

**FOUNDATION INVESTIGATION AND DESIGN REPORT
MAGNETAWAN RIVER NORTH CROSSING, E/W-N RAMP
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 479-93-01, SITE 44-400**

Geocres Number: 31E-217

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 E/W-N ramp from Three Mile Lake Road to the realigned Highway 11 over the Magnetawan River at the village of Katrine, Ontario. A previous, preliminary investigation had been carried out in the vicinity of the south abutment 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 and the stability of the river banks.

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 spans the Magnetawan River at a location where it is proposed that Highway 11 will cross the river. The site lies in the Village of Katrine, Armour Township, approximate 230 m east of existing Highway 11 and 220 m north of Three Mile Lake Road.

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 33 m wide and the maximum channel depth, based on May 2003 data, is 6 m. The river banks are low and no global stability problems were observed.

The area immediately to the south of the river is occupied by a seasonal campground. The active portion of the campground, nearer to the river, is open and grassy while closer to Three Mile Lake Road the ground is covered by bushes and scrubby trees. Some permanent buildings related to the campground are located close to the site.

To the north of the river, the land is wooded. It is low-lying and wet near the river but rises to the north. There is one residential building approximately 500 m north of the river crossing.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing for this project on the north bank of the Magnetawan River between November 26 and December 4, 2003. Further investigation was carried out on both banks of the river between February 8 and February 24, 2005. Investigation was carried out on the south bank between March 9 and March 14, 2001, as part of the preliminary investigation by S&P.

The current site investigation consisted of drilling and sampling five boreholes (Boreholes 400-1 to 400-5) to depths between 6.7 m at the north approach and 25.9 m at the north abutment. Borehole 400-1 was supplemented by dynamic cone penetration testing. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

Prior to the start of drilling, the borehole locations were staked in the field and utility clearances were obtained.

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 bedrock was encountered, it was proved by coring into it for a distance of at least 3 m.

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 | 400-2, 400-3 |
| North Abutment | 400-1, 400-5 |
| South Abutment | 400-4, MR 1*, MR 2* |
| South Approach | MR 3* |

* 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 tip, was installed in the borehole at the north abutment to monitor the groundwater level. A piezometer was installed at the south abutment during 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 400-1 | 25.9/268.9 | Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 23.5, bentonite seal to 22.9, grout to 0.6 and bentonite seal to the surface. |
| BH 400-5 | 21.3/273.8 | Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 19.2, bentonite seal to 18.6, grout to 0.9 and bentonite seal to the surface. |

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

4 LABORATORY TESTING

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

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A and to the Record of Borehole sheets prepared during the preliminary investigation included in Appendix C. 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. The bedrock is mantled by sand and gravel containing cobbles and boulders which is overlain by glacial outwash soils

deposited in glacio-fluvial and glacio-lacustrine environments. 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; discontinuous layers of silty clay; sandy silt and silty sand; a major sand unit; and bedrock.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

Topsoil was identified surficially in all boreholes drilled at the E/W-N ramp crossing. Topsoil thicknesses were established only at the borehole locations and ranged from 100 to 125 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Silty Clay

A layered silty clay unit was encountered below the topsoil in boreholes drilled on the south side of the river. This unit extended to 0.8 and 4.6 m depth (elevation 295.0 and 295.3) at the south abutment and south approach, respectively.

SPT values obtained in the clay ranged from 5 to 10 blows per 0.3 m of penetration. Two field vane tests indicated undrained shear strengths greater than 100 kPa. Based on this data, the clay is considered to have a firm to very stiff consistency.

Grain size distribution test results for this soil are reported on the Record of Borehole sheets and are plotted in Figure B4 of Appendix B and Figure 1 of Appendix C. Atterberg Limits determined for two samples, plotted on Figure 2 of Appendix C, show that the silty clay lies at the boundary of CL and ML soils. Moisture contents ranged from 29 to 39%.

5.4 Sandy Silt to Silty Sand

A layer of sandy silt to silty sand was encountered below the clay in the boreholes drilled on the south side of the river and below the topsoil on the north side of the river. The sandy silt/silty sand was loose to very loose with SPT values of 1 to 10 blows per 0.3 m. Moisture contents of 20 to 39% were measured. This layer was 0.6 to 3.0 m thick, extending to Elevation 294.4 to 292.6 on the south side of the river, and Elevation 291.8 and 293.3 on the north side.

Grain size distribution curves for the sandy silt and silty sand are included in Figure B1 of Appendix B and Figure 4 in Appendix C.

5.5 Sand

The topsoil and sandy silt/silty sand are underlain by a stratum of fine grained sand that forms the main stratum underlying the site.

At the south abutment, the sand extends to a depth of 22.4 m (Elevation 273.4) but is interrupted by a silt layer between 19.5 and 21.0 m depth. At the south approach, the sand

was not fully penetrated but extends at least to a borehole termination depth of 9.6 m (Elevation 290.3). At the north abutment, the sand extends to a depth of 22.6 m (Elevation 272.3). At the north approach, the sand was not fully penetrated but extends at least to the termination depth of 9.8 m (Elevation 285.0).

SPT values measured in the sand ranged from 0 (disturbed) to 8 blows per 0.3 m of penetration to depths of about 7.5 to 10.0 m (approximate elevations 284.5 to 288.0), indicating a very loose to loose condition. Below this level, the sand was compact to very dense, with SPT values ranging from 10 to 53 blows per 0.3 m of penetration. Dynamic cone penetration testing generally reflected a similar trend.

Cobbles were detected in the sand deposit below depths of 22.0 and 18.1 m (Elevation 273.8 and 276.7) at the south and north abutments, respectively. One SPT value of 50 blows for 25 mm of penetration measured in borehole 400-1 probably reflects the presence of the cobbles.

The measured natural moisture contents ranged from 19 to 31% with some higher values of up to 41%, notably at the north approach, that are attributed to the presence of organic inclusions. The soil is generally described as brown to grey in colour, wet and lies below the water table.

Grain size distributions for this soil are reported on the Record of Borehole sheets and are plotted in Figures B2 and B3 in Appendix B. Grain size results from the preliminary study are included in Appendix C as well.

5.6 Silt

A localized layer of firm silt was encountered within the sand from 19.5 to 21.0 m depth in the borehole at the south abutment (borehole MR1). The results of a grain size distribution analysis conducted on the silt are shown on Figure 9 in Appendix C.

5.7 Bedrock

The soils described above were found to be underlain by bedrock of the Pre-Cambrian Canadian Shield. The bedrock was proved by coring 3.2 m at the south abutment. At the north abutment, tricone wash boring was extended 450 mm into the rock in one borehole then coring was commenced, extending to total lengths of 3.1 to 3.3 m below the bedrock surface.

The rock is described as grey gneiss at the south abutment and as granitic gneiss at the north abutment. In general, the rock is laminated to thinly banded and unweathered to slightly weathered, with some jointed zones. The core recovery was essentially 100% and RQD values ranged from 82 to 100%, indicating a good to excellent quality rock.

Based on Point Load Testing, the unconfined compressive strength of the bedrock at the north abutment was estimated to range from 141 to 143 MPa. Based on these strength

values and the classification system given in the Canadian Foundation Engineering Manual, the rock was classified as very strong.

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 bedrock, are shown in Table 5.1. An SPT value of 50 blows for 25 mm of penetration measured near elevation 274 in borehole 400-1 is believed to reflect the presence of cobbles.

Table 5.1 – Refusal Depths (Elevations)

| Location | Borehole | Refusal Depth (Elevation), m | Material |
|----------------|----------|------------------------------|----------|
| North Abutment | 400-1 | 22.6 (272.3) | Bedrock |
| | 400-5 | 22.5 (272.6) | Bedrock |
| South Abutment | MR1 | 22.4 (273.4) | Bedrock |
| | 400-4 | 22.3 (273.3) | Bedrock |

5.9 Water Levels

The recorded groundwater depths and elevations are shown in Table 5.2.

Table 5.2 – Groundwater Depths and Elevations

| Location | Borehole | Date | Water Level (m) | | Comment |
|----------------|----------|------------------|-----------------|-----------|-----------------|
| | | | Depth | Elevation | |
| South Abutment | MR1 | March 14, 2001 | 0.9 | 294.9 | In piezometer |
| | | April 11, 2001 | 0.3 | 295.5 | In piezometer |
| South Approach | MR3 | March 9, 2001 | 5.6 | 294.3 | Not stabilized |
| North Abutment | 400-1 | - | - | - | - |
| North Approach | 400-2 | December 4, 2003 | 0.9 | 293.9 | Upon completion |

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

6 MISCELLANEOUS

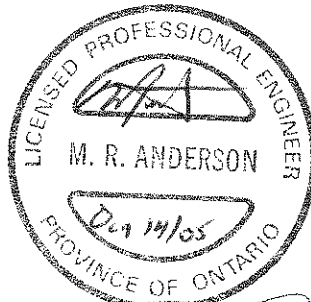
Marshall Macklin Monaghan completed field layout for the site investigation and provided borehole coordinates and ground surface elevations.

All-Terrain Drilling supplied and operated the drilling and sampling equipment used for the current investigation.

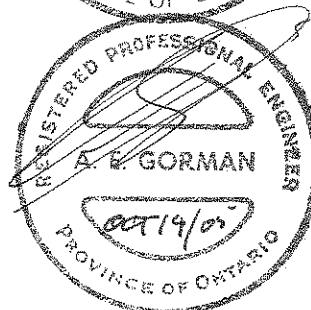
Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. Donald Parent, B.Sc. of Thurber.

The investigation report was prepared by Mr. Murray Anderson, P.Eng. Overall supervision of the field program, interpretation of the field data, and review of the report was performed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, Ph.D., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

A single-span, 52 m long, welded steel plate girder structure is proposed at this site and integral abutments are under consideration. Both approaches will lie on comparatively flat, low-lying land of the river flood plain. The undersides of the abutment stems will lie approximately 1 to 2 m above the river level.

At the north abutment, the finished grade will be about Elevation 301.9 and the original ground lies at Elevation 295.0±, resulting in an approach fill approximately 7 m high.

At the south abutment, the finished grade will be about Elevation 301.0 and the original ground lies at Elevation 295.5 ±, resulting in an approach fill approximately 5.5 m high.

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, together with the factual data from the previous investigation by S&P.

8 STRUCTURE FOUNDATIONS

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

Based on the results of the exploratory boreholes drilled at the proposed abutment locations, the stratigraphy consists of approximately 22.5 m of predominantly sand overlying bedrock. The sand is loose to very loose to depths of 7.5 to 10.0 m, and becomes compact to very dense below this level. Cobbles were encountered in the sand below depths of 18 to 22 m.

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 near surface soils at the abutment locations are considered too loose to provide adequate support to spread footings due to the low bearing resistance available and the potential for comparatively large settlements.

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

8.1.2 Footings on Engineered Fill

Very loose to loose soils were encountered to depths of about 7.5 to 10.0 m at both the south and north abutments. These soil conditions are considered unsuitable for the support of an engineered fill pad due to the low bearing resistance available and the potential for comparatively large settlements.

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. Although the investigation at this site indicated loose to dense sand and silt overlying bedrock, the two adjacent bridge structures are underlain by a layer of very dense gravelly sand, cobbles and boulders above the bedrock. Accordingly, it is considered prudent to design the piles at this site on the basis of achieving resistance in the very dense soil above the bedrock, while recognizing that they may in fact reach bedrock.

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) | Estimated Pile Tip Elevation | |
| | | | N. Abutment | S. Abutment |
| HP 310 X 110 | 1,800 kN | 1,600 kN | 272.3 | 273.4 |
| HP 360 X 132 | 2,100 kN | 1,800 kN | 272.3 | 273.4 |
| HP360 X 174 | 2,200 kN | 1,900 kN | 272.3 | 273.4 |

The pile tip elevations shown in Table 8.1 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.2.3 Pile Driving.

8.2.1 Pile Tips

The tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point), Pruyn Points or approved equivalent. The use of rock points is recommended for the following reasons:

- The piles will be penetrating into soil that may contain numerous cobbles and boulders, which requires a higher level of protection than driving into normal glacial till
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock
- Some piles may fully penetrate the zone of cobbles and boulders and achieve refusal on the 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 possible boulders in the sands just above bedrock.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal in the zone of cobbles and boulders above the rock.

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 Driving

Pile driving below Elevation 275.0 should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have the minimum values shown in Table 8.2.

Table 8.2 – Ultimate Geotechnical Resistance of Piles

| File | Ultimate Resistance, R |
|------------|------------------------|
| HP 310X110 | 3,600 kN |
| HP360X132 | 4,200 kN |
| HP360X174 | 4,400 kN |

The Contractor should be alerted to the fact that the piles may, in fact, achieve refusal on the bedrock at this site. If this happens, the Hiley formula is not applicable and a site decision must be made that bedrock has been encountered and that further pile driving must be controlled to adequately seat the pile in the bedrock.

To avoid overdriving and damaging the toe, a limiting criterion of 10 blows at full energy for 12 mm penetration for two consecutive sets of 10 blows should be established to control pile driving on bedrock. The geotechnical resistances given in Table 8.1 remain valid in this situation.

8.2.4 Downdrag

The soils at the abutments are non-cohesive and settlements induced in the native soils around the piles by construction of the approach embankments will be substantially complete as construction of the embankment is completed. Post-construction downdrag on the piles is therefore not considered to be an issue at this site. However, it is recommended that the approach embankments be constructed three months in advance of pile driving. The embankment should be constructed up to the level of the abutment from the forward slope to a distance back sufficiently far to allow access and operation of construction equipment. Beyond that distance, the embankment should be constructed to full height.

8.2.5 Lateral Resistance of Piles

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

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

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

where z = depth of embedment of pile in metres

D = pile width in metres

| | | |
|----------|---|---|
| n_h | = | coefficient of horizontal subgrade reaction (Table 8.3) |
| γ | = | unit weight (Table 8.3) |
| K_p | = | passive earth pressure coefficient (Table 8.3) |

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

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

Table 8.3 – Parameters for Lateral Pile Resistance

| Location | Elevation | n_h (kN/m ³) | K_p | Unit Weight (kN/m ³) |
|----------------|------------|-------------------------------|-------|-------------------------------------|
| South Abutment | OGL to 290 | 1,200 | 2.7 | 19 |
| | 290 to 285 | 3,000 | 2.9 | 19 |
| | 285 to 273 | 5,000 | 3.1 | 20 |
| North Abutment | OGL to 284 | 1,200 | 2.7 | 19 |
| | 284 to 277 | 4,000 | 3.0 | 20 |
| | 277 to 272 | 5,000 | 3.1 | 20 |

The total horizontal passive resistance of a single pile should not exceed the following values:

Table 8.4 – Maximum Horizontal Passive Resistance of Piles

| Pile | Maximum Passive Resistance | |
|------------|----------------------------|-------|
| | Factored ULS | SLS |
| HP 310X110 | 120 kN | 50 kN |
| HP360X132 | 160 kN | 60 kN |
| HP360X174 | 160 kN | 60 kN |

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.5. Intermediate values may be obtained by linear interpolation.

Table 8.5 – Subgrade Reaction Reduction Factors for Pile Spacing

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

* where D is the breadth of pile

In the case of 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 on bedrock. When attempting to found on bedrock, there could be difficulties sealing the liner to allow unwatering of the caisson and placement of concrete in the dry.

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

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

8.4 Recommended Foundation

The recommended foundation system for both abutments at this site is steel H-piles driven to rock or 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 loose to very loose silt and sand, which in its original state would provide sufficient flexibility. However, if the upper 3 m of the piles lies in compacted fill or if the native soil becomes 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 a 600 mm diameter CSP filled with sand (for a “true abutment” supported on piles) or by concentric CSPs in accordance with standard integral abutment design procedures (for a “false abutment”).

Backfill sand placed in the CSP should meet the gradation shown in Table 8.6 and must be placed after pile driving to minimize the potential for densification.

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% |

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. A 25 mm thickness of rigid, extruded polystyrene insulation is equivalent to 600 mm of earth cover. Synthetic insulation must be covered to provide protection where it is used.

The forward slope for the approach embankment must be constructed of rock fill to satisfy the assumptions made in the stability analyses. Frost may penetrate deeper through rock fill than earth fill and the possibility exists for freezing conditions to develop below the pile cap. Therefore, non-frost susceptible free-draining granular fill with less than 5% particles by mass finer than 75 µm should be specified for construction of the pile driving pad within the rock fill.

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 native soils 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 below the deepest excavation level by a sufficient depth 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 methods such as excavation within a cofferdam may be required. Selection and design of the appropriate dewatering system is the responsibility of the Contractor. The Contract Documents should alert him to the requirement to maintain a stable excavation and that any shoring system should be designed by a shoring specialist, taking account of the need to control groundwater and prevent basal instability within the excavation.

10 UNWATERING

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

The design of any dewatering system that may be required is 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 the side slopes and the forward slope. The required minimum setback of the abutments from the river has been determined on the basis of the results of the stability analysis of the forward slope.

11.1 South Approach Forward Slope

The computer output for the stability analysis of the south approach forward slope is shown in Figure F1 in Appendix F.

Various conditions were analysed to select an appropriate setback from the river's edge.

Based on these analyses, it has been determined that:

- The immediate approach fill and forward slope must be constructed of rock fill, at a maximum slope of 1.25H:1V.
- The minimum recommended distance from the abutment bearing centreline to the edge of the river is 7.5 m, for a minimum factor of safety of 1.4.

The edge of the river has been defined, for the purpose of this analysis as the edge of water when the river level is at Elevation 294.1.

11.2 South Approach Lateral Stability

The global and internal stability of the approach embankment side slopes was analysed on the basis of a rock fill embankment, as determined in Section 11.1.

The computer output for the stability analysis of the south approach side slope is shown in Figure F2 in Appendix F. This analysis shows that a 1.25H:1V side slope constructed of rock fill has a factor of safety of 2.7 against failure.

11.3 North Approach Forward Slope

The computer output for the stability analysis of the north approach forward slope is shown in Figure F3 in Appendix F.

Various conditions were analysed to select an appropriate setback from the river's edge. Based on these analyses, it has been determined that:

- The immediate approach fill and forward slope must be constructed of rock fill, at a maximum slope of 1.25H:1V.
- The minimum recommended distance from the abutment bearing centreline to the edge of the river is 8.5 m, for a minimum factor of safety of 1.5

The edge of the river has been defined, for the purpose of this analysis as the edge of water when the river level is at Elevation 294.1.

11.4 North Approach Lateral Stability

The global and internal stability of the approach embankment side slopes was analysed on the basis of a rock fill embankment, as determined in Section 11.1.

The computer output for the stability analysis of the north approach side slope is shown in Figure F4 in Appendix F. This analysis shows that a 1.25H:1V side slope constructed of rock fill has a factor of safety of 2.1 against failure.

11.5 Settlement

The native subsoils under the immediate approach embankments are regarded as behaving as cohesionless materials and settlements are expected to be immediate in nature. It is estimated that the settlement under the embankment loading will be in the order of 90 mm. This value was computed by calculating the stress increase under a rockfill embankment with a height of 7 m, platform width of 11.5 m, and sideslopes of 1.25H:1V as per Osterberg's elastic solution (1957), assuming typical values for the constrained elastic modulus of the subsoil at each metre of depth (from geotechnical literature), and summing the incremental elastic compression of each metre depth of soil under the applied load.

Settlement within the 7 m high embankment of rockfill itself is expected to be in the order of 10 mm after 3 months, 20 mm after 1 year, and 75 mm after 10 years.

11.6 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 14: Seismic Considerations.

11.7 Forward Slope Protection

The analysis of the forward slopes and the resulting recommendations are based on the river channel remaining in its present location and the forward slope being constructed of rock fill.

From a foundations perspective, no further protection of the forward slope is required. However, other factors may need consideration and if the hydraulic analysis indicates that scour can occur at the river edge then the rock fill must be embedded into the river bank to the maximum depth of scour.

11.8 General Embankment Requirements

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

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and included in Appendix E.

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

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

Earth fill 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 are not recommended as the best option for retaining structures at this site due to the loose near surface soils and consequent potential for unacceptable settlements. From a geotechnical viewpoint, a conventional abutment supported on the deep foundation system is recommended. However, recognizing the cost implications of this type of abutment and the preference for integral abutment design, an alternative wall type that can tolerate potential differential settlement, such as a mechanically stabilized earth (MSE) wall, may be considered.

If an RSS system is considered, the supplier/designer of the proprietary system must demonstrate that it will meet the Ministry’s specifications and performance requirements. The following minimum preparation of the base below the RSS is recommended:

- For a RSS bearing on the native soil, the base should be underlain by a minimum 2 m thickness of engineered fill and the highest level for the underside of the engineered fill should be Elevation 294 at the north abutment and Elevation 292 at the south abutment.
- For a RSS wall bearing on rock fill, the rock fill subgrade must be blinded with spall material and covered by a minimum 600 mm thickness of OPSS Granular B Type II fill.

An RSS constructed on a minimum 2 m thick pad of engineered fill may be designed using a factored bearing resistance at ULS of 300 kPa. Settlement of the RSS will be governed by the elastic compression of the subsoil under the embankment load; at the wall face, settlement is expected to be in the order of 50 mm below the embankment platform decreasing to 10 mm at the toe of a 1.25H:1V embankment slope. An unfactored friction angle of 28° should be assigned to the very loose silty sand/sandy silt for evaluation of sliding resistance along the base of the wall. Global stability of the forward slope of the embankment was discussed in Section 11.

The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design.

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 and including adequate spalls to fill voids in the rock fill.

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 Table 14.1)

γ = unit weight of retained soil (see Table 14.1)

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

q = value of any surcharge (kPa)

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

Table 14.1 – Earth Pressure Coefficients (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:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) |
| 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.

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.

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.

15 SEISMIC CONSIDERATIONS

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

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

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

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

15.2 Liquefaction Potential

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

Using this method, it was determined that the foundation soils at the abutments are not in danger of liquefaction under earthquake loading.

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

Table 15.1 – Earth Pressure Coefficients (K) for Seismic Design

| Condition | Earth Pressure Coefficient (K) for Earthquake Loading | | | | | |
|---|---|---|--|---|---|---|
| | OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$ | | OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$ | | Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \delta = 21^\circ$ | |
| | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) |
| Active*, K_{AE} (Unrestrained Wall) | 0.28 | 0.46 | 0.31 | 0.58 | 0.21 | .30 |
| At rest**, K_{OE} (Restrained Wall) | 0.53 | - | 0.58 | - | .44 | - |
| Passive*, K_{PE} (Movement Towards Soil Mass) | 7.0 | - | 5.5 | - | 14.1 | - |

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

** After Woods

In Table 15.1, the angle of friction between the wall and the backfill, δ , is taken as 50% of the angle of internal friction of the backfill, ϕ .

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

15.4 Slope Stability Considerations

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

16 CONSTRUCTION CONCERNS

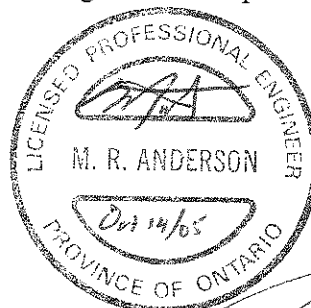
Potential construction concerns include, but are not necessarily limited to:

- The possibility of piles reaching refusal on cobbles and boulders above the bedrock. In this case, the Hiley formula should be applied and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- The nature of the fill used to construct the approach fills. Certain recommendations in the report regarding stability and the set back from the river's edge are based on construction with rock fill. If other material is substituted, these recommendations are no longer valid.
- Excavation and unwatering close to the river.

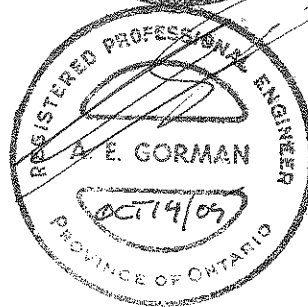
17 CLOSURE

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

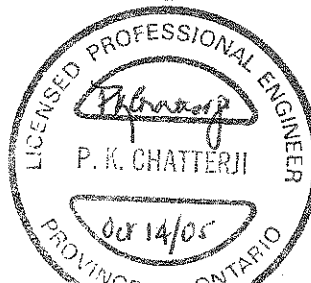
Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
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Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

| CLASSIFICATION | PARTICLE SIZE | VISUAL IDENTIFICATION |
|----------------|--------------------|---|
| Boulders | Greater than 200mm | same |
| Cobbles | 75 to 200mm | same |
| Gravel | 4.75 to 75mm | 5 to 75mm |
| Sand | 0.075 to 4.75mm | Not visible particles to 5mm |
| Silt | 0.002 to 0.075mm | Non-plastic particles, not visible to the naked eye |
| Clay | Less than 0.002mm | Plastic particles, not visible to the naked eye |

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

| TERMINOLOGY | PROPORTION |
|---------------------------------|---------------|
| Trace or Occasional | Less than 10% |
| Some | 10 to 20% |
| Adjective (e.g. silty or sandy) | 20 to 35% |
| And (e.g. sand and gravel) | 35 to 50% |

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

| DESCRIPTIVE TERM | UNDRAINED SHEAR STRENGTH (kPa) | APPROXIMATE SPT ⁽¹⁾ 'N' VALUE |
|------------------|--------------------------------|--|
| Very Soft | 12 or less | Less than 2 |
| Soft | 12 to 25 | 2 to 4 |
| Firm | 25 to 50 | 4 to 8 |
| Stiff | 50 to 100 | 8 to 15 |
| Very Stiff | 100 to 200 | 15 to 30 |
| Hard | Greater than 200 | Greater than 30 |

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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






 Water Level
 Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

| DISCONTINUITY SPACING | | STRENGTH CLASSIFICATION | | | |
|-------------------------------------|--|-------------------------|---|---------------------|--|
| Bedding | Bedding Plane Spacing | Rock Strength | Approximate Uniaxial Compressive Strength | | 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 400-1

1 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5048595.4 E 316392.9 Magnetawan River E/W-N Ramp, ST. 12+268.5 CL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core
 DATUM Geodetic DATE 26.11.03 - 27.11.03
 ORIGINATED BY DP
 COMPILED BY SS
 CHECKED BY AEG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|--|----|----|------------------------------------|-------------------------------------|-----------------------------------|--|--|-------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | WATER CONTENT (%) | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | | | | |
| 294.8 | | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | | | | |
| 294.8 | TOPSOIL | | | | | | | | | | | | | | | | | |
| 0.1 | Silty SAND to Sandy SILT, trace organics Very Loose Brown Wet | | 1 | SS | 1 | | | | | | | | ○ | | 0 53 44 3 | | | |
| | Grey | | 2 | SS | 2 | | | | | | | | ○ | | | | | |
| | | | 3 | SS | 2 | | | | | | | | ○ | | 0 45 55 (SI+CL) | | | |
| 291.8 | | | | | | | | | | | | | | | | | | |
| 3.0 | SAND, fine grained, some silt to silty, trace organics to 5.6m Very Loose to Compact Grey Wet | | 4 | SS | 3 | | | | | | | | ○ | | | | | |
| | | | 5 | SS | 2 | | | | | | | | ○ | | 0 74 23 3 | | | |
| | | | 6 | SS | 1 | | | | | | | | ○ | | | | | |
| | Brown | | 7 | SS | 8 | | | | | | | | ○ | | | | | |
| | | | 8 | SS | 6 | | | | | | | | ○ | | 0 79 21 (SI+CL) | | | |

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 400-1

2 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5048595.4 E 316392.9 Magnetawan River E/W-N Ramp, ST. 12+268.5 CL ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core COMPILED BY SS
DATUM Geodetic DATE 26.11.03 - 27.11.03 CHECKED BY AEG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT NATURAL MOISTURE CONTENT W _p W W _L WATER CONTENT (%) 20 40 60 | LIQUID LIMIT | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|-------------|------------|---------|------|------------|----------------------------|-----------------|---|---|--------------|---------------------------------------|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | | |
| | | | 9 | SS | 13 | | 284 | | | | | |
| | | | | | | | 283 | | | | | |
| | | | 10 | SS | 14 | | 282 | | | | | |
| | | | | | | | 281 | | | | | |
| | | | 11 | SS | 17 | | 280 | | | | | 0 58 40 2 |
| | | | | | | | 279 | | | | | |
| | | | 12 | SS | 11 | | 278 | | | | | |
| | | | | | | | 277 | | | | | |
| | | | 13 | SS | 39 | | 276 | | | | | |
| | | | | | | | 275 | | | | | |

Dense
with cobbles

Continued Next Page

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

RECORD OF BOREHOLE No 400-1

3 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5048595.4 E 316392.9 Magnetawan River E/W-N Ramp, ST. 12+268.5 CL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/INQ Core
 DATUM Geodetic DATE 26.11.03 - 27.11.03
 ORIGINATED BY DP
 COMPILED BY SS
 CHECKED BY AEG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT Y kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|-------------|----------------------------|-----------------|---|----------|--|------------------------------------|-------------------------------------|-----------------------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE | | | | | | | |
| | | | | | | | | ● QUICK TRIAXIAL x LAB VANE | | | | | | | |
| | | | | | | 20 40 60 80 100 | | | 20 40 60 | | | | | | |
| | | | 14 | SS | 50/ .025 | | 274 | | | | | | | | |
| | | | | | | | 273 | | | | | | | | |
| 272.3 | | | | | | | | | | | | | | | |
| 22.6 | GRANITIC GNEISS (BEDROCK) Slightly weathered, laminated to thinly banded, pale pink with subvertical dark banding | | 1 | GS | | | 272 | | | | | | | | |
| | | | 1 | RUN | | | 271 | | | | | | | | RUN 1# TCR=99%, SCR=95%, RQD=82%, UCS=143MPa, |
| | | | 2 | RUN | | | 270 | | | | | | | | RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=141MPa, |
| 268.9 | | | | | | | 269 | | | | | | | | |
| 25.9 | END OF BOREHOLE AT 25.93m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. | | | | | | | | | | | | | | |

RUN 1#
TCR=99%,
SCR=95%,
RQD=82%,
UCS=143MPa,

RUN 2#
TCR=100%,
SCR=100%,
RQD=100%,
UCS=141MPa,

ONTM14 MAGENTAWAN RIVER.GPJ 15/004

RECORD OF BOREHOLE No 400-2

1 OF 1

METRIC

W.P. 479-93-01 LOCATION N 5048623.9 E 316356.0 Magnetawan River, E/W-N Ramp, ST. 12+317.5, O/S 5L ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 04.12.03 - 04.12.03 CHECKED BY AEG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|-----------------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | | |
| 294.8 294.0 0.1 | TOPSOIL Silty SAND to Sandy SILT, fine grained Very Loose Brown Wet (SM) | | 1 | SS | 1 | | 294 | | | | | | | |
| 293.4 1.5 | SAND, fine grained, trace silt to silty, trace organics, occasional wood fibers Very Loose Grey Wet (SP) | | 2 | SS | 1 | | 293 | | | | | | | |
| | | | 3 | SS | 2 | | 292 | | | | | | | |
| | | | 4 | SS | 2 | | 291 | | | | | | | |
| | | | 5 | SS | 1 | | 290 | | | | | | | 0 64 32 4 |
| | Brown | | 6 | SS | 3 | | 289 | | | | | | | |
| 288.1 6.7 | END OF BOREHOLE AT 6.71m. BOREHOLE OPEN TO 1.52m. WATER LEVEL IN OPEN BOREHOLE AT 0.91m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE. | | | | | | | | | | | | | |

ONTM14 MAGENTAWAN RIVER.GPJ 15/10/04

RECORD OF BOREHOLE No 400-3

1 OF 2

METRIC

W.P. 479-93-01 LOCATION N 5 048 612.4 E 316 375.8, Magnetawan River Bridge, E/W-N Ramp ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 24.02.05 - 24.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|-----------------|-----------------|-----------------|-----------------|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | | |
| 294.8 | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL (75 mm) | | | | | | | | | | | | | |
| 0.1 | SAND, some silt to silty, some organics, trace rootlets Loose to Very Loose Grey/Brown Wet | | 1 | SS | 6 | | | | | | | | | |
| | | | 2 | SS | 3 | | 294 | | | | | | | |
| 293.3 | | | | | | | | | | | | | | |
| 1.5 | SAND, some silt, trace rootlets, occasional organics, trace iron oxide staining Very Loose to Loose Grey Wet | | 3 | SS | 4 | | 293 | | | | | | | |
| | | | 4 | SS | 4 | | 292 | | | | | | | 0 71 26 3 |
| | | | 5 | SS | 2 | | 291 | | | | | | | |
| | | | 6 | SS | 7 | | 290 | | | | | | | |
| | | | | | | | 289 | | | | | | | |
| 288.7 | | | | | | | | | | | | | | |
| 6.1 | SAND, fine to medium grained, some silt, trace clay Loose to Compact Brown Wet | | 7 | SS | 6 | | 288 | | | | | | | 0 79 21 (SI+CL) |
| | | | 8 | SS | 19 | | 287 | | | | | | | |
| | | | 9 | SS | 15 | | 286 | | | | | | | |
| 285.0 | | | | | | | | | | | | | | |
| 9.8 | END OF BOREHOLE AT 9.75 m. | | | | | | | | | | | | | |

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 400-3

2 OF 2

METRIC

W.P. 479-93-01 LOCATION N 5 048 612.4 E 316 375.8, Magnetawan River Bridge, E/W-N Ramp ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 24.02.05 - 24.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT | NATURAL MOISTURE CONTENT | LIQUID LIMIT | UNIT WEIGHT Y kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|-------------------|------------------|--------------------------------|-----------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | WATER CONTENT (%) | | | | | |
| | BOREHOLE OPEN TO 9.75 m. BOREHOLE GROUTED TO SURFACE. WATER LEVEL IN OPEN BOREHOLE AT 0.3m DEPTH UPON COMPLETION. | | | | | | | | | | | | | |

ONTM14S 2316(396, 400).GPJ 14/10/05

RECORD OF BOREHOLE No 400-4

1 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5 048 561.5 E 316 425.6, Magnetawan River Bridge, EW-N Ramp ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 08.02.05 - 09.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT Y kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|------------------------------------|-------------------------------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | |
| 295.6 | | | | | | | | | | | | |
| 0.0 | SILT, some sand to sandy, trace rootlets Loose Brown Wet | | 1 | SS | 6 | | | | | | | |
| | | | | | | | 295 | | | | | |
| | | | | | | | 294 | | | | | |
| | | | | | | | 293 | | | | | |
| 292.6 | | | | | | | | | | | | |
| 3.0 | SAND, fine grained, trace silt Loose Brown Moist | | 2 | SS | 5 | | | | | | | |
| | | | | | | | 292 | | | | | |
| | | | | | | | 291 | | | | | |
| | | | | | | | 290 | | | | | |
| 289.5 | | | | | | | | | | | | |
| 6.1 | SAND, fine to medium grained, trace silt Loose to Compact Brown Wet | | 3 | SS | 7 | | | | | | | |
| | | | | | | | 289 | | | | | |
| | | | | | | | 288 | | | | | |
| | | | | | | | 287 | | | | | |
| | | | 4 | SS | 8 | | | | | | | |
| | | | | | | | 286 | | | | | |

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

[illegible]

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 400-4

3 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5 048 561.5 E 316 425.6, Magnetawan River Bridge, E/W-N Ramp ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 08.02.05 - 09.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|-----------------|------------------------------------|-------------------------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | 20 40 60 80 100 | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | |
| 273.7 | | | 8 | SS | 20 | | 275 | | | | | | |
| 21.9 | | | | | | | 274 | | | | | | |
| 273.3 | SAND and GRAVEL, trace silt Brown | | | | | | | | | | | | |
| 22.3 | Wet END OF SOIL SAMPLING AT 22.33 m. CORING STARTED AT 22.33 m. Fresh, coarse grained, massive, grey, white/black, strong to very strong GNEISS | | 1 | RUN | | | 273 | | | | | | |
| | | | 2 | RUN | | | 272 | | | | | | |
| | | | 3 | RUN | | | 271 | | | | | | |
| 270.3 | | | | | | | | | | | | | |
| 25.3 | END OF BOREHOLE AT 25.30 m. BOREHOLE OPEN TO 25.30 m. BOREHOLE GROUTED TO SURFACE. | | | | | | | | | | | | |

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 400-5

2 OF 3

METRIC

W.P. 479-93-01 LOCATION N 5 048 593.1 E 316 385.3, Magnetawan River Bridge, E/W-N Ramp ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 23.02.05 - 23.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|------------------------------------|-------------------------------------|-----------------------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | | | | | |
| | | | | | | | 285 | | | | | | | | |
| | | | | | | | 284 | | | | | | | | |
| | | | 5 | SS | 20 | | 283 | | | | | | | | |
| | | | | | | | 282 | | | | | | | | |
| | | | | | | | 281 | | | | | | | | |
| | | | 6 | SS | 11 | | 280 | | | | | | | | |
| | | | | | | | 279 | | | | | | | | |
| | | | | | | | 278 | | | | | | | | |
| 276.8 | | | | | | | 277 | | | | | | | | |
| 18.3 | SILT, trace clay, trace sand Very Dense Grey | | 7 | SS | 55 | | | | | | | | | | 0 2 92 5 |
| 275.6 | | | | | | | 276 | | | | | | | | |
| 19.5 | SAND, trace silt, occasional cobbles Brown Wet | | | | | | | | | | | | | | |

Continued Next Page

+ 3 . x 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

[illegible]

Appendix B

Laboratory Test Results

FIGURE B1

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

| Grain Size (mm) | Percent Finer Than (Open Circles) | Percent Finer Than (Solid Circles) |
|-----------------|-----------------------------------|------------------------------------|
| 100 | 100 | 100 |
| 40 | 100 | 100 |
| 30 | 100 | 100 |
| 16 | 100 | 100 |
| 8 | 100 | 100 |
| 4 | 100 | 100 |
| 2 | 100 | 100 |
| 1 | 100 | 100 |
| 0.85 | 100 | 100 |
| 0.75 | 100 | 100 |
| 0.6 | 100 | 100 |
| 0.425 | 100 | 100 |
| 0.3 | 100 | 100 |
| 0.25 | 100 | 100 |
| 0.2 | 100 | 100 |
| 0.15 | 100 | 100 |
| 0.125 | 100 | 100 |
| 0.106 | 100 | 100 |
| 0.075 | 100 | 100 |
| 0.06 | 100 | 100 |
| 0.05 | 100 | 100 |
| 0.0425 | 100 | 100 |
| 0.0375 | 100 | 100 |
| 0.03 | 100 | 100 |
| 0.025 | 100 | 100 |
| 0.02 | 100 | 100 |
| 0.015 | 100 | 100 |
| 0.0125 | 100 | 100 |
| 0.0106 | 100 | 100 |
| 0.0085 | 100 | 100 |
| 0.0075 | 100 | 100 |
| 0.006 | 100 | 100 |
| 0.00425 | 100 | 100 |
| 0.003 | 100 | 100 |
| 0.0025 | 100 | 100 |
| 0.002 | 100 | 100 |
| 0.0015 | 100 | 100 |
| 0.00125 | 100 | 100 |
| 0.00106 | 100 | 100 |
| 0.00085 | 100 | 100 |
| 0.00075 | 100 | 100 |
| 0.0006 | 100 | 100 |
| 0.000425 | 100 | 100 |
| 0.0003 | 100 | 100 |
| 0.00025 | 100 | 100 |
| 0.0002 | 100 | 100 |
| 0.00015 | 100 | 100 |
| 0.000125 | 100 | 100 |
| 0.000106 | 100 | 100 |
| 0.000085 | 100 | 100 |
| 0.000075 | 100 | 100 |
| 0.00006 | 100 | 100 |
| 0.0000425 | 100 | 100 |
| 0.00003 | 100 | 100 |
| 0.000025 | 100 | 100 |
| 0.00002 | 100 | 100 |
| 0.000015 | 100 | 100 |
| 0.0000125 | 100 | 100 |
| 0.0000106 | 100 | 100 |
| 0.0000085 | 100 | 100 |
| 0.0000075 | 100 | 100 |
| 0.000006 | 100 | 100 |
| 0.00000425 | 100 | 100 |
| 0.000003 | 100 | 100 |
| 0.0000025 | 100 | 100 |
| 0.000002 | 100 | 100 |
| 0.0000015 | 100 | 100 |
| 0.00000125 | 100 | 100 |
| 0.00000106 | 100 | 100 |
| 0.00000085 | 100 | 100 |
| 0.00000075 | 100 | 100 |
| 0.0000006 | 100 | 100 |
| 0.000000425 | 100 | 100 |
| 0.0000003 | 100 | 100 |
| 0.00000025 | 100 | 100 |
| 0.0000002 | 100 | 100 |
| 0.00000015 | 100 | 100 |
| 0.000000125 | 100 | 100 |
| 0.000000106 | 100 | 100 |
| 0.000000085 | 100 | 100 |
| 0.000000075 | 100 | 100 |
| 0.00000006 | 100 | 100 |
| 0.0000000425 | 100 | 100 |
| 0.00000003 | 100 | 100 |
| 0.000000025 | 100 | 100 |
| 0.00000002 | 100 | 100 |
| 0.000000015 | 100 | 100 |
| 0.0000000125 | 100 | 100 |
| 0.0000000106 | 100 | 100 |
| 0.0000000085 | 100 | 100 |
| 0.0000000075 | 100 | 100 |
| 0.000000006 | 100 | 100 |
| 0.00000000425 | 100 | 100 |
| 0.000000003 | 100 | 100 |
| 0.0000000025 | 100 | 100 |
| 0.000000002 | 100 | 100 |
| 0.0000000015 | 100 | 100 |
| 0.00000000125 | 100 | 100 |
| 0.00000000106 | 100 | 100 |
| 0.00000000085 | 100 | 100 |
| 0.0000 | | |

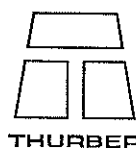
| | | | | | | |
|----------------|--------|------|--------|--------|------|---------------|
| COBBLE SIZE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT and CLAY |
| | GRAVEL | | SAND | | | FINE GRAINED |

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|-------|-----------|-----------|
| ● | 400-1 | 1.06 | 293.77 |
| ☒ | 400-1 | 2.59 | 292.24 |

THURBGSD 2316(396, 400).GPJ 14/10/05

Date October 2005

Project 479-93-01



Prep'dHS.....

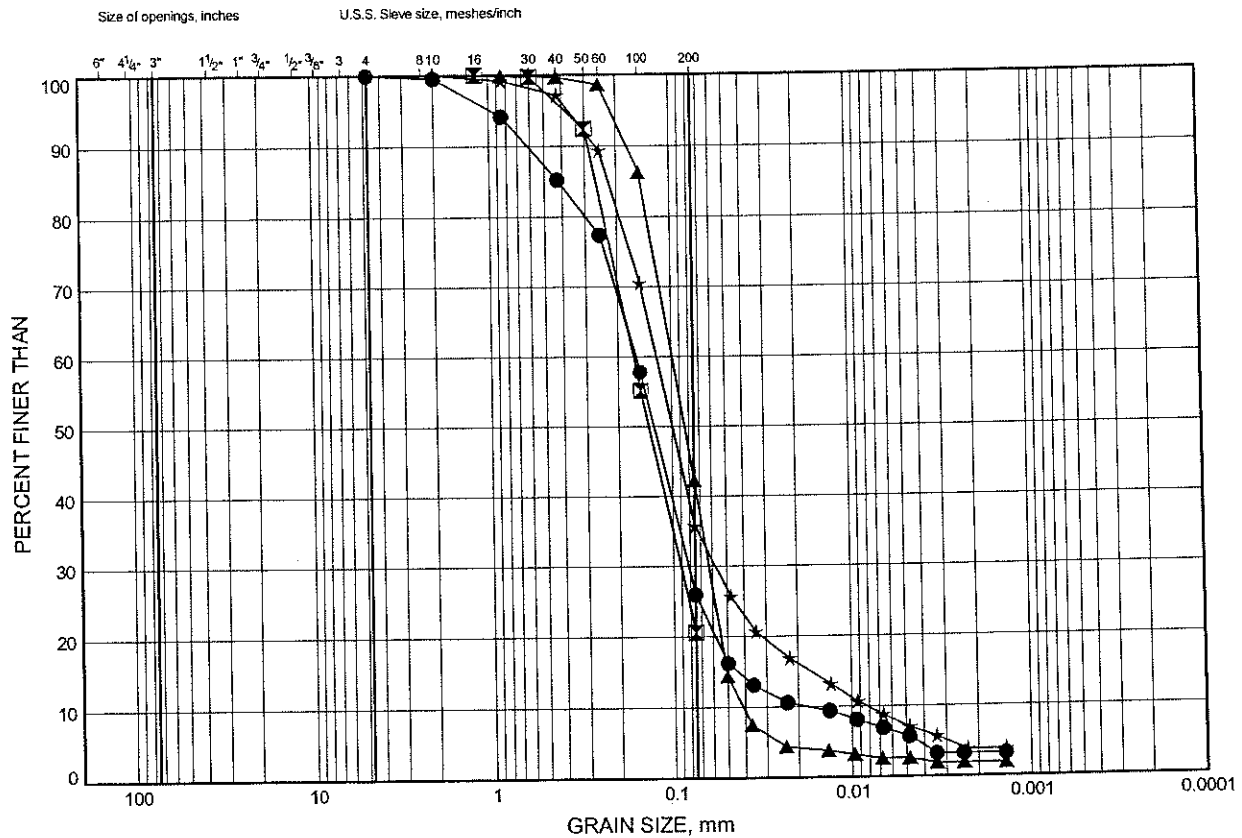
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND, some silt to silty

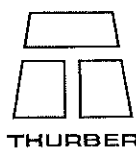


| COBBLE SIZE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT and CLAY |
|----------------|--------|------|--------|--------|------|---------------|
| | GRAVEL | | SAND | | | FINE GRAINED |

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|-------|-----------|-----------|
| ● | 400-1 | 4.88 | 289.95 |
| ⊠ | 400-1 | 9.45 | 285.38 |
| ▲ | 400-1 | 14.95 | 279.88 |
| ★ | 400-2 | 4.88 | 289.92 |

Date October 2005

Project 479-93-01



Prep'd HS

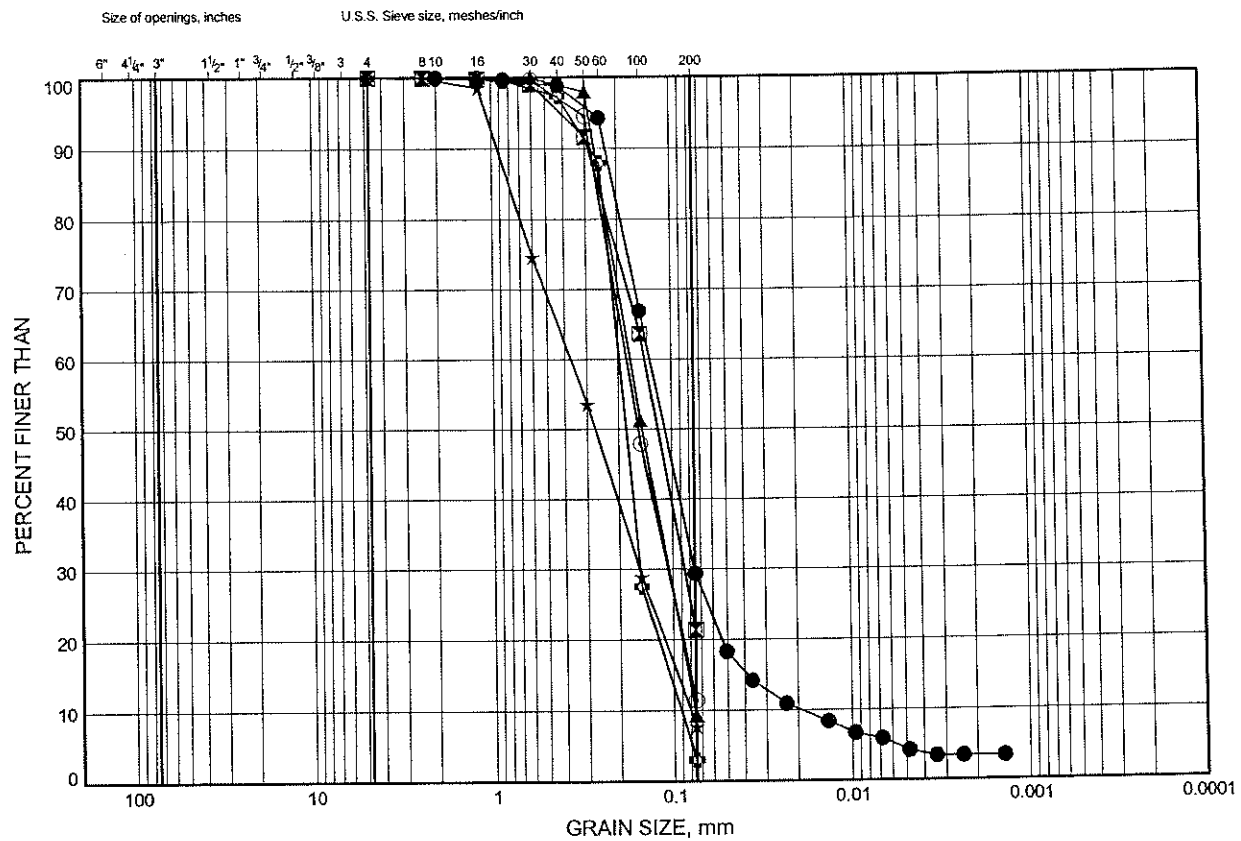
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND, some silt to silty

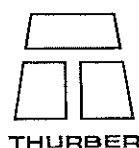


| COBBLE SIZE | COARSE | FINE | COARSE | MEDIUM | FINE | SILT and CLAY |
|----------------|--------|------|--------|--------|------|---------------|
| | GRAVEL | | SAND | | | FINE GRAINED |

| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|--------|-------|-----------|-----------|
| ● | 400-3 | 2.59 | 292.21 |
| ⊠ | 400-3 | 6.40 | 288.40 |
| ▲ | 400-4 | 3.35 | 292.25 |
| ★ | 400-4 | 12.50 | 283.10 |
| ⊙ | 400-4 | 18.59 | 277.01 |
| ⊗ | 400-5 | 9.45 | 285.65 |

Date October 2005

Project 479-93-01



Prep'd HS

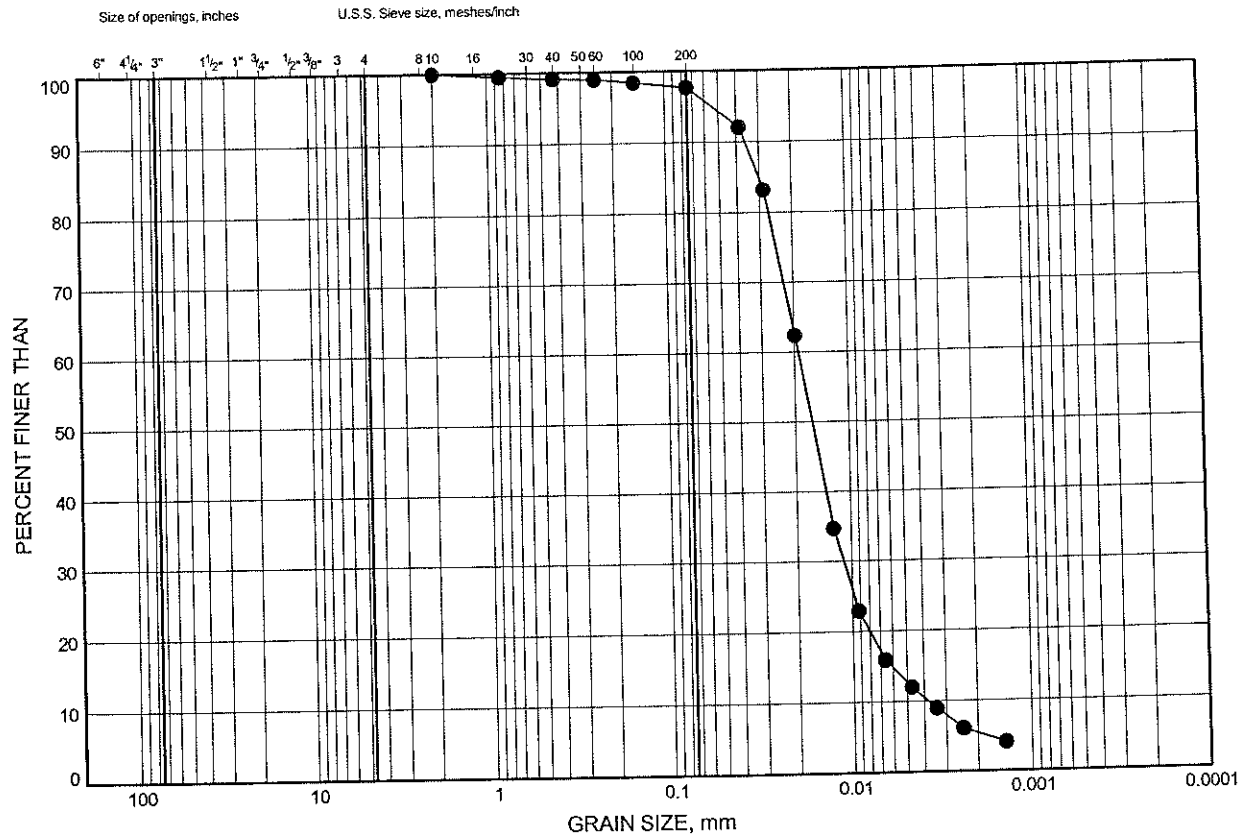
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY



Appendix C

Data From Shaheen & Peaker Report

RECORD OF BOREHOLE No MR1

1 OF 2

METRIC

W.P. 314-99-00 LOCATION Ramp E, W-N Crossing over Magnetawan River Co-ords: N 5 048 558.7; E 316 421.4 ORIGINATED BY G.I.
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers, Casing and wash boring & NQ Rock Core COMPILED BY G.T.
DATUM Geodetic DATE 12.03.01 to 14.03.01 CHECKED BY Z.O.

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W.P. | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W.L. | UNIT WEIGHT Y | 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 | | | | | |
| 295.8 0.0 | Ground Surface | | | | | | | | | | | | | | |
| 295.0 | 125 mm Topsoil SILTY CLAY: with Clayey Silt & Silt seams, grey, moist | | 1 | SS | 9 | | 295 | | | | | | | | partially frozen |
| 0.8 | SANDY SILT: loose, grey, wet | | 2 | SS | 9 | | | | | | | | | | 0 37 62 1 |
| 294.4 | | | | | | | | | | | | | | | |
| 1.4 | | | 3 | SS | 7 | | 294 | | | | | | | | |
| | | | 4 | SS | 4 | | | | | | | | | | |
| | Silty to 3.0 m | | 5 | SS | 2 | | 293 | | | | | | | | |
| | | | 6 | SS | 3 | | 292 | | | | | | | | |
| | FINE SAND some silt, occasional silty sand/ sandy silt zones, wet | | 7 | SS | 3 | | 291 | | | | | | | | |
| | | | 8 | SS | 6 | | 290 | | | | | | | | |
| | grey | | 9 | SS | 11 | | 289 | | | | | | | | commenced washboring |
| | ----- | | 10 | SS | 7 | | 288 | | | | | | | | |
| | brown | | 11 | SS | 17 | | 287 | | | | | | | | |
| | | | | | | | 286 | | | | | | | | |
| | loose | | 12 | SS | 10 | | 285 | | | | | | | | |
| | ----- | | | | | | 284 | | | | | | | | |
| | compact | | 13 | SS | 30 | | 283 | | | | | | | | |
| | | | 14 | SS | 20 | | 282 | | | | | | | | 0 77 21 2 |
| | | | | | | | 281 | | | | | | | | |
| 280.8 | | | 15 | SS | 26 | | | | | | | | | | |
| | brown | | | | | | | | | | | | | | |
| | ----- | | | | | | | | | | | | | | |
| | grey | | | | | | | | | | | | | | |

15.0 Continued Next Page

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

RECORD OF BOREHOLE No MR1

2 OF 2

METRIC

W.P. 314-99-00 LOCATION Ramp E, W-N Crossing over Magnetawan River Co-ords: N 5 048 558.7; E 316 421.4
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers, Casing and wash boring & NQ Rock Core
DATUM Geodetic DATE 12.03.01 to 14.03.01
ORIGINATED BY G.I.
COMPILED BY G.T.
CHECKED BY Z.O.

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | | | |
|----------------|--|------------|---------|----------|--------------|----------------------------|-----------------|--|----|----|------------------------------------|-------------------------------------|-----------------------------------|--|--|-------------------|--|--|--|--|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | WATER CONTENT (%) | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE | | | | | | | | | | | | |
| 280.8 | | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | | | | | | |
| 15.0 | FINE SAND trace to some silt, occasional sandy silt zones, dense to very dense brown/gray, wet | | 16 | SS | 33 | | | | | | | ○ | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 17 | SS | 30 | | | | | | | ○ | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 18 | SS | 53 | | | | | | | ○ | | | | | | | | |
| 276.3 | | | | | | | | | | | | | | | | | | | | |
| 19.5 | SILT trace clay and sand, firm, grey, wet | | 19 | SS | 7 | | | | | | | ○ | | | 0 7 83 10 | | | | | |
| 274.8 | | | | | | | | | | | | | | | | | | | | |
| 21.0 | FINE SAND dense | | 20 | SS | 34 | | | | | | | ○ | | | | | | | | |
| 273.4 | frequent cobbles below 22.0 m | | | | | | | | | | | | | | | | | | | |
| 22.4 | GNEISS BEDROCK grey, unweathered | | 21 | NQ RC | Rec. 100% | | | | | | | | | | RQD=100% | | | | | |
| | | | 22 | NQ RC | Rec. 100% | | | | | | | | | | RQD=100% | | | | | |
| | | | 23 | NQ RC | Rec. 100% | | | | | | | | | | RQD=100% | | | | | |
| 270.2 | | | | | | | | | | | | | | | | | | | | |
| 25.6 | End of borehole Piezometer installed at 20.0 m, upon completion Water level in piezometer at: Mar.14/2001 - 0.9 m Mar.15/2001 - 0.75 m Mar.16/2001 - 0.75 m Mar.19/2001 - 0.70 m Apr. 02/2001 - 0.70 m Apr. 04/2001 - 0.65 m Apr. 06/2001 - 0.65 m Apr. 09/2001 - 0.90 m Apr. 11/2001 - 0.30 m | | | | | | | | | | | | | | | | | | | |

+ 3, X 3: Numbers refer to Sensitivity

| RECORD OF BOREHOLE No MR2 | | | | | | | | | | 2 OF 2 | | METRIC | |
|---------------------------|--------------------------------------|---|--------|------|----------------------------|-------------------|---|--------------------|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| W.P. 314-99-00 | | LOCATION Ramp E, W-N Crossing over Magnetawan River Co-ords: N 5 048 560.1; E 316 416.9 | | | | ORIGINATED BY G.I | | | | | | | |
| DIST 52 HWY 11 | | BOREHOLE TYPE Dynamic Cone Penetration Test | | | | COMPILED BY G.T | | | | | | | |
| DATUM Geodetic | | DATE 09.03.01 | | | | CHECKED BY Z.O | | | | | | | |
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | |
| 279.6 | | | | | | | 20 40 60 80 100 | | 20 40 60 | | | | |
| 15.0 | | | | | | | 20 40 60 80 100 | | 20 40 60 | | | | |
| 279.0 | | | | | | | 20 40 60 80 100 | | 20 40 60 | | | | |
| 15.6 | End of Dynamic Cone Penetration Test | | | | | 279 | 20 40 60 80 100 | | 20 40 60 | | | | |

RECORD OF BOREHOLE No MR3

1 OF 2

METRIC

W.P. 314-99-00 LOCATION Ramp E, W-N Crossing over Magnetawan River Co-ords: N 5 048 553.3; E 316 435.3 ORIGINATED BY G.I
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers & D.C.P.T. COMPILED BY G.T
DATUM Geodetic DATE 09.03.01 CHECKED BY Z.O

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|--------|------|----------------------------|-----------------|---|-----------------|--|---|----------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | SHEAR STRENGTH kPa | | W _p | W | W _L | | |
| 299.9 | Ground Surface | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | | | | | |
| 0.0 | 125 mm Topsoil | | 1 | SS | 5 | | | | | | | 19.2 | |
| | brown | | | | | | | | | | | | |
| | grey | | 2 | SS | 8 | | | | | | | 18.8 | 0 10 55 35 |
| | SLTY CLAY: layered, occasional rootlets to 0.8 m, layered, firm to very stiff | | 3 | SS | 7 | | | | | | | 17.9 | |
| | | | 4 | SS | 5 | | | | | | | 17.1 | 0 20 64 16 |
| | | | 5 | TW | PH | | | | | | | | |
| | | | 6 | SS | 10 | | | | | | | 18.7 | |
| 295.3 | | | | | | | | | | | | | |
| 4.6 | SANDY SILT layered, very loose to loose grey, wet | | 7 | SS | 10 | | | | | | | | |
| | | | 8 | SS | 3 | | | | | | | | 0 28 69 3 |
| 293.8 | | | | | | | | | | | | | Started to use drilling mud |
| 6.1 | | | 9 | SS | 8 | | | | | | | | |
| | FINE SAND some silt, loose to very loose, grey, wet | | | | | | | | | | | | |
| | | | 10 | SS | 2 | | | | | | | | |
| | | | | | | | | | | | | | |
| 290.3 | | | 11 | SS | 0 | | | | | | | | |
| 9.6 | End of borehole *Water level at 5.6 m (not stabilized) on completion Dynamic Cone Penetration Test (DCPT) performed from 9.8 m to 18.3 m | | | | | | | | | | | | |
| 284.9 | | | | | | | | | | | | | |

15.0 Continued Next Page

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

RECORD OF BOREHOLE No MR3

2 OF 2

METRIC

W.P. 314-99-00 LOCATION Ramp E, W-N Crossing over Magnetawan River Co-ords: N 5 048 553.3; E 316 435.3
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers & D.C.P.T. ORIGINATED BY G.I.
DATUM Geodetic DATE 09.03.01 & COMPILED BY G.T.
CHECKED BY Z.O.

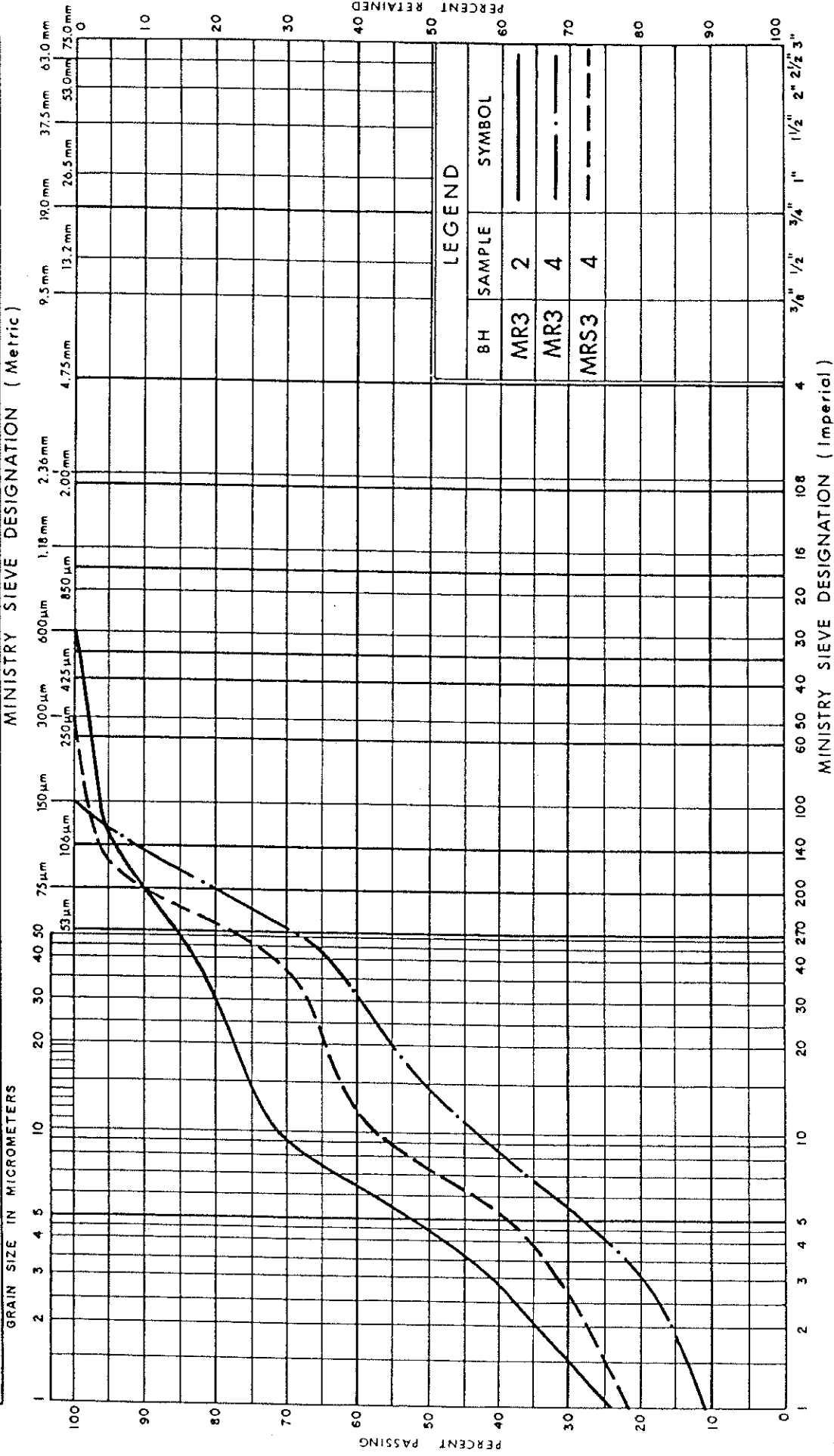
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | | | | | |
| 284.9 | | | | | | | | | | | | | | |
| 15.0 | | | | | | | | | | | | | | |
| 281.6 | | | | | | | | | | | | | | |
| 18.3 | End of Dynamic Cone Penetration Test | | | | | | | | | | | | | |

+³ ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM

| CLAY & SILT | | SAND | | | GRAVEL | | |
|-------------|--|------|--------|--------|--------|--------|--|
| | | Fine | Medium | Coarse | Fine | Coarse | |



GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIG No 1

W P 314-99-00

SPT 1010B

Ministry of
Transportation



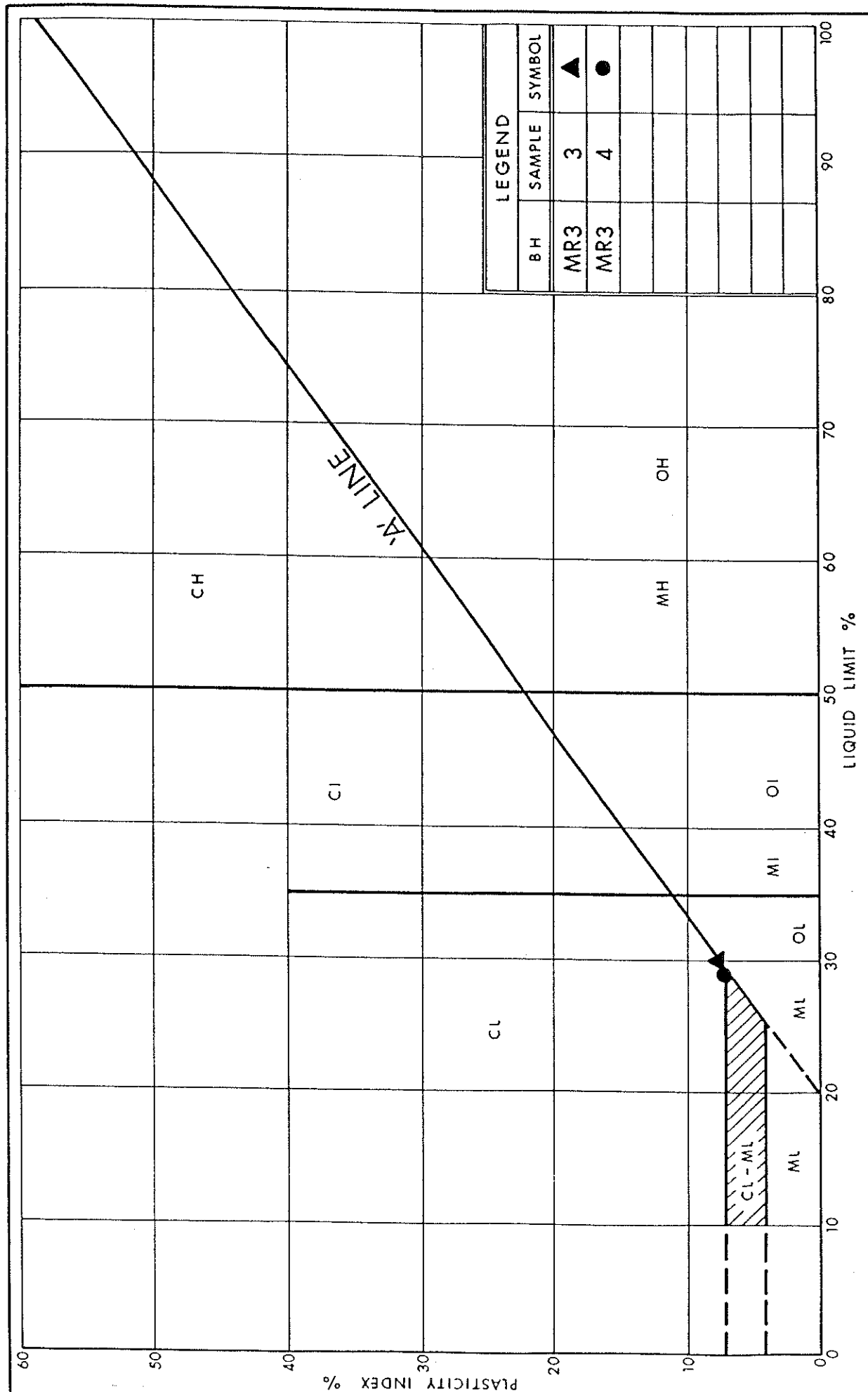


FIG No 2

PLASTICITY CHART SILTY CLAY

Ministry of
Transportation



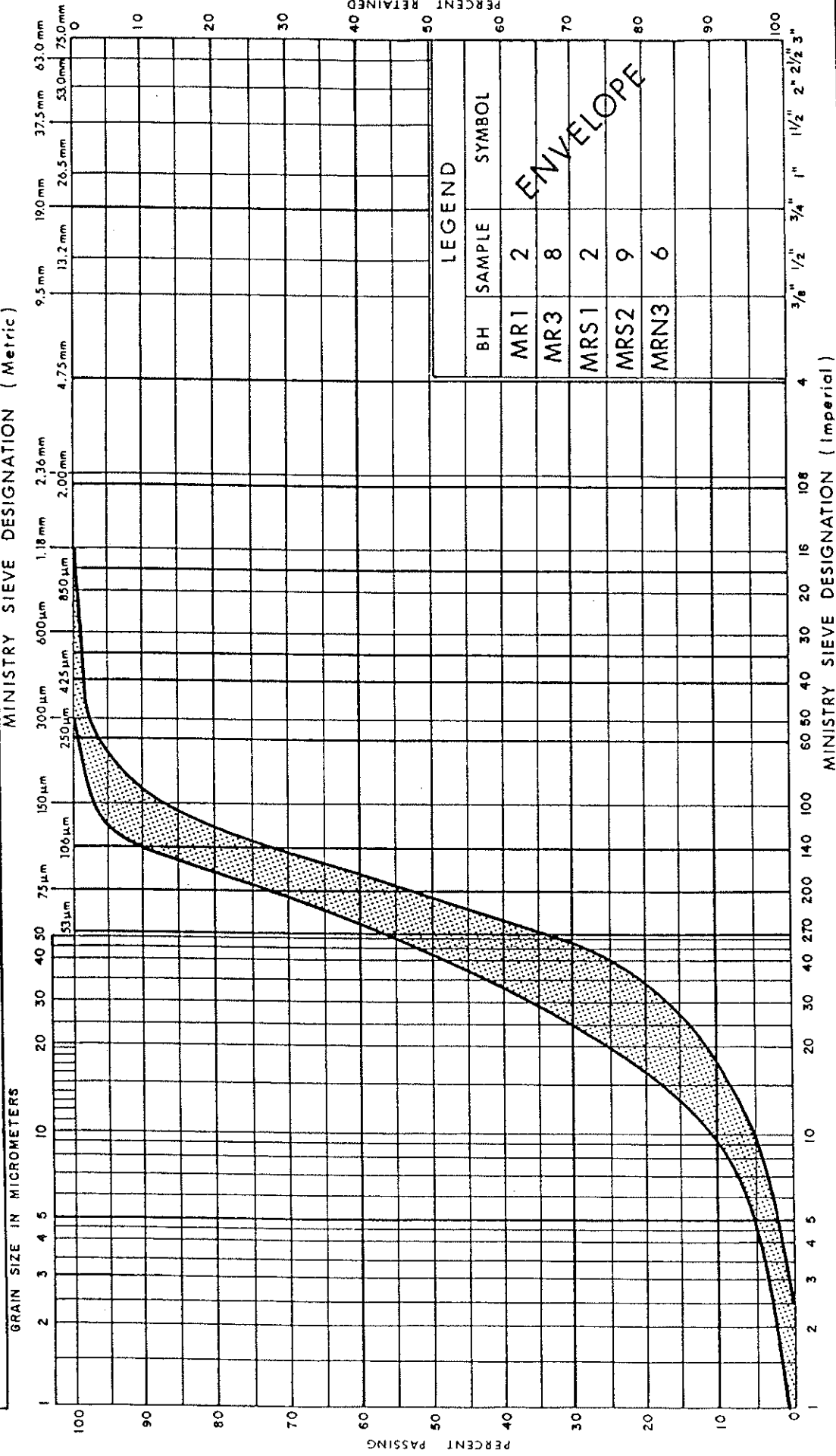
W P 314-99-00

SPT 1010B

UNIFIED SOIL CLASSIFICATION SYSTEM

| CLAY & SILT | | SAND | | | GRAVEL | | |
|-------------|--|------|--------|--------|--------|--------|--|
| | | Fine | Medium | Coarse | Fine | Coarse | |

GRAIN SIZE IN MICROMETERS



GRAIN SIZE DISTRIBUTION

SANDY SILT TO SILTY SAND

FIG No 4

W P 314-99-00

SPT 1010B

Ministry of
Transportation



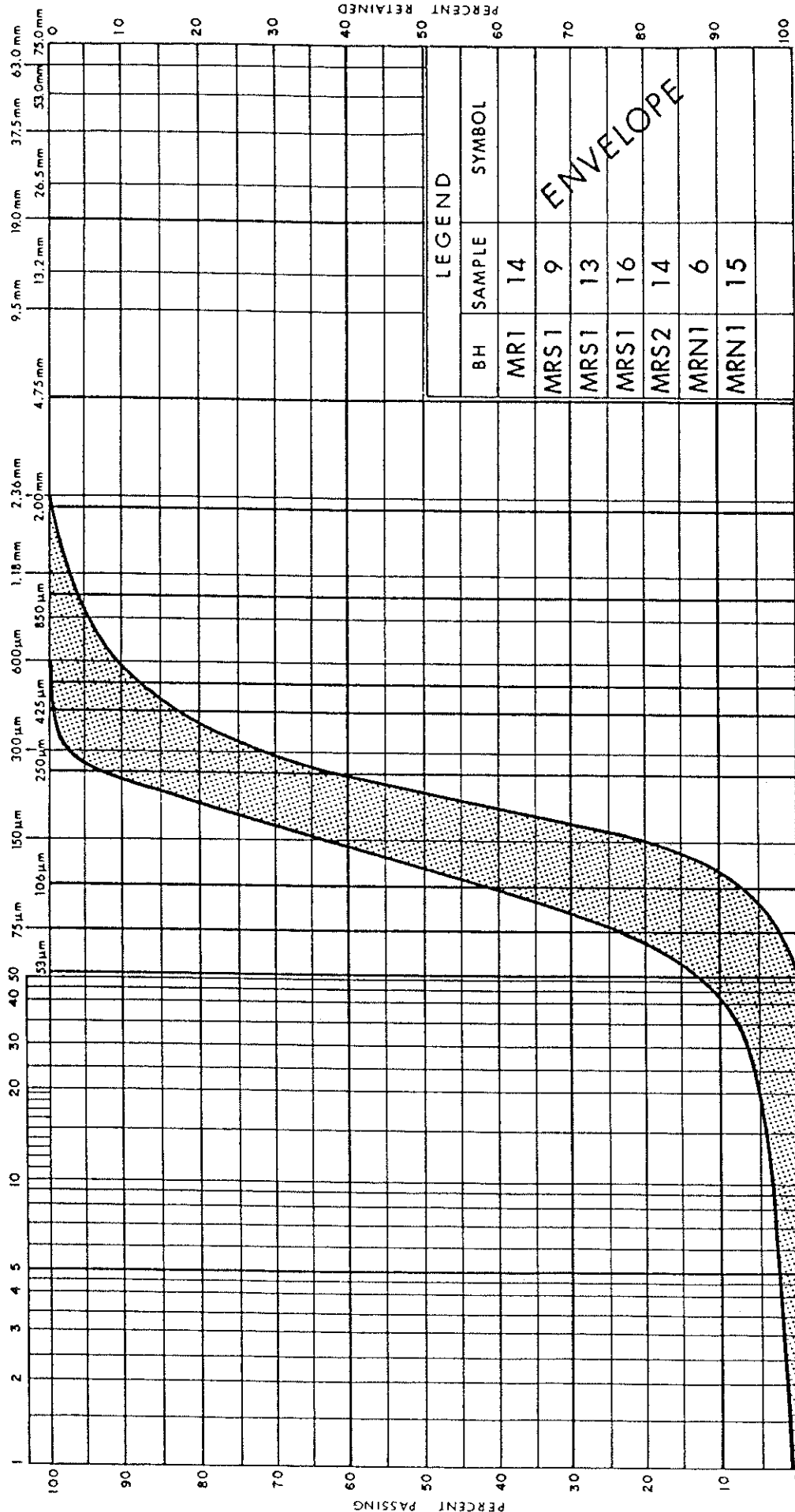
Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM

| CLAY & SILT | | SAND | | | GRAVEL | | |
|-------------|--|------|--------|--------|--------|--------|--|
| | | Fine | Medium | Coarse | Fine | Coarse | |

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

FINE SAND

FIG No 6

W P 314-99-00

SPT 1010B

Ministry of
Transportation



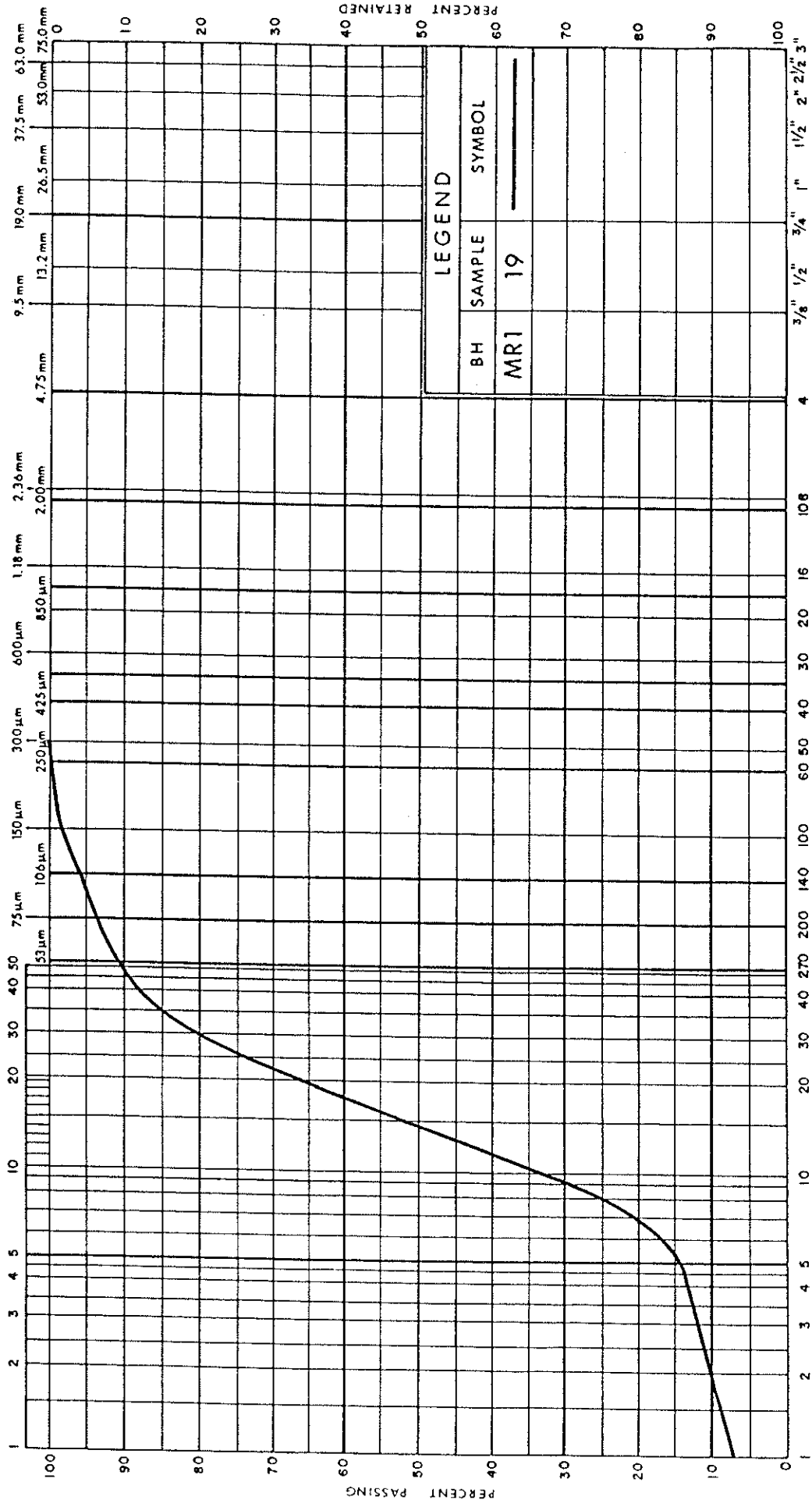
Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM

| | | | | | | | |
|-------------|--|------|--------|--------|--------|--------|--|
| CLAY & SILT | | SAND | | | GRAVEL | | |
| | | Fine | Medium | Coarse | Fine | Coarse | |

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION

SILT

FIG No 9

WP 314-99-00

SPT 1010B

Ministry of
Transportation



Ontario

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

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

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
- Standard Special Provision No. 105S10

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

“ The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 277. 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
 Hwy11 - Katrine
 December 2003
 North Crossing - EW-N Ramp - South Bank
 Setback from May 2003 Water Level: 7.5m

| | Gamma C | Phi | Piezo |
|-------------|---------------------|-----|-----------|
| | kN/m3 | kPa | deg Surf. |
| Water | 9.81 | 0 | 0 |
| Backfill | 21 | 0 | 30 |
| Rockfill | 20 | 0 | 42 |
| Fine Sand | 19 | 0 | 28 |
| Fine Sand | 20 | 0 | 30 |
| Fine Sand | 21 | 0 | 30 |
| Hard Bottom | (Infinitely Strong) | | |

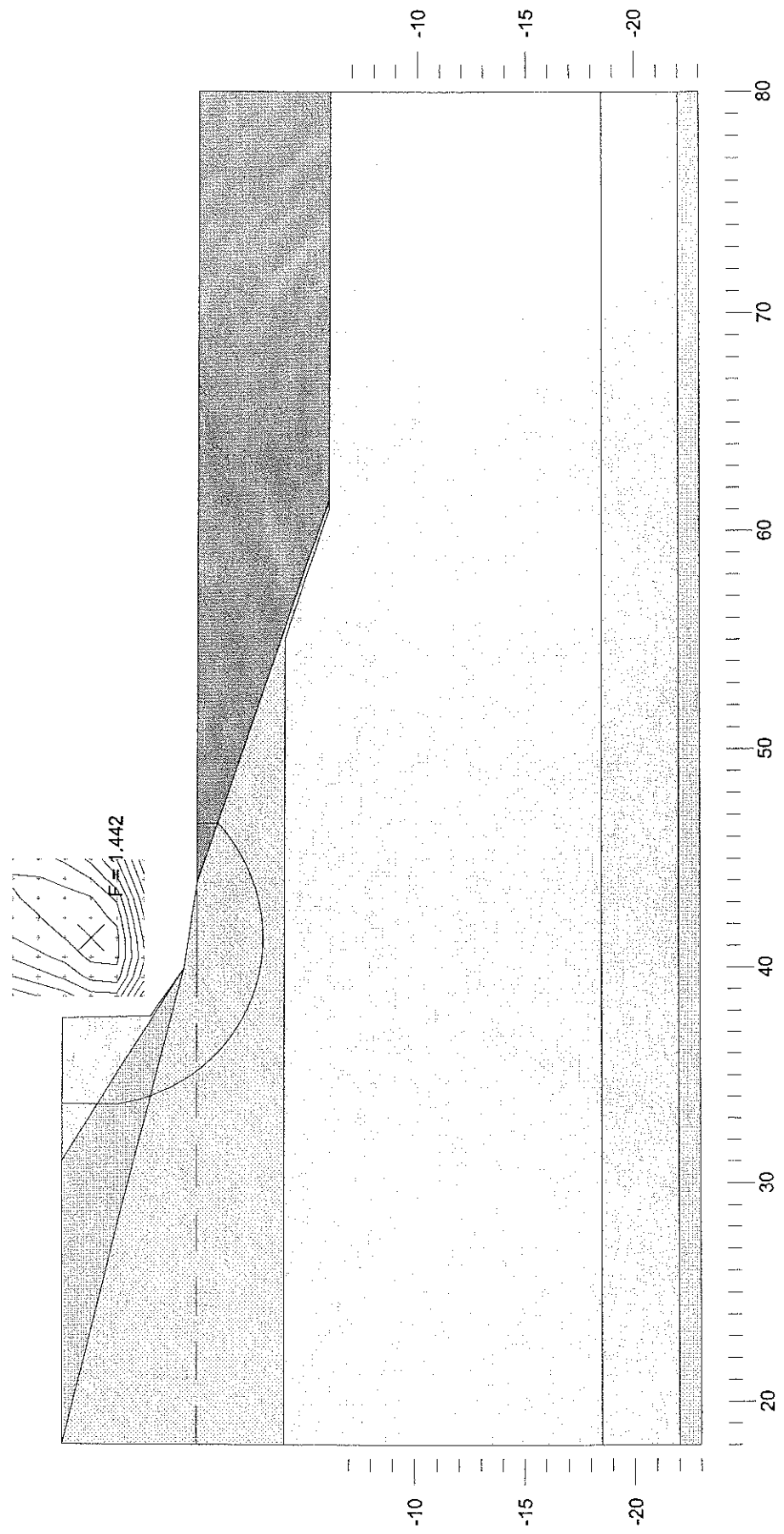


Figure 1

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy11 - Katrine
 October 15 2004
 North Crossing - Ramp - South Bank
 Lateral stability

| | Gamma C | Phi | Piezo |
|-----------------|---------------------|-----|-----------|
| | kN/m3 | kPa | deg Surf. |
| Rockfill | 20 | 0 | 42 |
| Silty Sand | 19 | 0 | 28 |
| Fine Sand | 20 | 0 | 30 |
| Gravel and Sand | 21 | 0 | 32 |
| Hard Bottom | (Infinitely Strong) | | |

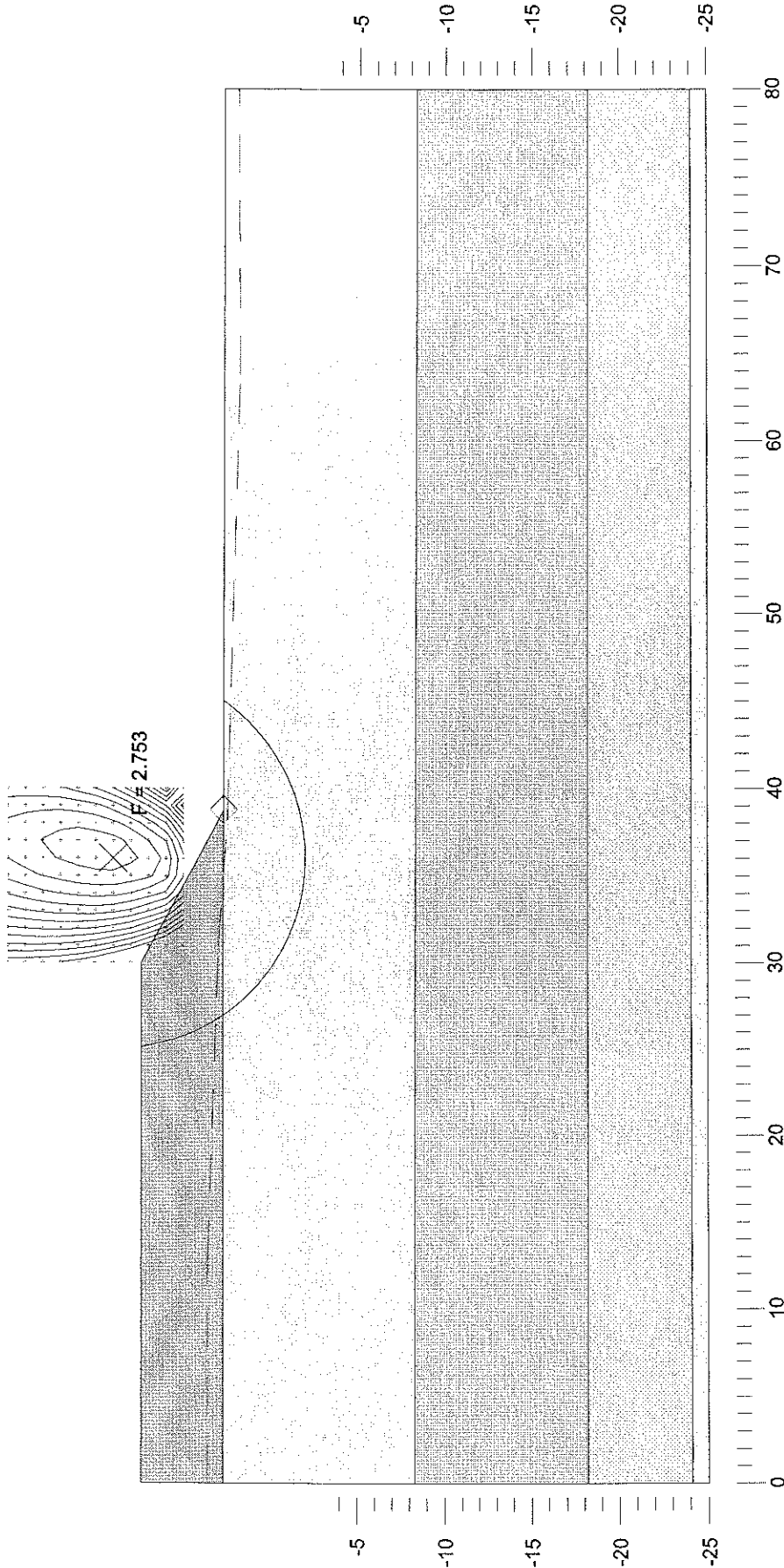


Figure 2

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy11 - Katrine
 December 2003
 North Crossing - EW-N Ramp - North Bank
 Setback from May 2003 Water Level: 8.5m

| | Gamma C | Phi | Piezo |
|-------------|---------------------|-----|-------|
| | kN/m3 | kPa | deg |
| Water | 9.81 | 0 | 0 |
| Backfill | 21 | 0 | 30 |
| Rockfill | 20 | 0 | 42 |
| Fine Sand | 19 | 0 | 28 |
| Fine Sand | 20 | 0 | 30 |
| Fine Sand | 21 | 0 | 30 |
| Hard Bottom | (Infinitely Strong) | | |

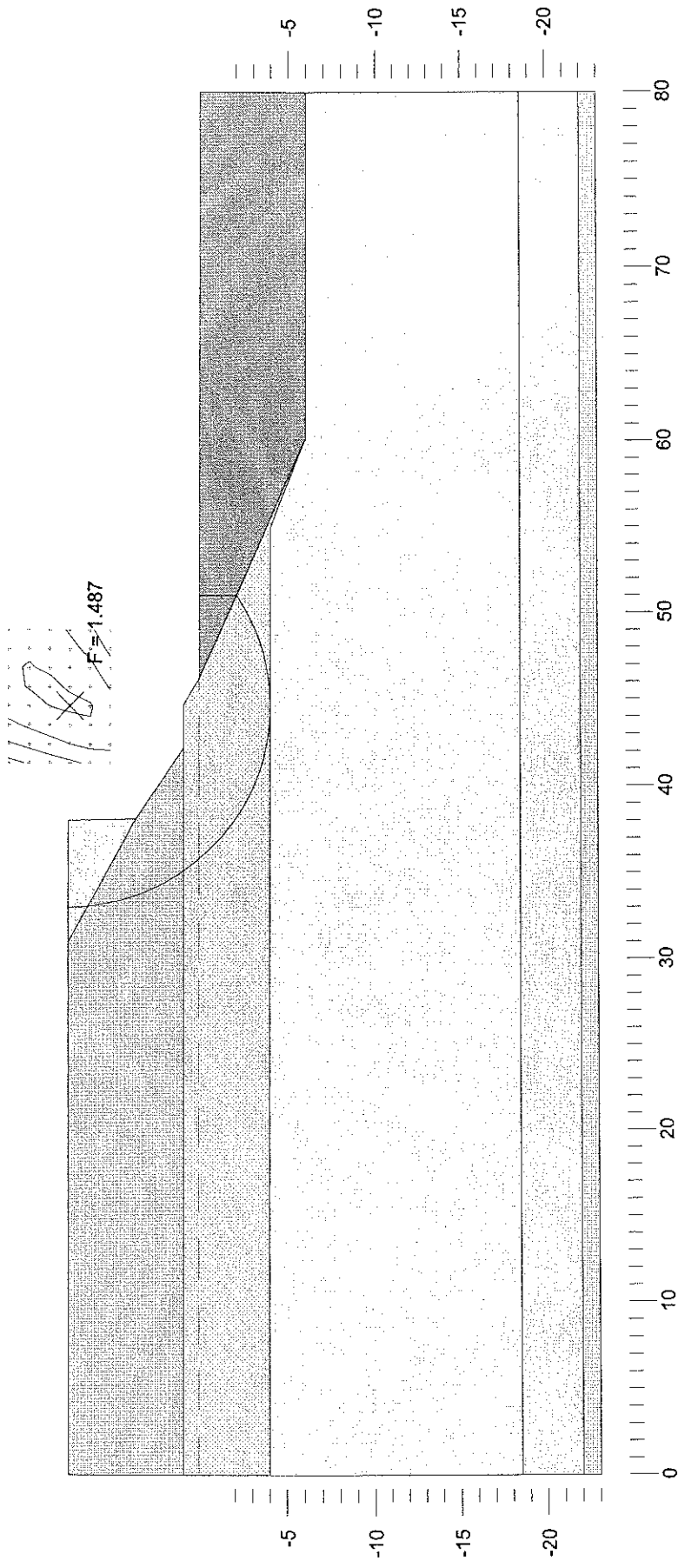


Figure 3

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy11 - Katrine
 October 15 2004
 North Crossing - Ramp - North Bank
 Lateral stability

| | Gamma C | Phi | Piezo |
|-----------------|---------------------|-----|-----------|
| | kN/m3 | kPa | deg Surf. |
| Rockfill | 20 | 0 | 42 |
| Silty Sand | 19 | 0 | 28 |
| Fine Sand | 20 | 0 | 30 |
| Gravel and Sand | 21 | 0 | 32 |
| Hard Bottom | (Infinitely Strong) | | |

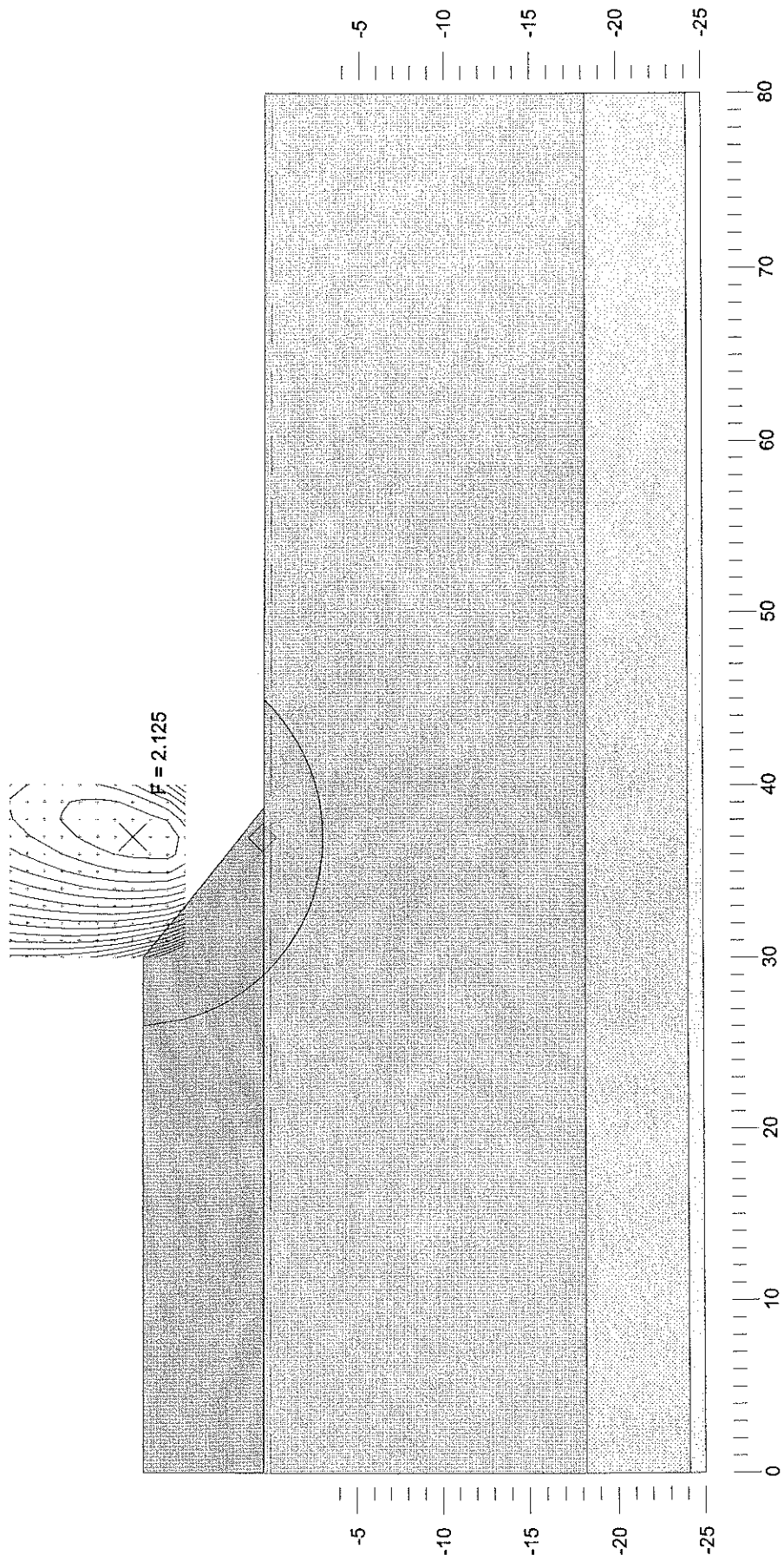
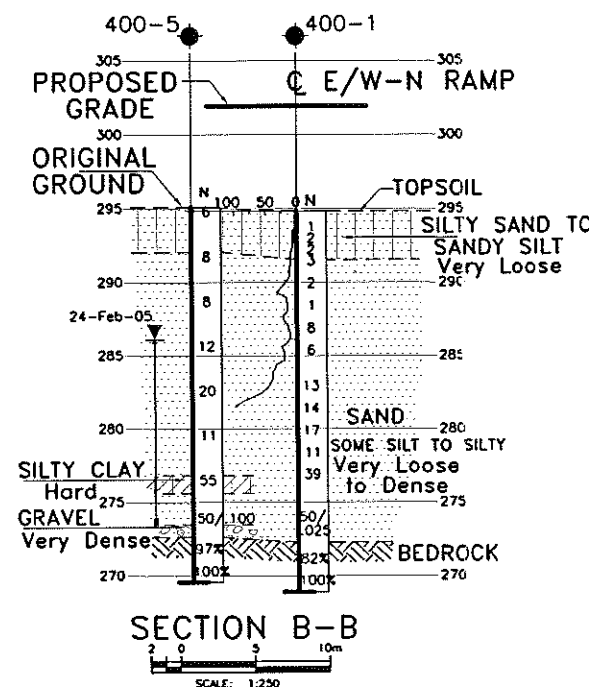
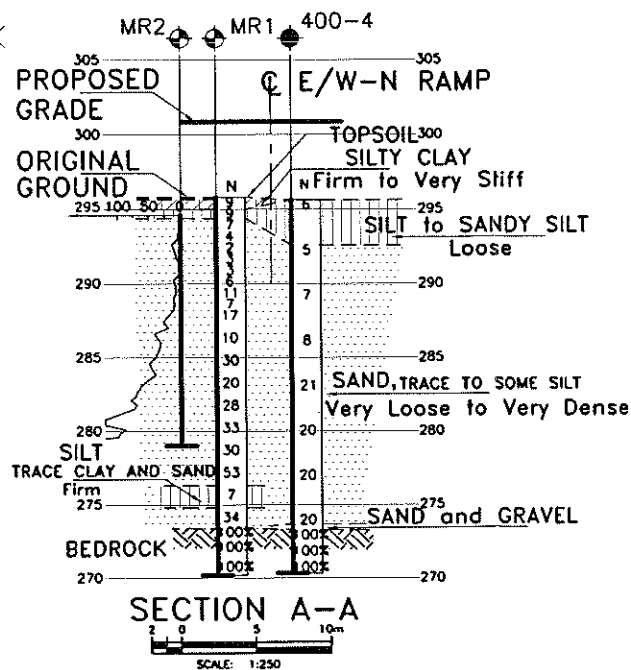
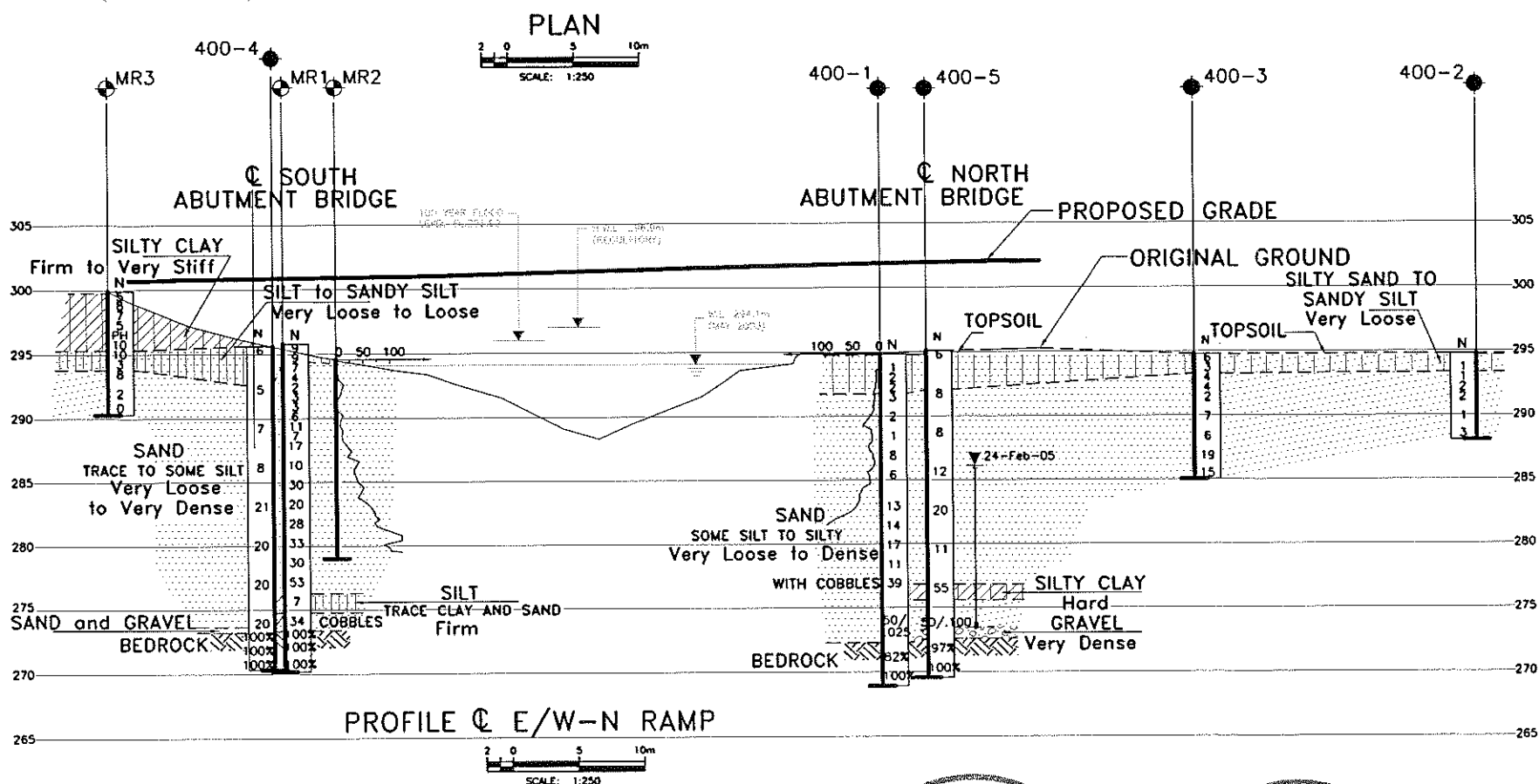
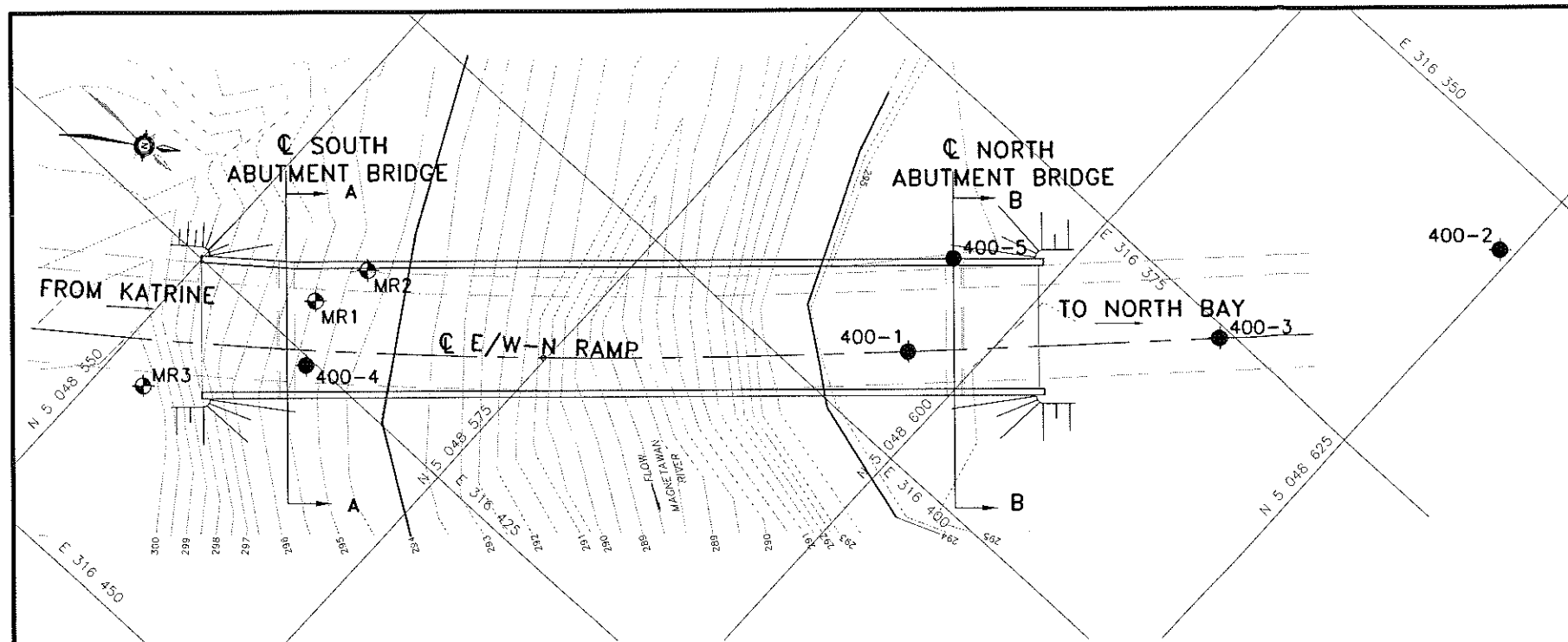


Figure 4

Appendix G

Borehole Locations and Soil Strata



HWY 11
CONT No 2006-5148
WP No 479 93 01

MAGNETAWAN RIVER BRIDGE
E/W-N RAMP
BOREHOLE LOCATIONS AND SOIL STRATA

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.

SHEET
801

KEYPLAN

| LEGEND | | | |
|--------|--------------------------------------|-------------|-----------|
| ● | BoreHole by THURBER | | |
| ⊕ | Dynamic Cone penetration Test (cone) | | |
| ⊗ | BoreHole by SHAHEEN & PEAKER LIMITED | | |
| N | Blow/0.3m (Std Pen Test, 475 J/blow) | | |
| CONE | Blows/0.3m (60' Cone, 475 J/blow) | | |
| PH | Pressure, Hydraulic | | |
| WL | Head Artesian Water | | |
| ⊥ | Piezometer | | |
| 90% | Rock Quality Designation (ROD) | | |
| NO | ELEVATION | NORTHING | EASTING |
| MR1 | 295.8 | 5 048 558.7 | 316 421.4 |
| MR2 | 294.6 | 5 048 560.1 | 316 416.9 |
| MR3 | 299.9 | 5 048 553.3 | 316 435.3 |
| 400-1 | 294.8 | 5 048 595.4 | 316 392.9 |
| 400-2 | 294.8 | 5 048 623.9 | 316 356.0 |
| 400-3 | 294.8 | 5 048 612.4 | 316 375.8 |
| 400-4 | 295.6 | 5 048 561.5 | 316 425.6 |
| 400-5 | 295.1 | 5 048 593.1 | 316 385.3 |

-- NOTE --

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

DHO BENCHMARK 333-67
EL. 307.808
TABLET SET VERT. IN ROCK CUT
21.3 LT 2.4km N OF HWY 518
40.057 LT 10+706.020



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

| REVISIONS | DATE | BY | DESCRIPTION |
|------------|---------|-----------------|-----------------|
| DESIGN AEG | CHK PKC | CODE CHBDC 2000 | LOAD 01-625-001 |
| DRAWN SS | CHK AEG | SITE 44-400 | STRUCT. SCHEME |

DATE JUNE 2004
DWG 2