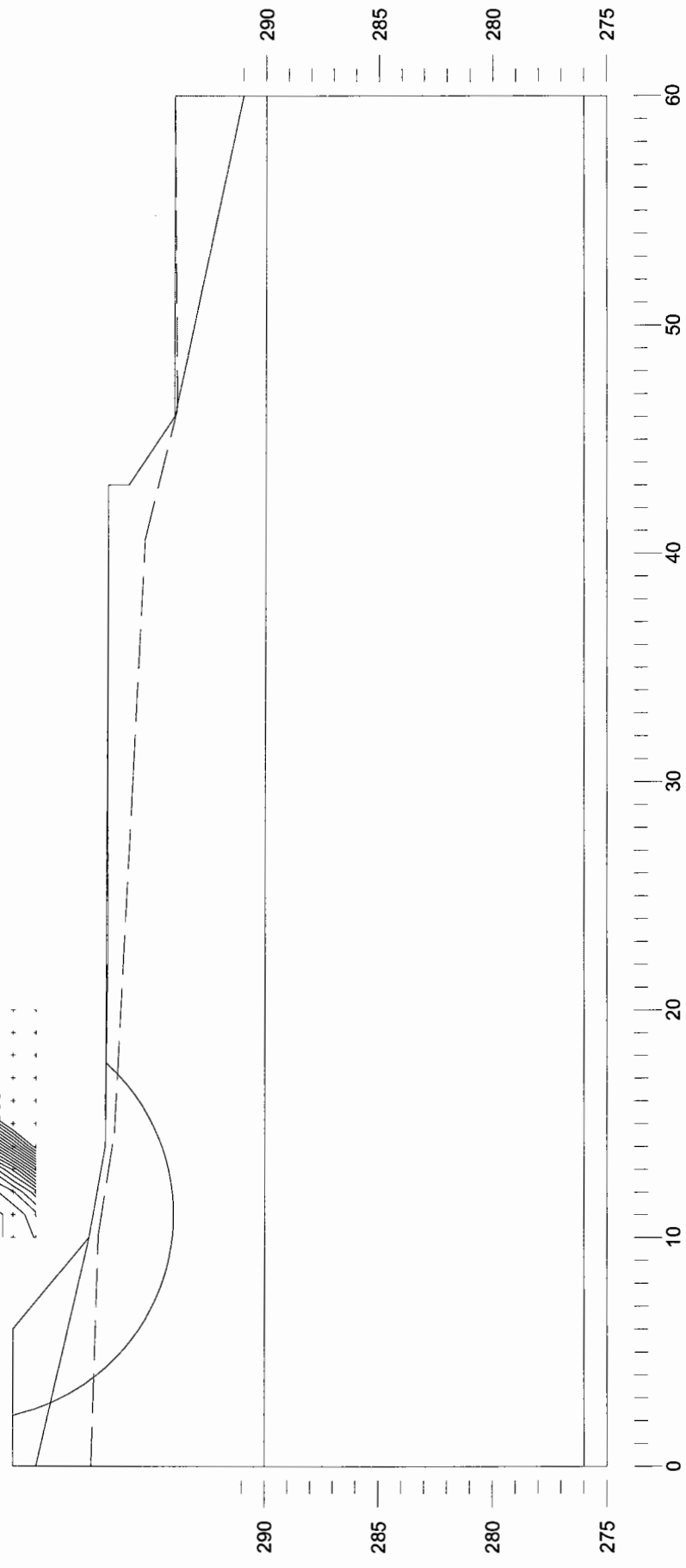
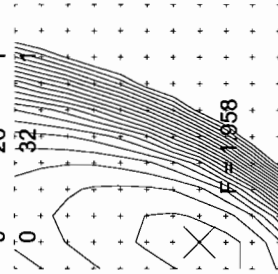


Figure F5

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL North Approach  
 Earth Fill 2:1 Flood and Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Rock fill	20	0	1
Sand and Silt	21	0	1
Clay	20	0	1
Gravel	22	0	1
Seismic coefficient = 0.08			

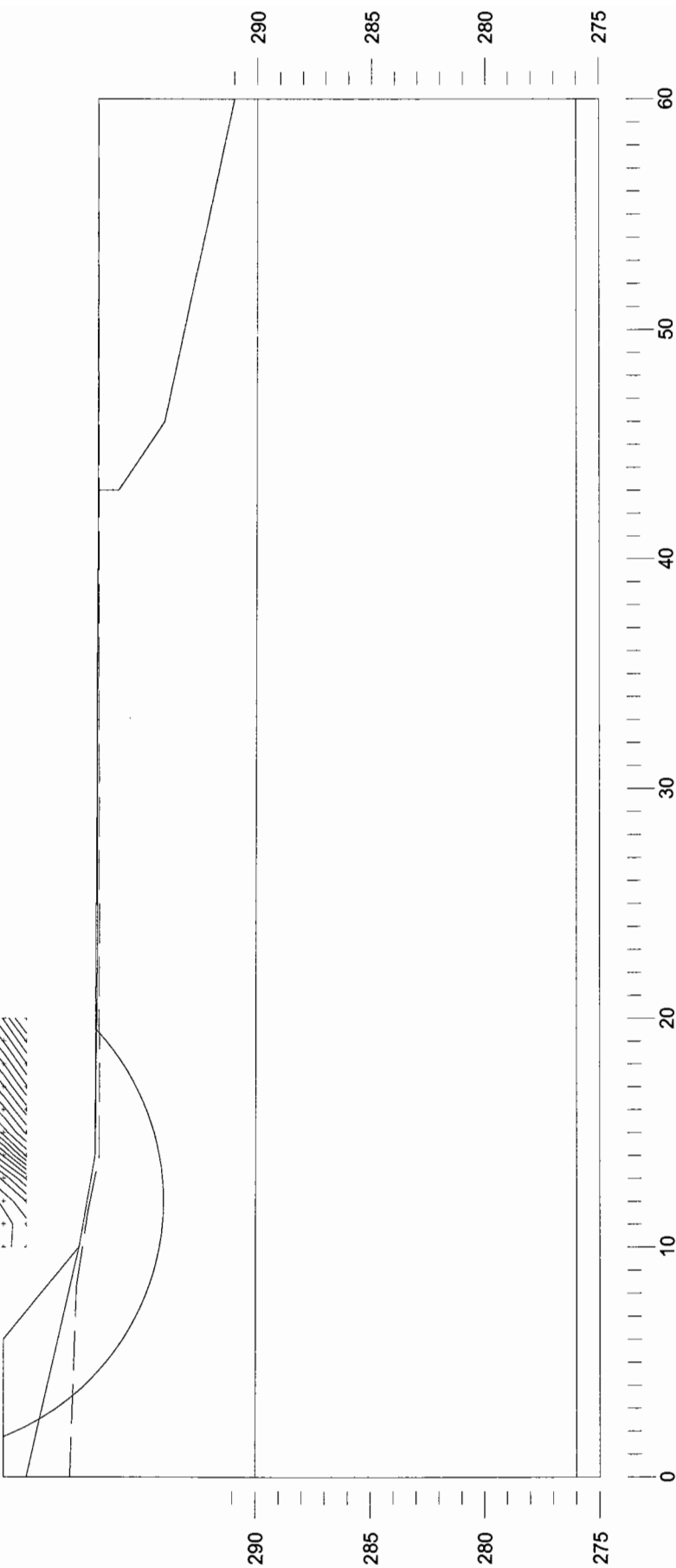
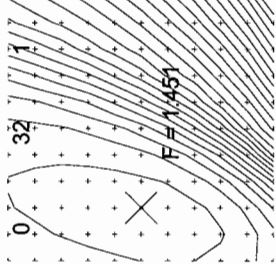


Figure F6

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL North Approach  
 Earth Fill 2:1 Base

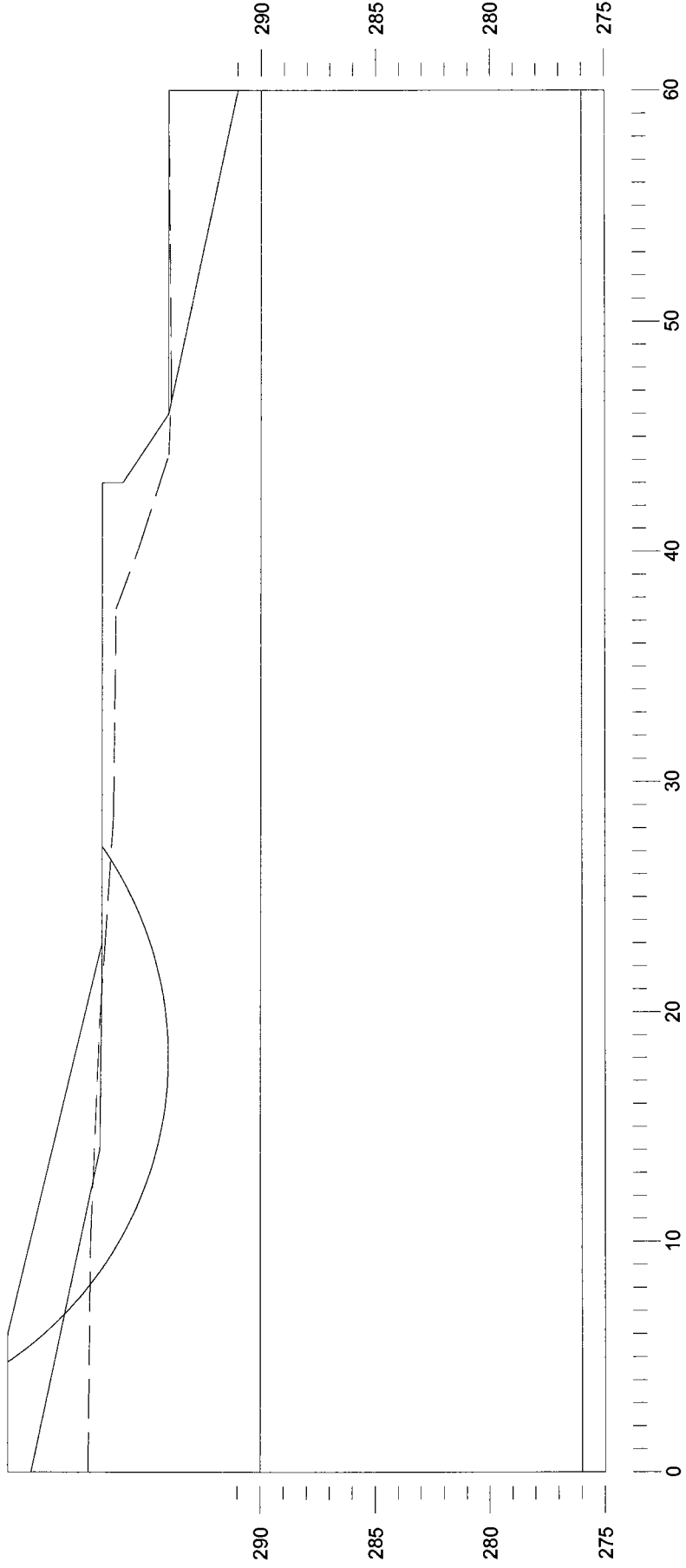
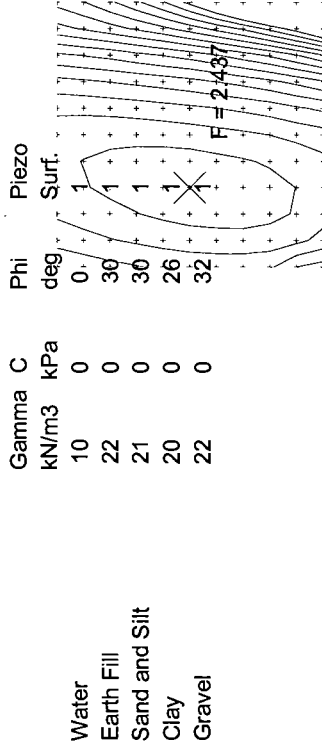


Figure F7

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 January 10, 2005  
 Magnetawan South NBL North Approach  
 Earth Fill 2:1 Flood and Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	10	0	1
Earth Fill	22	30	1
Sand and Silt	21	30	1
Clay	20	26	1
Gravel	22	32	1

Seismic coefficient = 0.08

$F = 1.778$

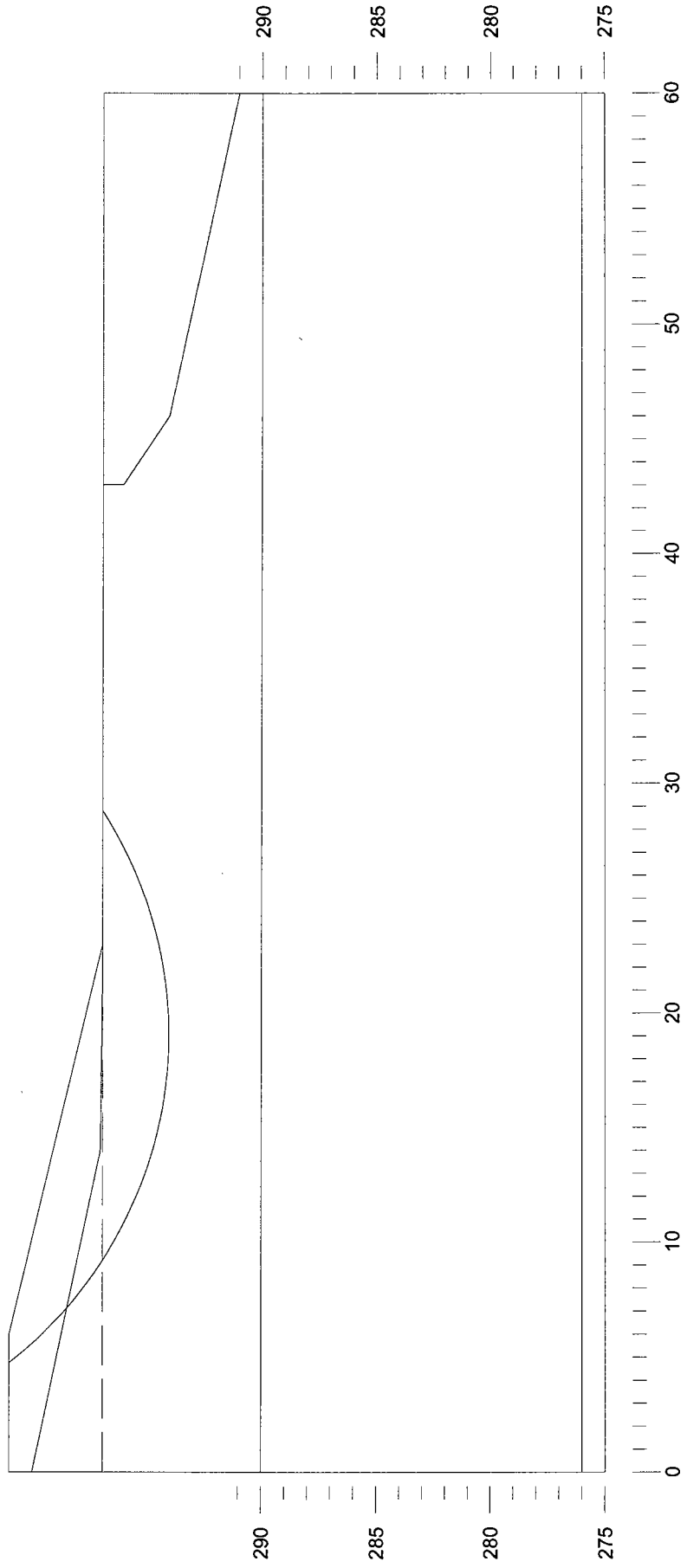


Figure F8

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 September 9, 2005  
 Magnetawan South NBL South Approach  
 Rock Fill 1.25:1 Construction

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Water	10	0	0	1
Rock Fill	20	0	42	1
Silt & Sand	21	0	30	2
Gravel	22	0	32	3

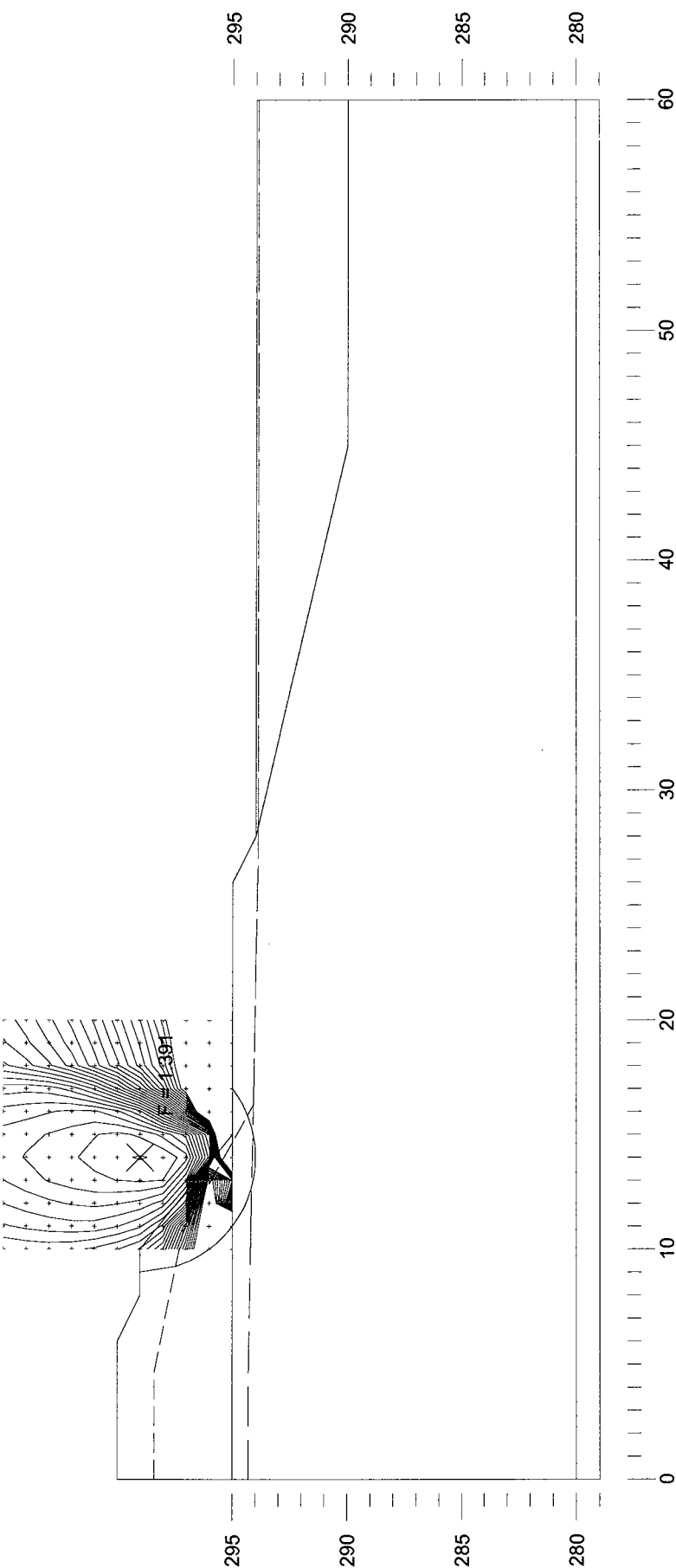


Figure F9

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 September 9, 2005  
 Magnetawan South NBL South 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	0	1
Silt & Sand	21	0	30	2
Gravel	22	0	32	3

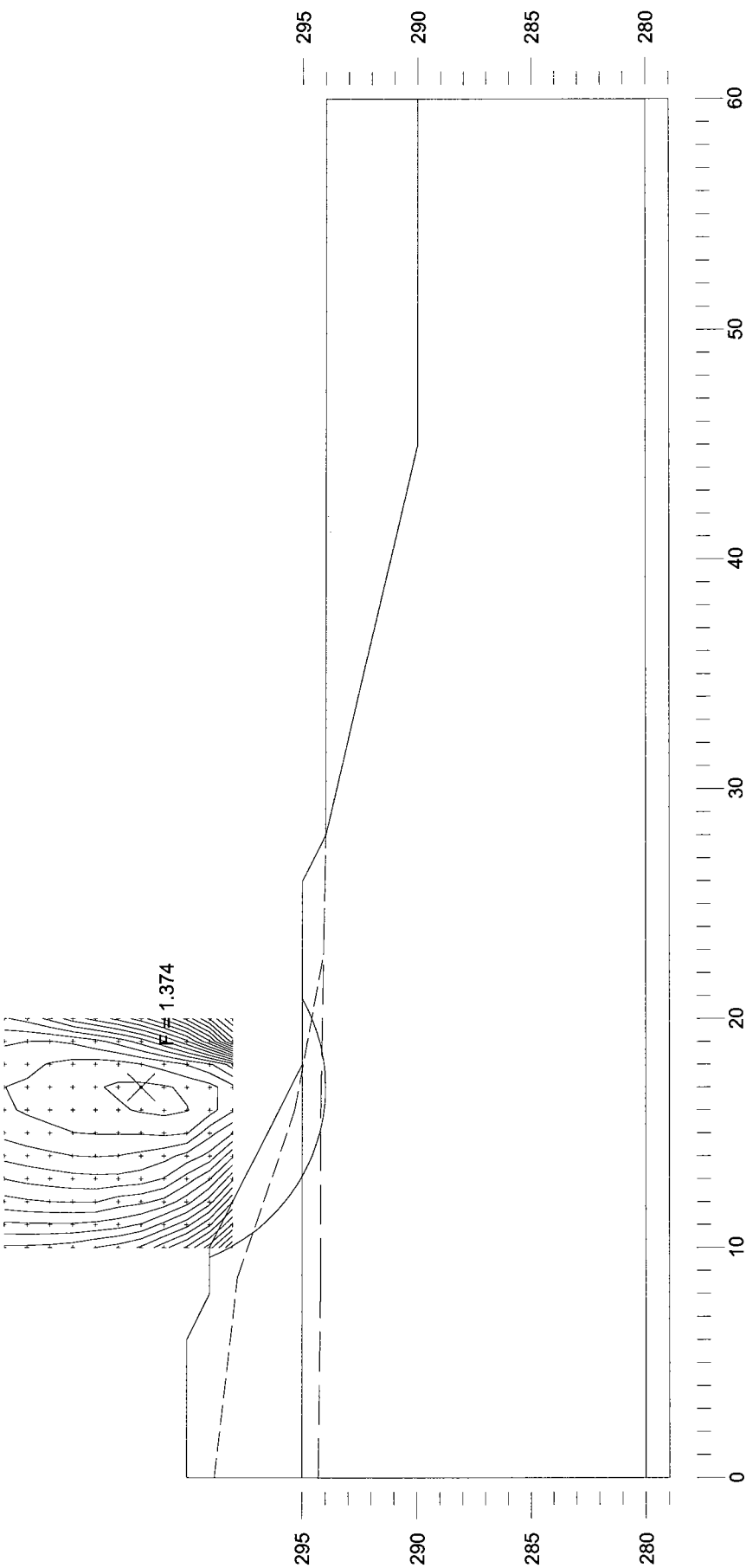


Figure F10

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 September 9, 2005  
 Magnetawan South NBL North Approach  
 Rock Fill 2:1 End of Construction

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Water	10	0	0	1
Rock fill	20	0	0	1
Sand and Silt	21	0	0	2
Clay	20	0	0	3
Gravel	22	0	0	4

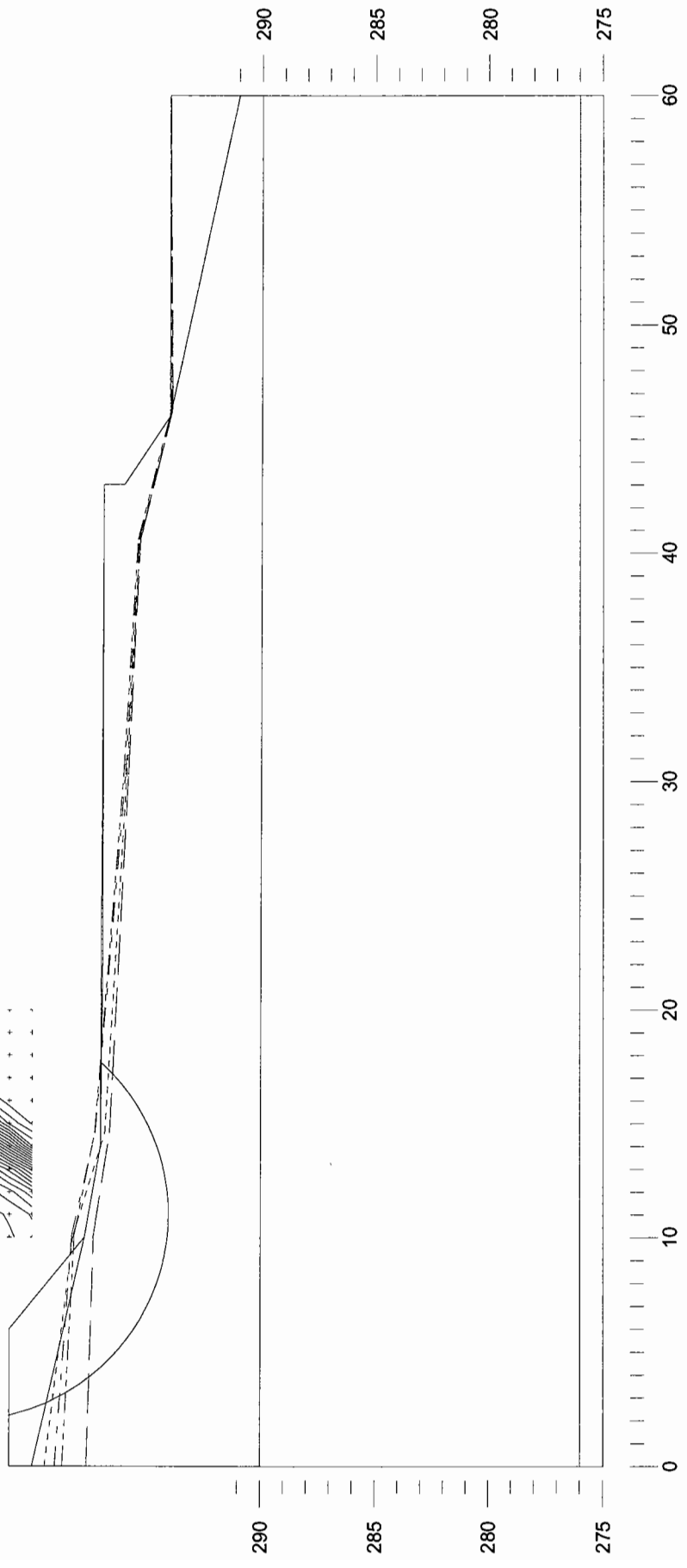
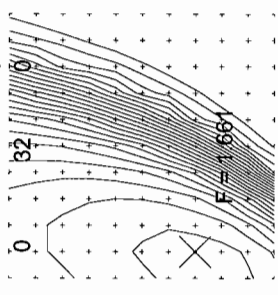


Figure F11

Thurber Engineering Ltd. - Toronto  
 19-1423-16  
 Hwy 11, Katrine  
 September 9, 2005  
 Magnetawan South NBL North Approach  
 Earth Fill 2:1 End of Construction

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Water	10	0	0	1
Earth Fill	22	0	0	1
Sand and Silt	21	30	0	2
Clay	20	30	0	3
Gravel	22	32	0	4

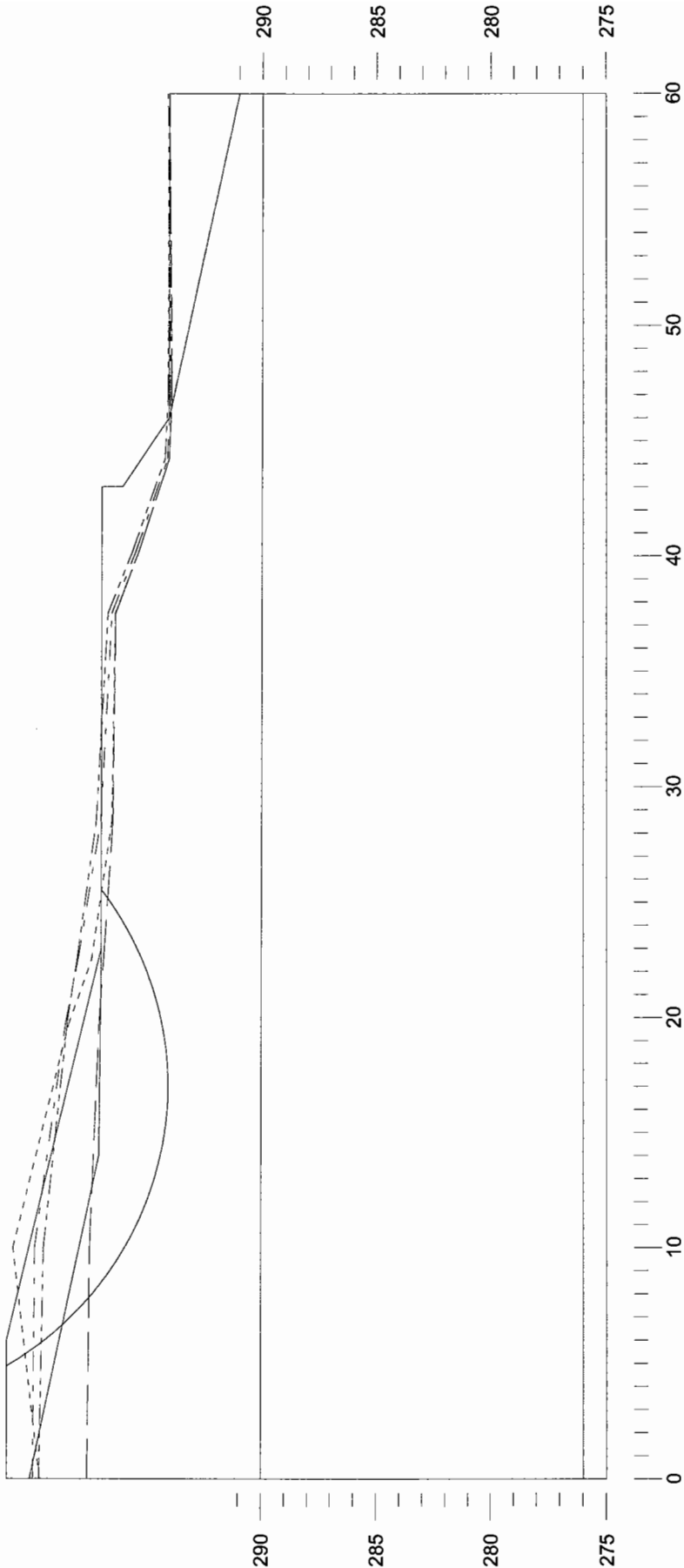
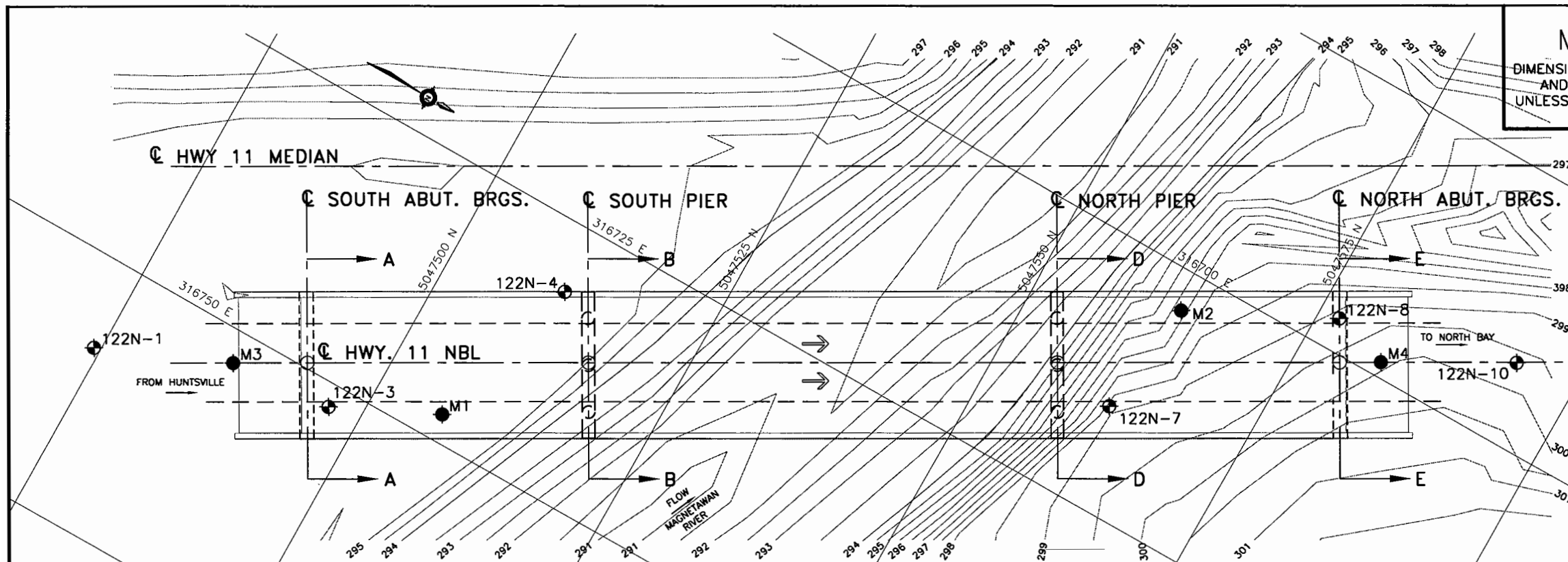


Figure F12

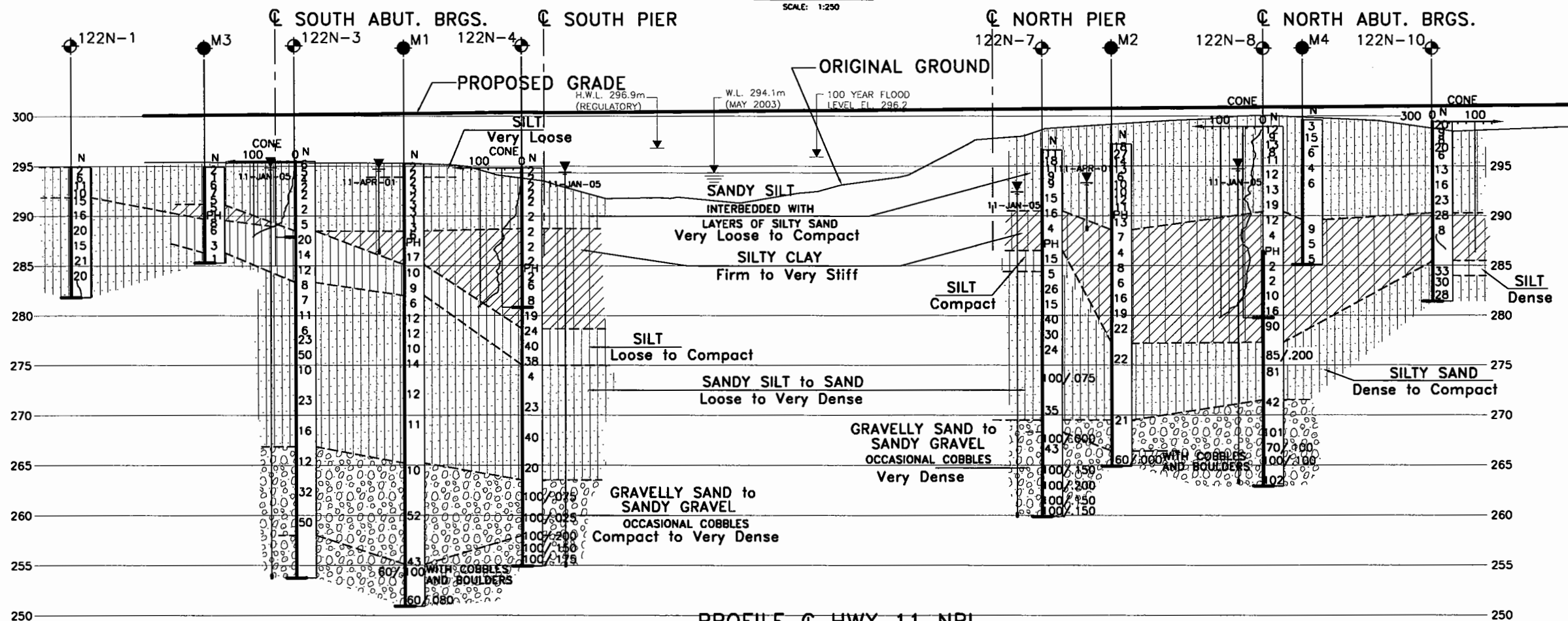


## **Appendix G**

### **Drawings**



PLAN  
SCALE: 1:250



PROFILE  $\perp$  HWY 11 NBL

SCALE: 1:250

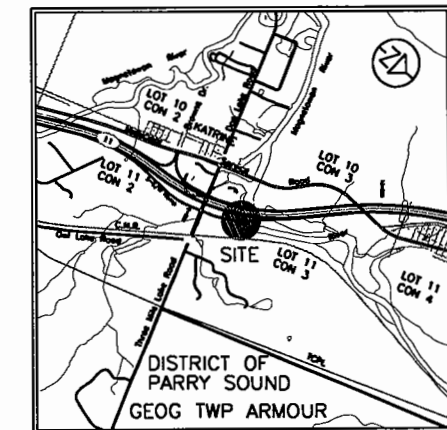
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

HWY 11  
CONT No  
WP No 474-93-01

MAGNETAWAN RIVER  
SOUTH CROSSING NBL  
GENERAL ARRANGEMENT

**Marshall Macklin Monaghan**  
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

**THURBER ENGINEERING LTD.**  
THURBER



KEY PLAN

SCALE: 1:5000

LEGEND

- $\odot$  Bore Hole by THURBER
- $\oplus$  Dynamic Cone Penetration Test (cone)
- $\bullet$  Bore Hole by SHAHEEN & PEAKER LIMITED
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- WL Head Artesian Water
- $\uparrow$  Piezometer
- 90% Rock Quality Designation (ROD)

NO	ELEVATION	NORTHING	EASTING
122N-1	294.9	5047474.4	316758.4
122N-3	295.9	5047496.8	316752.2
122N-4	294.8	5047511.2	316731.3
122N-7	296.6	5047562.0	316715.1
122N-8	299.0	5047577.0	316696.9
122N-10	299.4	5047594.0	316692.2
M1	295.2	5047506.7	316747.4
M2	297.2	5047563.5	316703.7
M3	294.9	5047486.8	316753.0
M4	299.6	5047582.5	316698.6

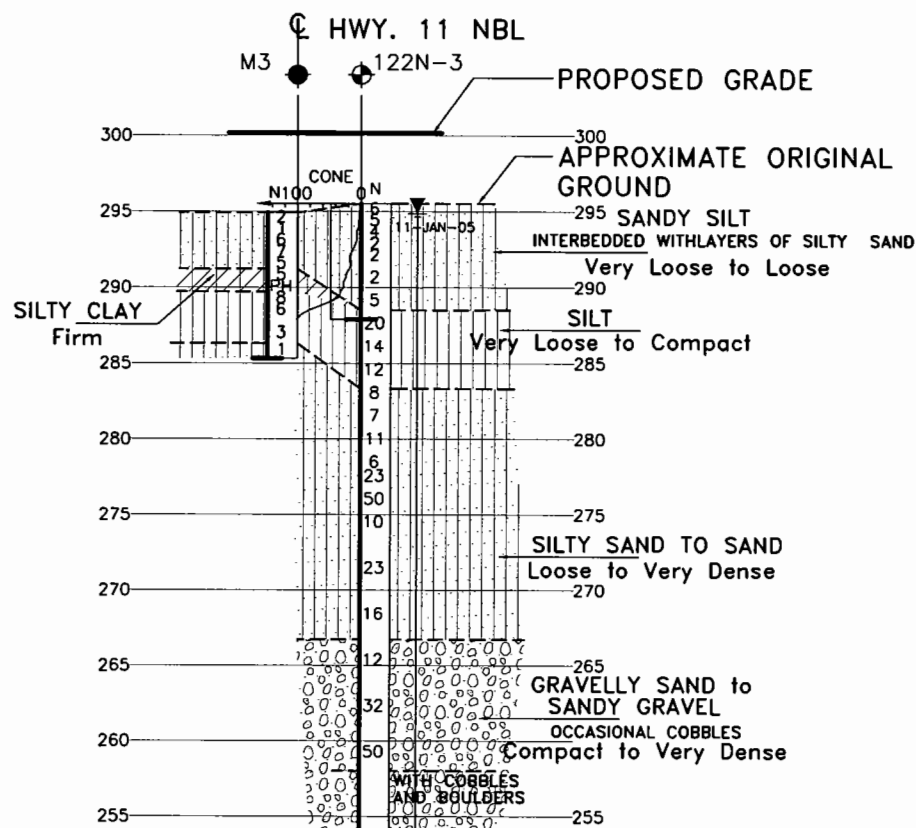
NOTE

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

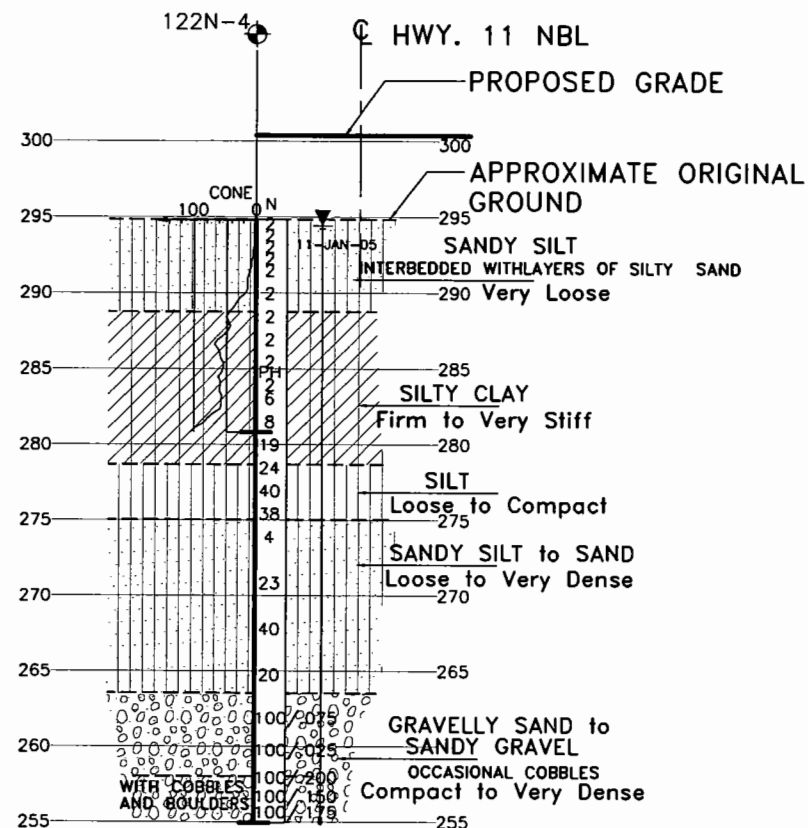
VCP : HCP No. 112  
EL. 298.289  
19mm  $\times$  1.52m IRON BAR  
2.3 LT 2.9km N OF HWY 518  
24.9 LT STA: 11+177.031

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

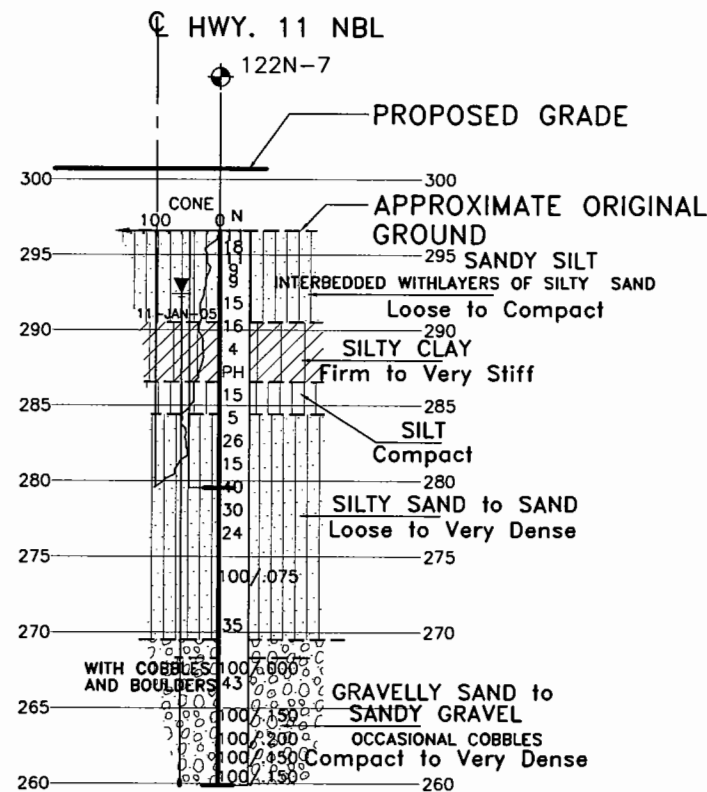
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CHK	CODE CHBDC 2000 [LOAD CL-625-0M]	DATE
DRAWN	CHK	SITE 44-122N [STRUCT.]	SCHEME DWG



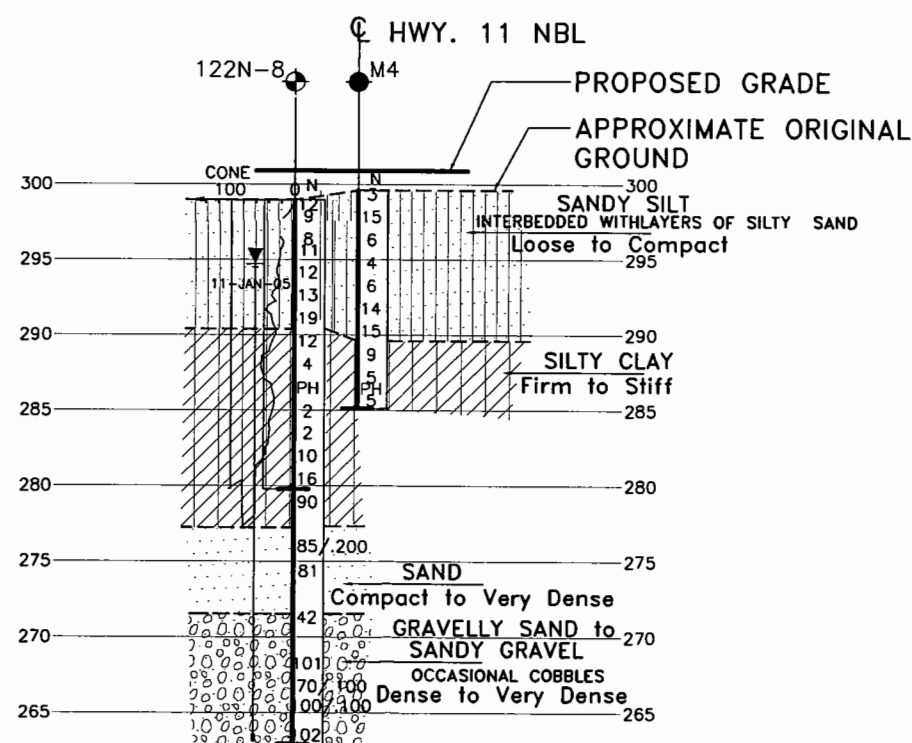
SECTION A-A  
SCALE: 1:250



SECTION B-B  
SCALE: 1:250



SECTION C-C  
SCALE: 1:250



SECTION D-D  
SCALE: 1:250

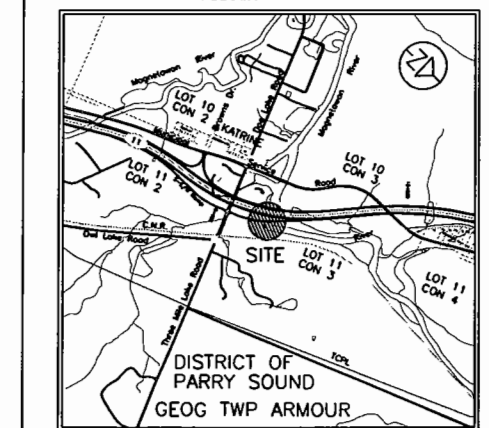
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

HWY 11  
CONT No  
WP No 474-93-01

MAGNETAWAN RIVER  
SOUTH CROSSING NBL  
GENERAL ARRANGEMENT

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**THURBER ENGINEERING LTD.**  
THURBER



KEY PLAN  
0 500m 1km

# LEGEND

- BoreHole by THURBER
- ⊕ Dynamic Cone Penetration Test (cone)
- BoreHole by SHAHEEN & PEAKER LIMITED
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- WL Head Artesian Water
- ⊕ Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
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M2	297.2	5047563.5	316703.7
M3	294.9	5047486.8	316753.0
M4	299.6	5047582.5	316698.6

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

VCP : HCP No. 112  
EL. 298.289  
19mmø x 1.52m IRON BAR  
2.3 LT 2.9km N OF HWY 518  
24.9 LT STA: 11+177.031

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CHK	CODE CHBDC 2000[LOAD CL-625-ONT]	DATE
DRAWN	CHK	SITE 44-122N[STRUCT.]	SCHEME [DWG]