

REVISION 1
FOUNDATION INVESTIGATION AND DESIGN REPORT
MAGNETAWAN RIVER NORTH CROSSING, NBL
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 477-93-01, SITE 44-396N

Geocres Number: 31E-214

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 North Bound Lanes of 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 lies across 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 160 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 45 m wide and the maximum channel depth, based on May 2003 data, is 4 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 21 and December 4, 2003. Investigation was carried out on the south bank between March 6 and March 20, 2001, as part of the preliminary investigation by S&P. Thurber returned to the site to conduct further investigation between January 19, 2005, and February 15, 2005.

The current site investigation consisted of drilling and sampling four boreholes (Boreholes 396N-1 to 396N-4) to depths between 6.7 and 26.1 m. Borehole 396N-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.

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

All-Terrain Drilling supplied and operated the drilling and sampling equipment used for the current investigation. 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 | 396N-2 |
| North Abutment | 396N-1, 396N-3 |
| South Abutment | 396N-4, MRN 1*, MRN 1A*, MRN 2* |
| South Approach | MRN 3* |

* Boreholes drilled for preliminary investigation 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. Piezometers were installed at the south abutment in the course of the preliminary investigation.

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

Table 3.2 – Piezometer Details

| Piezometer Location | Piezometer Details | |
|---------------------|-------------------------|---|
| | Tip Depth/ Elevation | Completion Details |
| BH 396N-1 | 25.8/269.0 | Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 22.9, bentonite seal to 22.6, grout to 0.6 and bentonite seal to the surface. |
| BH 396N-3 | 26.1/268.9 | Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 22.6, bentonite seal to 21.6, grout to the surface. |

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

4 LABORATORY TESTING

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

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

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; fill; discontinuous layers of silty clay; silt and silty sand; a major sand unit; gravelly sand, cobbles and boulders; and bedrock.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil and Organic Silt

Topsoil was identified surficially in the boreholes drilled on the north side of Magnetawan River and in one borehole at the south abutment. Topsoil thicknesses were established only at the borehole locations and ranged from 100 to 150 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

A 300 mm thick layer of organic silt was revealed below possible fill in borehole MRN1 at the south abutment.

5.3 Fill

Some fill may be encountered overlying the topsoil/organic material in the area of the south approach, particularly under the driveways of the campground. Probable fill encountered in one borehole at the south abutment consisted of very loose ($N=2$), wet ($w=29\%$) silt to 1.0 m depth. In the borehole at the south approach, the fill consisted of a thin layer of sand and gravel.

5.4 Silty Clay

A layered stiff silty clay unit was encountered in one borehole (MRN3) at the south approach only. This unit was 0.8 m thick. A standard penetration test N value of 11 blows per 300 mm penetration and moisture content of 29% were measured in this layer.

5.5 Silts and Silty Sands

Silt with clayey silt seams/layers was encountered below the topsoil and silty clay layers in two boreholes (MRN2 and MRN3) drilled at the south approach and abutment. The consistency of this deposit varied from soft to stiff. The silt became very loose and sandy below 3.5 m depth at the south approach.

A 0.5 m thick layer of silty sand was penetrated in the north approach hole (BH 396N-2). Very loose to loose silty sand with organics was revealed below the organic layer in one borehole (MRN1) at the south abutment. This unit was underlain by a 0.7 m thick layer of very loose silt.

SPT values of 2 to 8 blows per 300 mm penetration were obtained in the silt and silty sand layers. Moisture contents ranged from 20 to 30% with one determination of 40%.

The silt and silty sand deposits extend to depths of 2.1 to 4.5 m (Elevation 290.0 to 293.2) on the south side of Magnetawan River, and to 0.6 m depth (Elevation 293.9) at the north approach. The silt/silty sand was not identified in the north abutment borehole.

Grain size distributions charts for the silts, prepared during the preliminary investigation, are included in Appendix C.

5.6 Sand

The silt and 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 depths between 18.3 and 20.6 m (Elevation 276.1 and 274.7). At the south approach, the sand was not fully penetrated at the borehole termination depth of 9.6 m (Elevation 287.4). At the north abutment, the sand extends to depths between 17.5 and 20.7 m (Elevation 277.3 and 274.3). At the north approach, the sand was not fully penetrated but extends at least to the borehole termination depth of 6.7 m (Elevation 287.8).

SPT values measured in the sand ranged from 0 (disturbed) to 10 blows per 0.3 m of penetration to depths of about 8.5 to 9.0 m (approximate elevations 285.0 to 286.5), indicating a very loose to loose condition. Below this level, the sand was compact, with SPT values ranging from 10 to 25 blows per 0.3 m of penetration. Dynamic cone penetration testing generally reflects a similar trend.

The measured natural moisture contents ranged from 20 to 30% with some higher values 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 Figure B1 in Appendix B. Grain size results from the preliminary study are included in Appendix C as well.

5.7 Silt

A layer of dense silt was encountered below the sand in one borehole at the north abutment only. This layer was revealed in one split spoon sample only, and extended to 18.8 m depth (Elevation 276.0). The results of a grain size distribution analysis conducted on the sandy silt are shown on Figure B2 in Appendix B.

5.8 Gravelly Sand, Cobbles and Boulders

Underlying the sand unit and the isolated silt layer, a deposit described as gravelly sand with boulders at the north abutment and as cobbles and boulders at the south abutment was encountered. Due to the frequency of the cobbles and boulders, only one SPT value was obtained in this layer, resulting in 50 blows for less than 150 mm of penetration. Rock

coring methods were necessary to penetrate this deposit in one borehole at the south abutment.

At the south abutment, the cobbles and boulders extended to depths of 22.3 and 23.9 m (Elevations 272.8 and 271.4). At the north abutment, the gravelly sand with boulders extended to 22.1 to 21.5 m depth (Elevation 272.9 to 272.7).

Sample recovery was extremely limited due to the coarse nature of this deposit. Representative laboratory testing of recovered samples was therefore not possible.

5.9 Bedrock

The soils described above were found to be underlain by bedrock of the Pre-Cambrian Canadian Shield. The bedrock was proved by coring 3.0 m at the south abutment and 3.7 m at the north abutment.

The rock is described as grey gneiss at the south abutment and as granitic gneiss at the north abutment.

The core recovery was 77 to 100% at the south abutment and 95 to 99% at the north abutment. A recovery of 50% measured for the first run entering the bedrock at one location at the south approach included partial coring through the overlying cobbles and boulders. RQD values generally ranged from 70 to 97%, indicating a fair to excellent quality rock. In the upper 1.6 m of bedrock core from one borehole at the south abutment, lower RQD values of 22 to 42% were obtained, indicating a poor quality rock.

Based on Point Load Testing, the unconfined compressive strength of the bedrock at the north abutment was estimated to range from 133 to 150 MPa. Based on these strength values and the classification system given in the Canadian Foundation Engineering Manual, the rock was classified as very strong.

5.10 Depths to Refusal

The depths at which effective refusal was encountered, defined as an SPT value exceeding 100 blows for 0.3 m of penetration or bedrock, are shown in Table 5.1. At Borehole MRN1, the drill casings broke when extended from 21.3 to 24.4 m; the borehole was redrilled as Borehole MRN1A and coring was necessary to penetrate cobbles and boulders below the refusal depth shown on the table.

Table 5.1 – Refusal Depths (Elevations)

| Location | Borehole | Refusal Depth (Elevation), m | Material |
|----------------|----------|------------------------------|----------------------|
| North Abutment | 396N-1 | 22.1 (272.7) | Bedrock |
| | 396N-3 | 21.5 (273.5) | Cobbles and Boulders |
| | | 22.3 (272.8) | Bedrock |
| South Abutment | MRN1A | 18.5 (275.4) | Cobbles and Boulders |
| | | 22.3 (271.6) | Bedrock |
| | 396N-4 | 21.5 (272.9) | Bedrock |
| | MRN2 | 23.9 (271.4) | Bedrock |

5.11 Water Levels

The initial and final 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 | MRN2 | March 23, 2001 | 0.7 | 294.6 | In piezometer |
| | MRN2 | April 11, 2001 | 0.2 | 295.1 | In piezometer |
| | MRN2 | January 20, 2005 | 1.4 | 193.9 | In piezometer |
| | MRN2 | February 9, 2005 | 0.0 | 295.3 | Frozen |
| South Approach | MRN3 | March 9, 2001 | 2.5 | 294.5 | Not stabilized |
| North Abutment | 396N-1 | - | - | - | - |
| | 396N-3 | February 9, 2005 | 0.3 | 294.7 | Frozen |
| | 396N-3 | February 17, 2005 | 0.1 | 294.7 | - |
| North Approach | 396N-2 | December 4, 2003 | 0.9 | 293.6 | 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.

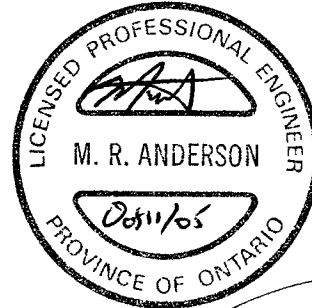
6 MISCELLANEOUS

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

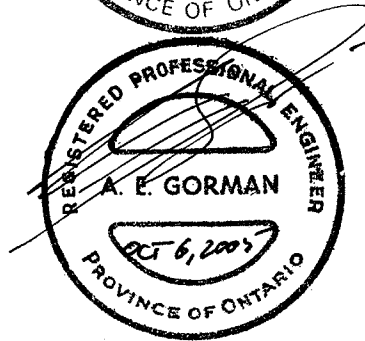
Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. Donald Parent, B.Sc. of Thurber.

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, 57.5 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 301.6 and the original ground lies at Elevation 294.5 \pm , resulting in an approach fill approximately 7 m high.

At the south abutment, the finished grade will be about Elevation 302.2 and the original ground lies at Elevation 294.0 \pm , resulting in an approach fill approximately 8 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 18 to 21 m of generally very loose to compact sandy silt and sand overlying dense to very dense gravelly sand with cobbles and boulders followed by bedrock.

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 9 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.

The piles are expected to encounter refusal in the layer of sand containing cobbles and boulders lying immediate above the bedrock. In some cases, a pile may penetrate this layer without being obstructed by boulders and will meet refusal on the 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 | 274 to 273 | 273 to 275 |
| HP 360 X 132 | 2,100 kN | 1,800 kN | 274 to 273 | 273 to 275 |
| HP 360 X 174 | 2,200 kN | 1,900 kN | 274 to 273 | 273 to 275 |

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

Due to the presence of cobbles and boulders in the expected founding layer, 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 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 on a large boulder.
- The possibility that some piles may fully penetrate the zone of cobbles and boulders and achieve refusal on the bedrock.

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 276.0 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

| Pile | 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 penetrate through the cobble and boulder layer and contact bedrock. 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 | OGI to 285 | 1,200 | 2.7 | 19 |
| | 285 to 275 | 4,000 | 3.0 | 20 |
| | 275 to 272 | 6,000 | 3.3 | 20 |
| North Abutment | OGI to 286 | 1,200 | 2.7 | 19 |
| | 286 to 277 | 4,000 | 3.0 | 20 |
| | 277 to 276 | 5,000 | 3.1 | 20 |
| | 276 to 273 | 6,000 | 3.3 | 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, or possibly in very dense gravelly sand.

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.

In the case of caissons founded in the very dense gravelly sand, it would be impossible to achieve a seal and slurry excavation and tremie concreting would be necessary.

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

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

8.4 Recommended Foundation

The recommended foundation system for both abutments at this site is steel H-piles driven to 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 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 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 the dewatering system that may be required should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering should remain with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

The design of the dewatering system should be coordinated with the design of the sheet-pile cofferdam, where required.

11 APPROACH EMBANKMENTS

The global and internal stability of the approach embankments was analyzed for both 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 6.5 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.

11.2 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 10.1.

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 1.9 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 7.5 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 10.1.

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 1.9 against failure.

11.5 Settlement

The soils 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 175 mm.

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.

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

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

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

12 RETAINED SOIL SYSTEMS

RSS walls used in conjunction with bridge abutments must be "High Performance" and are not recommended as the best option for retaining structures at this site due to the loose near surface soils.

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.

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.

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 OPSS 501.06.

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 below)

γ = unit weight of retained soil (see table below)

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

q = value of any surcharge (kPa)

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

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.

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

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

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.

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.

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 .

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

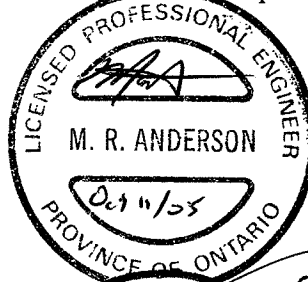
Potential construction concerns include, but are not necessarily limited to:

- The possibility of piles reaching refusal on large boulders or bedrock. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- 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.

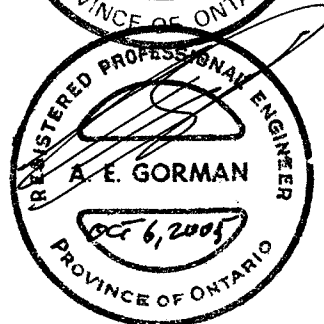
17 CLOSURE

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.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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



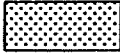


 Water Level
 C_{pen} Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

| MAJOR DIVISIONS | | GROUP SYMBOL | TYPICAL DESCRIPTION |
|----------------------|---------------------------------|--------------|---|
| COARSE GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | GW | Well-graded gravels or gravel-sand mixtures, little or no fines. |
| | | GP | Poorly-graded gravels or gravel-sand mixtures, little or no fines. |
| | | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SAND AND SANDY SOILS | SW | Well-graded sands or gravelly sands, little or no fines. |
| | | SP | Poorly-graded sands or gravelly sands, little or no fines. |
| | | SM | Silty sands, sand-silt mixtures. |
| | | SC | Clayey sands, sand-clay mixtures. |
| FINE GRAINED SOILS | SILTS AND CLAYS $W_L < 50\%$ | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity. |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$). |
| | | CI | Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$). |
| | | OL | Organic silts and organic silty-clays of low plasticity. |
| | SILTS AND CLAYS $W_L > 50\%$ | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | | CH | Inorganic clays of high plasticity, fat clays. |
| | | OH | Organic clays of medium to high plasticity, organic silts. |
| HIGHLY ORGANIC SOILS | | Pt | 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. |
| | | Weak | 5.0 to 25.0 | 750 to 3,500 | Can be peeled by a pocket knife with difficulty |
| | | Very Weak | 1.0 to 5.0 | 150 to 750 | Can be peeled by a pocket knife, crumbles under firm blows of geological pick. |
| | | Extremely Weak (Rock) | 0.25 to 1.0 | 35 to 150 | Indented by thumbnail |

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

RECORD OF BOREHOLE No 396N-1

1 OF 3

METRIC

W.P. 477-93-01 LOCATION N 5048586.1 E 316363.9 Magnetawan River NBL, ST. 12+283, O/S 1R ORIGINATED BY DP
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core COMPILED BY SS
 DATUM Geodetic DATE 21.11.03 - 25.11.03 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 | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | | |
| 294.8 294.0 | TOPSOIL | | | | | | | | | | | | | |
| 0.1 | SAND, fine grained, trace to some silt Very Loose to Loose Brown Wet | | | | | | | | | | | | | |
| | heavily stained by organics, trace rootlets, grey | | 1 | SS | 6 | | 294 | | | | | | | |
| | | | 2 | SS | 5 | | 293 | | | | | | | |
| | | | 3 | SS | 4 | | 292 | | | | | | | |
| | | | 4 | SS | 3 | | 291 | | | | | | | |
| | Brown | | 5 | SS | 3 | | 290 | | | | | | | |
| | | | 6 | SS | 6 | | 289 | | | | | | | |
| | | | 7 | SS | 9 | | 288 | | | | | | | |
| | becoming Compact | | 8 | SS | 19 | | 287 | | | | | | | |
| | | | | | | | 286 | | | | | | | |
| | | | | | | | 285 | | | | | | | |

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 396N-1

3 OF 3

METRIC

W.P. 477-93-01 LOCATION N 5048586.1 E 316363.9 Magnetawan River NBL, ST. 12+283, O/S 1R ORIGINATED BY DP
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core COMPILED BY SS
DATUM Geodetic DATE 21.11.03 - 25.11.03 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 (%) GR SA SI CL | | | |
|---------------|-------------|------------|---------|------|------------|----------------------------|-----------------|---|----------|--|--|--|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) w _p — w — w _L | | |
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ONTMT4 MAGENTAWAN RIVER.GPJ 20/07/04

RECORD OF BOREHOLE No 396N-2

1 OF 1

METRIC

W.P. 477-93-01 LOCATION N 5048604.0 E 316350.3 Magnetawan River NBL, ST. 12+305, O/S 18.75R 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 | | |
| 294.5 | TOPSOIL | | | | | | | | | | | | | |
| 294.0 | Silty SAND, fine grained Brown | | | | | | | | | | | | | |
| 293.9 | | | | | | | | | | | | | | |
| 0.6 | SAND, fine grained, trace to some silt Loose to Very Loose Brown Wet | | 1 | SS | 7 | | 294 | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 2 | SS | 2 | | 293 | | | | | | | |
| | grey | | | | | | | | | | | | | |
| | | | 3 | SS | 2 | | 292 | | | | | | | |
| | | | | | | | | | | | | | | |
| | occasional decayed wood fibers, occasional black staining from 3.0m to 4.1m | | 4 | SS | 1 | | 291 | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | 290 | | | | | | | |
| | | | 5 | SS | 3 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | 289 | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 6 | SS | 4 | | 288 | | | | | | | |
| 287.8 | | | | | | | | | | | | | | |
| 6.7 | END OF BOREHOLE AT 6.7m. BOREHOLE OPEN TO 1.7m. WATER LEVEL IN OPEN BOREHOLE AT 0.9m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE. | | | | | | | | | | | | | |

ONTMT4 MAGENTAWAN RIVER.GPJ 20/07/04

RECORD OF BOREHOLE No 396N-3

1 OF 3

METRIC

W.P. 477-93-01 LOCATION N=5 048 587.9 , E=316 349.7 Magnetawan River Bridge, NC-N ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 11.02.05 - 15.02.05 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 | | |
| 295.0 | | | | | | | | | | | | | | |
| 0.0 | Sandy SILT, trace clay, trace rootlets Compact Brown Wet | | 1 | SS | 15 | | 295 | | | | | | | |
| | | | | | | | 294 | | | | | | | |
| | | | | | | | 293 | | | | | | | |
| 292.6 | | | | | | | | | | | | | | |
| 2.4 | SAND, fine grained, some silt, trace clay Compact to Loose Grey Wet | | 2 | SS | 12 | | 292 | | | | | | | |
| | | | | | | | 291 | | | | | | | |
| | | | | | | | 290 | | | | | | | |
| | | | | | | | 289 | | | | | | | |
| | | | 3 | SS | 7 | | 288 | | | | | | | |
| | | | | | | | 287 | | | | | | | |
| | | | | | | | 286 | | | | | | | |
| 285.9 | | | | | | | | | | | | | | |
| 9.1 | SAND, fine to medium grained, trace silt Loose to Compact Brown Wet | | 4 | SS | 6 | | | | | | | | | |

0 72 24 5

Continued Next Page

+ ³ . x ³ : Numbers refer to
Sensitivity

20
15- ϕ 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 396N-3

2 OF 3

METRIC

W.P. 477-93-01 LOCATION N=5 048 587.9 , E=316 349.7 Magnetawan River Bridge, NC-N ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 11.02.05 - 15.02.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|-------------|------------|---------|------|------------|----------------------------|-----------------|---|----|----|-----|--|---|---|----------------|---------------------------------------|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | W _p | W | W _L | | |
| | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| | | | | | | | 285 | | | | | | | | | | |
| | | | | | | | 284 | | | | | | | | | | |
| | | | 5 | SS | 9 | | 283 | | | | | | | | | | 0 93 7 (SI+CL) |
| | | | | | | | 282 | | | | | | | | | | |
| | | | | | | | 281 | | | | | | | | | | |
| | | | 6 | SS | 12 | | 280 | | | | | | | | | | |
| | | | | | | | 279 | | | | | | | | | | |
| | | | | | | | 278 | | | | | | | | | | |
| | | | | | | | 277 | | | | | | | | | | |
| | | | 7 | SS | 18 | | 276 | | | | | | | | | | 0 96 4 (SI+CL) |

Continued Next Page

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

ONTM/T4S 2316(396, 400), GPJ 06/09/05

RECORD OF BOREHOLE No 396N-3

3 OF 3

METRIC

W.P. 477-93-01 LOCATION N=5 048 587.9 , E=316 349.7 Magnetawan River Bridge, NC-N ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 11.02.05 - 15.02.05 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 | | |
| 274.3 | SAND and GRAVEL Very Dense Brown Wet | | 8 | SS | 100/ 225 | | 275 | | | | | | | |
| 20.7 | | | | | | | 274 | | | | | | | |
| 272.8 | Fresh, coarse grained, massive, grey/white, strong to very strong GNEISS | | 1 | RUN | | | 273 | | | | | | | |
| 22.3 | | | | | | | 272 | | | | | | | |
| | | | | | | | 271 | | | | | | | |
| | | | | | | | 270 | | | | | | | |
| 268.9 | END OF BOREHOLE AT 26.11 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. | | 2 | RUN | | | 269 | | | | | | | |
| 26.1 | | | | | | | | | | | | | | |

WATER LEVEL READINGS:
 DATE DEPTH
 (m)
 16-Feb-05 0.51
 17-Feb-05 0.25 (Frozen)

+ 3 , x 3 : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 396N-4

2 OF 3

METRIC

W.P. 477-93-01 LOCATION N=5 048 535.9 , E=316 380.7 Magnetawan River Bridge, NC-N ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 19.01.05 - 20.01.05 CHECKED BY MA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|---|---|----------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | | |
| 282.2 | SAND, fine to medium grained, trace silt Compact Brown Wet | | 5 | SS | 13 | | | | | | | | | | | | |
| 281 | | | | | | | | | | | | | | | | | |
| 280 | | | | | | | | | | | | | | | | | |
| 279 | | | 6 | SS | 13 | | | | | | | | | | | | |
| 278 | | | | | | | | | | | | | | | | | |
| 277 | | | | | | | | | | | | | | | | | |
| 276 | SILT, some sand Compact Grey Wet | | 7 | SS | 11 | | | | | | | | | | | | |
| 275 | | | | | | | | | | | | | | | | | |

ONTMT4S 2316(396, 400).GPJ 06/09/05

Continued Next Page

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

RECORD OF BOREHOLE No 396N-4

3 OF 3

METRIC

W.P. 477-93-01 LOCATION N=5 048 535.9, E=316 380.7 Magnetawan River Bridge, NC-N ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 19.01.05 - 20.01.05 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 | | |
| 273.6 | | | | | | | | | | | | | | |
| 20.7 | GRAVEL and SAND, trace silt, occasional cobbles Grey Wet | | | | | | | | | | | | | |
| 272.9 | | | | | | | | | | | | | | |
| 21.5 | END OF SOIL SAMPLING AT 21.49 m. CORING STARTED AT 21.49 m. Fresh, coarse grained, massive, grey, white/black, strong to very strong GNEISS | | 1 | RUN | | | | | | | | | FI | RUN 1# TCR=100%, SCR=100%, RQD=100% |
| | | | 2 | RUN | | | | | | | | | 2 | RUN 2# TCR=100%, SCR=50%, RQD=47% |
| | | | | | | | | | | | | | >5 | |
| | | | | | | | | | | | | | 0 | |
| | | | | | | | | | | | | | 1 | |
| | | | | | | | | | | | | | 0 | RUN 3# TCR=100%, SCR=100%, RQD=98% |
| | | | 3 | RUN | | | | | | | | | 1 | |
| | | | | | | | | | | | | | 2 | |
| | | | | | | | | | | | | | 0 | |
| 269.6 | | | | | | | | | | | | | 0 | |
| 24.8 | END OF BOREHOLE AT 24.79 m. BOREHOLE OPEN TO 24.79 m AND WATER LEVEL AT 0.61 m UPON COMPLETION. BOREHOLE GROUTED TO SURFACE. | | | | | | | | | | | | | |

ONTMT4S 2316(396, 400).GPJ 06/09/05

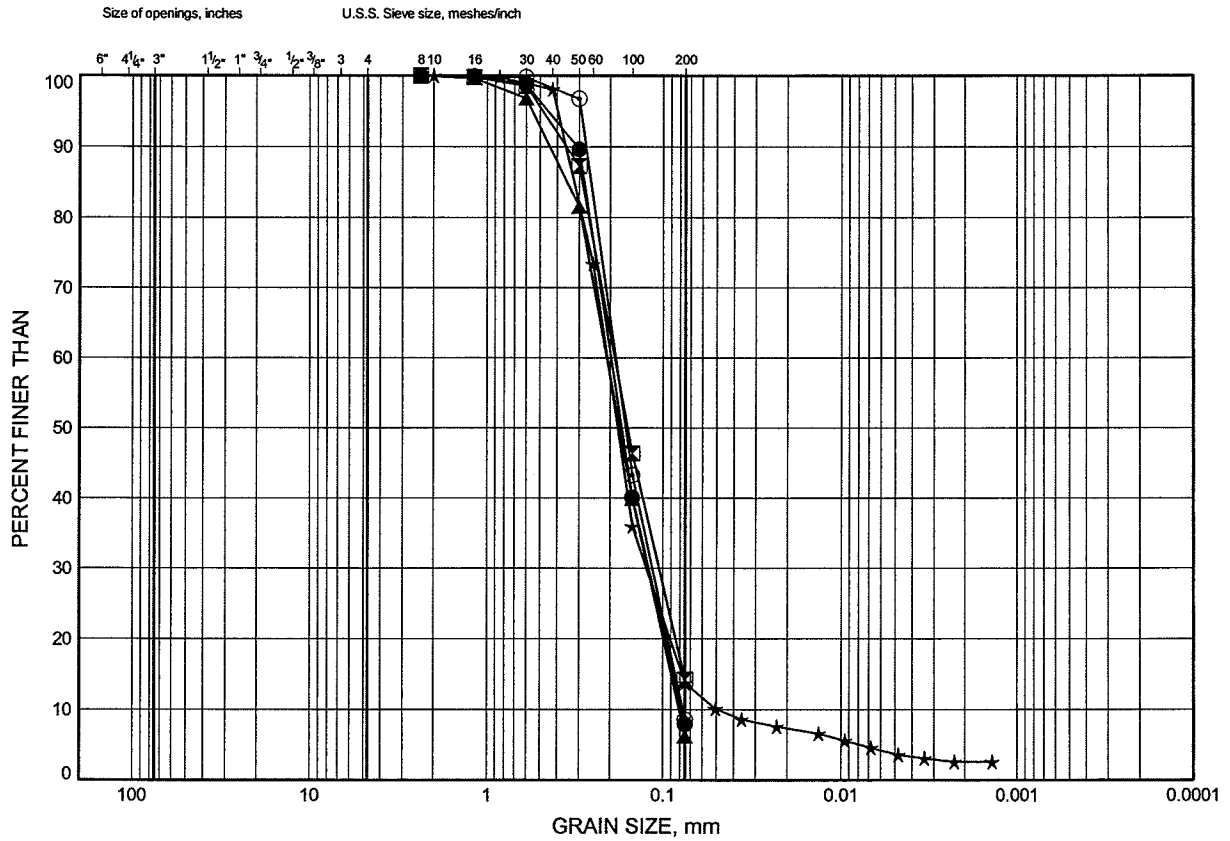
Appendix B

Laboratory Test Results

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND, trace to some silt

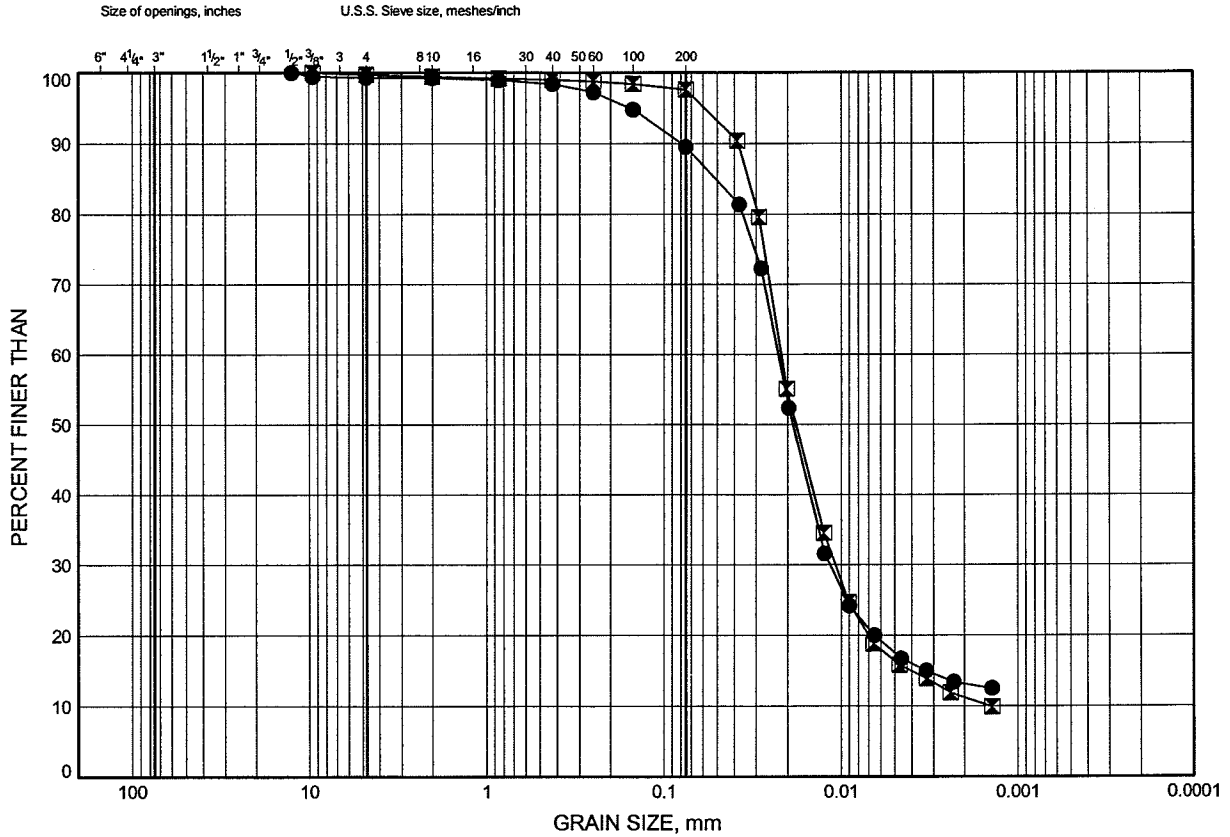


Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILT, some sand



Appendix C

Data From Shaheen & Peaker Report

RECORD OF BOREHOLE No MRN1

1 OF 2

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 546.9; E 316 388.0 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers, Casing and wash boring & NQ Rock Core COMPILED BY G.T.
DATUM Geodetic DATE 06.03.01 & 07.03.01 CHECKED BY Z.O.

| 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 | | |
| 293.6 | Ground Surface | | | | | | | | | | | | | |
| 0.0 | SILT, trace organics, very loose, grey, wet (possible fill) | | 1 | SS | 2 | | 293 | | | | | | | |
| 292.6 | | | 2 | SS | 2 | | | | | | | | | |
| 1.0 | ORGANIC SILT-sandy | | | | | | | | | | | | | |
| 292.3 | dark grey/black, very loose | | 3 | SS | 3 | | 292 | | | | | | | |
| 1.3 | SILTY FINE SAND | | | | | | | | | | | | | |
| | trace of organics and decayed wood, grey/dark grey, very loose to loose, wet | | 4 | SS | 7 | | 291 | | | | | | | |
| 290.7 | | | | | | | | | | | | | | |
| 2.9 | SILT, trace organics, very loose, grey, wet | | 5 | SS | 4 | | 290 | | | | | | | |
| 290.0 | | | 6 | SS | 5 | | | | | | | | | |
| 3.6 | | | 7 | SS | 10 | | 289 | | | | | | | |
| | | | 8 | SS | 4 | | 288 | | | | | | | |
| | | | 9 | SS | 5 | | 287 | | | | | | | |
| | | | 10 | SS | 6 | | 286 | | | | | | | |
| | | | 11 | SS | 14 | | 285 | | | | | | | |
| | | | 12 | SS | 15 | | 284 | | | | | | | |
| | | | 13 | SS | 10 | | 283 | | | | | | | |
| | | | 14 | SS | 16 | | 282 | | | | | | | |
| | | | | | | | 281 | | | | | | | |
| | | | | | | | 280 | | | | | | | |
| | | | | | | | 279 | | | | | | | |
| 278.6 | | | | | | | | | | | | | | |

15.0

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

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15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MRN1

2 OF 2

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 546.9; E 316 388.0 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers, Casing and wash boring & NQ Rock Core COMPILED BY G.T.
DATUM Geodetic DATE 06.03.01 & 07.03.01 CHECKED BY Z.O.

| 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 |
|----------------|---|------------|--------|------|----------------------------|-----------------|---|----|----|----|-----|---|--|
| FLEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | 20 | 40 | 60 | 80 | 100 | | |
| 278.6 | | | | | | | | | | | | | |
| 15.0 | FINE SAND traces to some silt, some silty zones, grey, wet | | 15 | SS | 17 | 278 | | | | | | | 0 94 (6) |
| | compact | | 16 | SS | 18 | 277 | | | | | | | |
| | ----- | | | | | 276 | | | | | | | |
| | dense | | 17 | SS | 45 | 275 | | | | | | | March 06 |
| | ----- | | | | | | | | | | | | March 07 |
| | gravel and sand inferred | | | | | 274 | | | | | | | |
| 273.8 | | | | | | | | | | | | | |
| 19.8 | End of borehole Casing advanced to 21.3 m (probably bent). Casing advanced to 24.4 m where casing broke (probably sliding on sloping rock surface). *Water level not stabilized on completion. Borehole grouted on completion Moved 4.0 m to the south and redrilled See borehole NRN1A | | | | | | | | | | | | |

RECORD OF BOREHOLE No MRN1A

1 OF 2

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 543.4; E 316 390.2 ORIGINATED BY R.A.
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 NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|----------------|---|------------|--------|------|----------------------------|-----------------|---|-----------------|---|-----------|-------------------|---|--|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | 20 40 60 80 100 | 20 40 60 | W P W W L | WATER CONTENT (%) | | |
| 293.9 0.0 | Ground Surface | | | | | | | | | | | | |
| | Borehole extended to 18.5 m without sampling; see MRN1 | | | | | | | | | | | | |
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| | | | | | | | | | | | | | |
| 278.9 15.0 | | | | | | | | | | | | | |

Continued Next Page

RECORD OF BOREHOLE No MRN1A

2 OF 2

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 543.4; E 316 390.2 ORIGINATED BY R.A.
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 | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|----------------|---|------------|---------|----------|--------------|----------------------------|-----------------|---|--|--|--|---|---|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | |
| | | | | | | | | | | | | | |
| 278.9 | | | | | | | | | | | | | |
| 15.0 | Refer to borehole MRN1 for soil stratigraphy | | | | | | | | | | | | |
| 275.4 | | | | | | | | | | | | | |
| 18.5 | COBBLES AND BOULDERS | | 1 | NQ RC | Rec. 43% | | | | | | | | |
| | | | 2 | NQ RC | Rec. 75% | | | | | | | | |
| | | | 3 | NQ RC | Rec. 22% | | | | | | | | |
| 271.6 | | | | | | | | | | | | | |
| 22.3 | moderately weathered ----- unweathered GNEISS BEDROCK gray | | 4 | NQ RC | Rec. 50% | | | | | | | | |
| | | | 5 | NQ RC | Rec. 77% | | | | | | | | |
| | | | 6 | NQ RC | Rec. 100% | | | | | | | | |
| | | | 7 | NQ RC | Rec. 98% | | | | | | | | |
| 268.5 | | | | | | | | | | | | | |
| 25.4 | End of borehole Artesian condition experienced while drilling @ 18.5 m depth | | | | | | | | | | | | |

RECORD OF BOREHOLE No MRN2

1 OF 3

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 533.8; E 316 395.9 ORIGINATED BY R.A.
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers, Casing and wash boring, NQ Rock Core & D.C.P.T. COMPILED BY G.T.
 DATUM Geodetic DATE 14.03.01 to 20.03.01 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 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 |
| 295.3 | Ground Surface | | | | | 20 40 60 80 100 | 20 40 60 | | | | | | | | |
| 0.0 | 150 mm Topsoil SILT with Clayey Silt layers, some organics, brown/dark brown/grey, soft to firm | | 1 | SS | 2 | | | | | | | | | | |
| | | | 2 | SS | 5 | | | | | | | | | | |
| 293.2 | | | 3 | SS | 5 | | | | | | | | 0 13 82 5 | | |
| 2.1 | | | 4 | SS | 3 | | | | | | | | | | |
| | | | 5 | SS | 4 | | | | | | | | | | |
| | | | 6 | SS | 5 | | | | | | | | 0 41 59 0 | | |
| | | | 7 | SS | 4 | | | | | | | | | | |
| | silty gray ----- brown | | 8 | SS | 4 | | | | | | | | | | |
| | | | 9 | SS | 2 | | | | | | | | | | |
| | FINE SAND trace to some silt, wet | | 10 | SS | 0* | | | | | | | | Experienced quick condition. Commenced casing and washboring *No recovery | | |
| | | | 11 | SS | 1 | | | | | | | | | | |
| | loose to very loose ----- compact | | 12 | SS | 11 | | | | | | | | | | |
| | | | 13 | SS | 11 | | | | | | | | | | |
| | | | 14 | SS | 25 | | | | | | | | | | |
| | | | 15 | SS | 13 | | | | | | | | | | |
| 280.3 | | | | | | | | | | | | | | | |

15.0

Continued Next Page

+ 3. × 3. Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

| RECORD OF BOREHOLE No MRN2 | | | | | | | | | | 2 OF 3 | | METRIC | | | |
|----------------------------|--|--|--------|----------|----------------------------|-------------------|---|--------------------|----|------------------------------------|-------------------------------------|-----------------------------------|--|---|-------------------|
| W.P. 314-99-00 | | LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 533.8; E 316 395.9 | | | | ORIGINATED BY R.A | | | | | | | | | |
| DIST 52 HWY 11 | | BOREHOLE TYPE Solid Stem Augers, Casing and wash boring, NO Rock Core & D.C.P.T. | | | | COMPILED BY G.T | | | | | | | | | |
| DATUM Geodetic | | DATE 14.03.01 to 20.03.01 | | | | 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 kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | |
| FLY DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | | | WATER CONTENT (%) |
| 280.3 | | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | |
| 15.0 | FINE SAND trace to some silt, compact, wet | | 16 | SS | 10 | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | |
| | silt and clayey silt seams | | 17 | SS | 22 | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| 274.7 | | | | | | | | | | | | | | | |
| 20.6 | COBBLES & BOULDERS with sand | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | |
| 271.4 | | | | | | | | | | | | | | | |
| 23.9 | GNEISS BEDROCK grey unweathered | | 20 | NQ RC | Rec. 100% | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| 268.4 | | | | | | | | | | | | | | | |
| 26.9 | End of borehole Piezometer installed at 23.1 m Water level at: March 23/2001 - 0.70 m March 26/2001 - 0.60 m March 27/2001 - 0.60 m March 28/2001 - 0.55 m March 29/2001 - 0.55 m April 02/2001 - 0.50 m April 04/2001 - 0.55 m April 06/2001 - 0.55 m | | | | | | | | | | | | | | |

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

3 OF 3

METRIC

| | | | | | |
|-------|-----------|----------|--|---------------|--|
| W.P. | 314-99-00 | LOCATION | Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 533.8; E 316 395.9 | ORIGINATED BY | R.A |
| DIST | 52 | HWY | 11 | BOREHOLE TYPE | Solid Stem Augers, Casing and wash boring, NQ Rock Core & D.C.P.T. |
| DATUM | Geodetic | DATE | 14.03.01 to 20.03.01 | COMPILED BY | G.T |
| | | | | CHECKED BY | Z.O |

[illegible]

RECORD OF BOREHOLE No MRN3

1 OF 2

METRIC

W.P. 314-99-00 LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 516.2; E 316 406.2 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augers & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 08.03.01 & 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 kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
|----------------|---|-------------|---------|------|------------|----------------------------|-----------------|---|------------------|--------------|------------------------------------|-------------------------------------|-----------------------------------|--|---|-------------------|------------|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | WATER CONTENT (%) | |
| | | | | | | | | ○ UNCONFINED | ● QUICK TRIAXIAL | + FIELD VANE | | | | | | | × LAB VANE |
| | | | | | | | | | | | | | | | | | |
| 297.0 | Ground Surface | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | | | | |
| 296.8 | SAND AND GRAVEL (FILL) | | | | | | | | | | | | | | | | |
| 0.2 | SILTY CLAY layered stiff, grey | | 1 | SS | 11 | | | | | | | ○ | | 18.7 | | | |
| 296.0 | | | 2 | SS | 7 | | | | | | | ○ | | | | | |
| 1.0 | SILT some clayey seams, firm to stiff, grey wet, dilatent | | 3 | SS | 7 | | | | | | | ○ | | 17.5 | 0 1 91 8 | | |
| | | | 4 | SS | 8 | | | | | | | ○ | | | March 08 | | |
| | | | 5 | SS | 6 | | | | | | | ○ | | | March 09 | | |
| | ----- sandy, very loose | | 6 | SS | 3 | | | | | | | ○ | | | 0 25 75 0 | | |
| 292.5 | | | | | | | | | | | | | | | | | |
| 4.5 | FINE SAND occasional silt and thin silty clay seams, brown, greyish brown, loose, wet | | 7 | SS | 6 | | | | | | | ○ | | | | | |
| | silty ----- some silt | | 8 | SS | 6 | | | | | | | ○ | | | | | |
| | | | 9 | SS | 8 | | | | | | | ○ | | | | | |
| | | | 10 | SS | 2 | | | | | | | ○ | | | | | |
| 287.4 | | | 11 | SS | 6 | | | | | | | ○ | | | | | |
| 9.6 | End of borehole *Water level at 2.5 m upon completion (not stabilized) Dynamic Cone Penetration Test performed from 9.6 m to 15.6 m | | | | | | | | | | | | | | | | |
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15.0

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15-20.5
22 (%) STRAIN AT FAILURE

| RECORD OF BOREHOLE No MRN3 | | | | | | | | | | 2 OF 2 | | METRIC | |
|----------------------------|--------------------------------------|---|--------|------|----------------------------|-----------------|---|--------------------|---|-------------------|--|---------------------------------------|--|
| W.P. 314-99-00 | | LOCATION Hwy 11 NBL over Magnetawan River Co-ords: N 5 048 516.2; E 316 406.2 | | | | | | ORIGINATED BY R.A | | | | | |
| DIST 52 HWY 11 | | BOREHOLE TYPE Solid Stem Augers & D.C.P.T. | | | | | | COMPILED BY G.T | | | | | |
| DATUM Geodetic | | DATE 08.03.01 & 09.03.01 | | | | | | CHECKED BY Z.O | | | | | |
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| FLEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | WATER CONTENT (%) | | | |
| 282.0 | | | | | | | 20 40 60 80 100 | 20 40 60 | | | | | |
| 15.0 | | | | | | | 20 40 60 80 100 | 20 40 60 | | | | | |
| 281.4 | | | | | | | 20 40 60 80 100 | 20 40 60 | | | | | |
| 15.6 | End of Dynamic Cone Penetration Test | | | | | | | | | | | | |

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

GRAIN SIZE IN MICROMETERS

Find

SAND

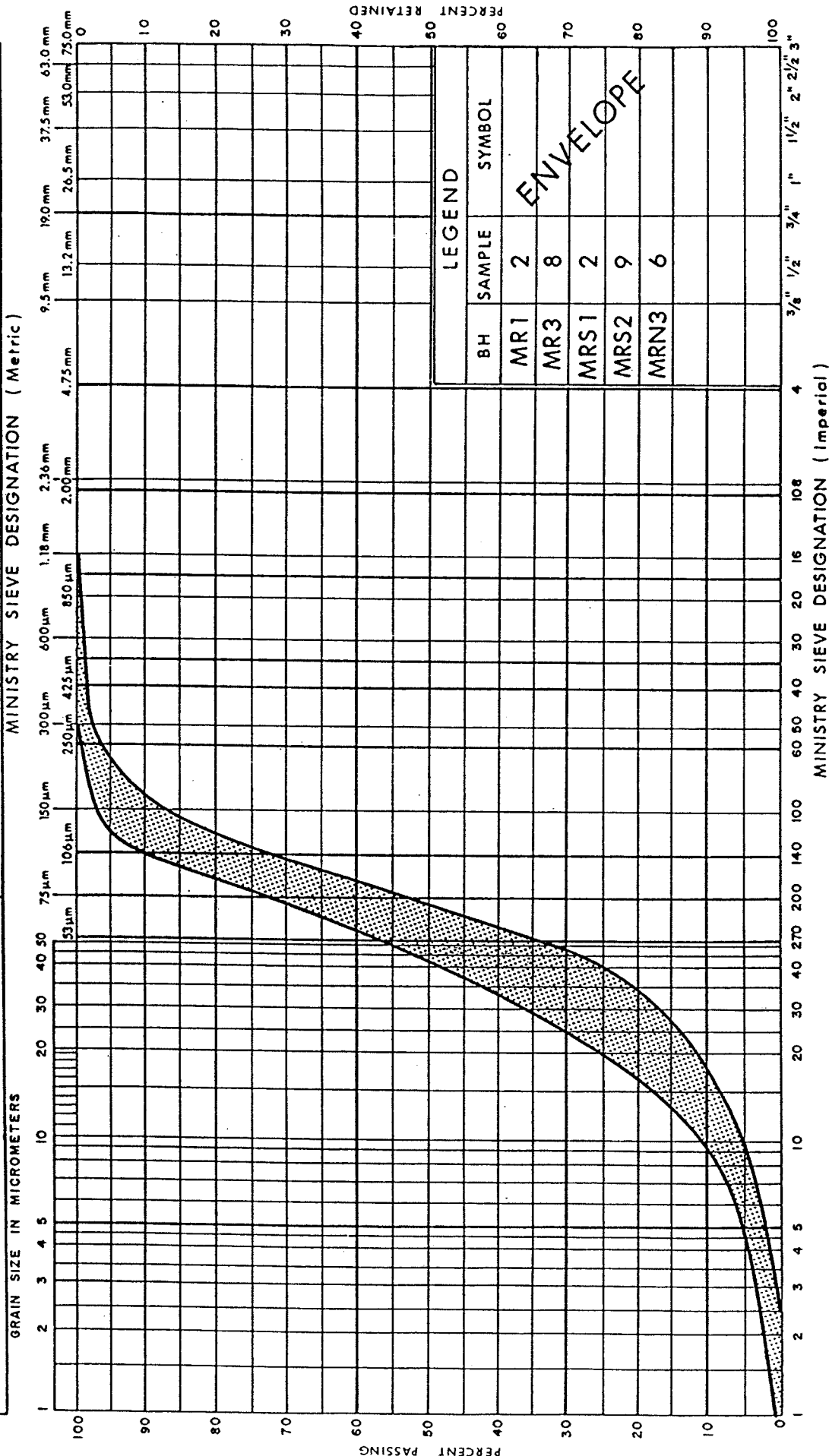
Medium

Coarse

Fine

GRAVEL

Coarse

Ministry of
Transportation

Ontario

GRAIN SIZE DISTRIBUTION SANDY SILT TO SILTY SAND

FIG No 4

W P 314-99-00

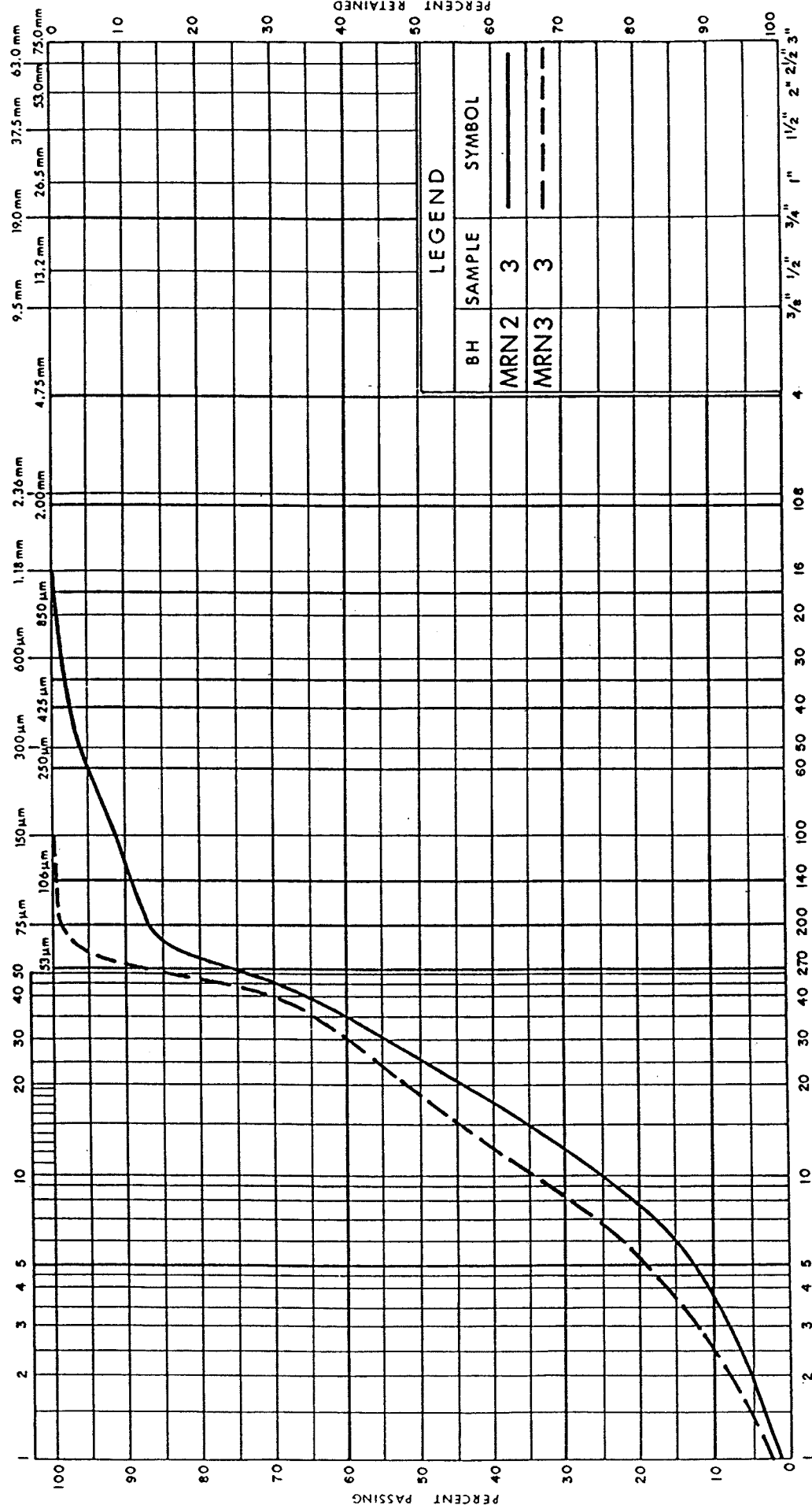
SPT 1010B

78 12 M

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE IN MICROMETERS



LEGEND

| BH | SAMPLE | SYMBOL |
|------|--------|--------|
| MRN2 | 3 | — |
| MRN3 | 3 | - - - |

MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SILT, TRACE CLAY

FIG No 5

W P 314-99-00

SPT 1010B

Ministry of
Transportation

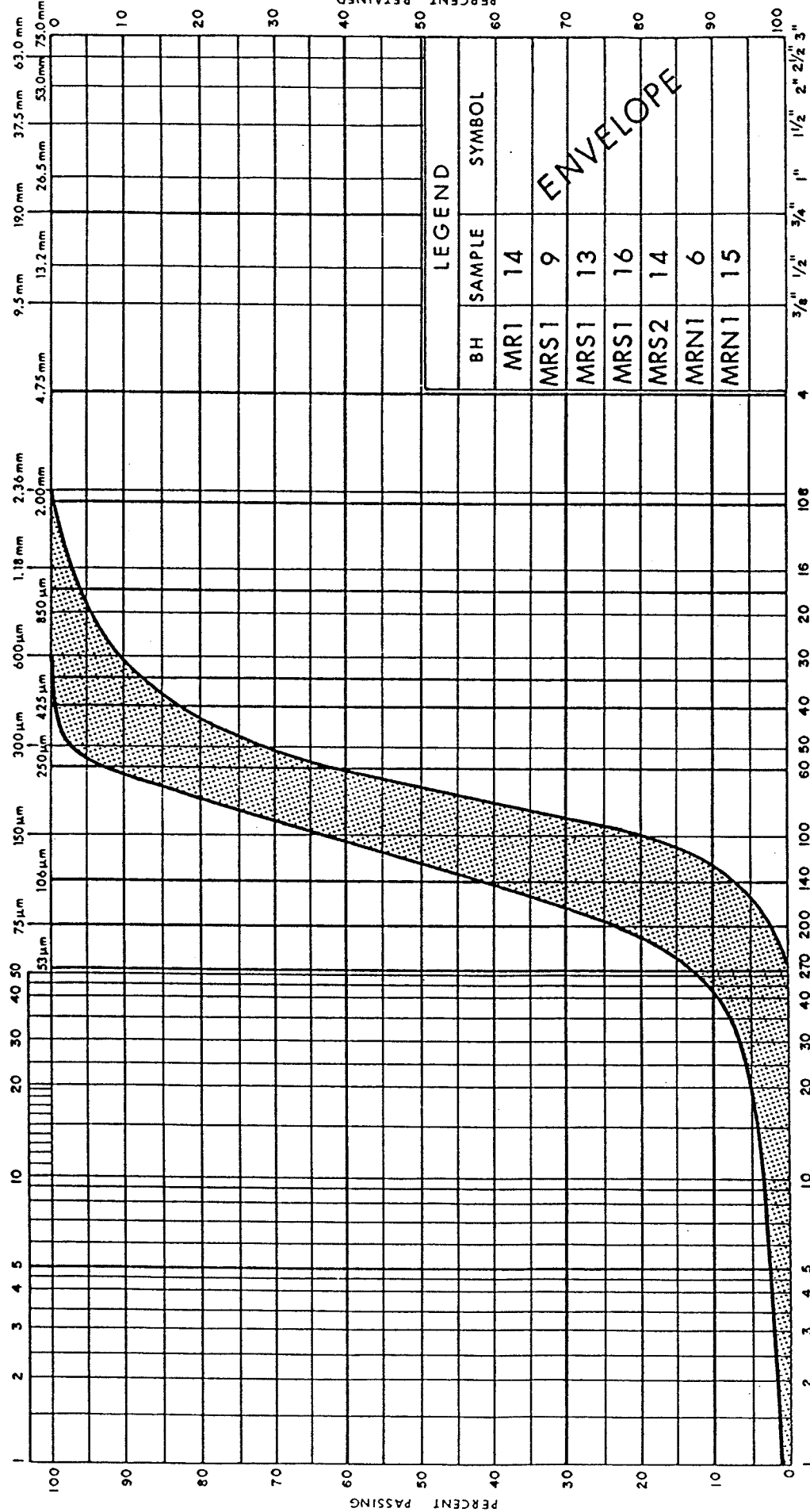


UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE IN MICROMETERS

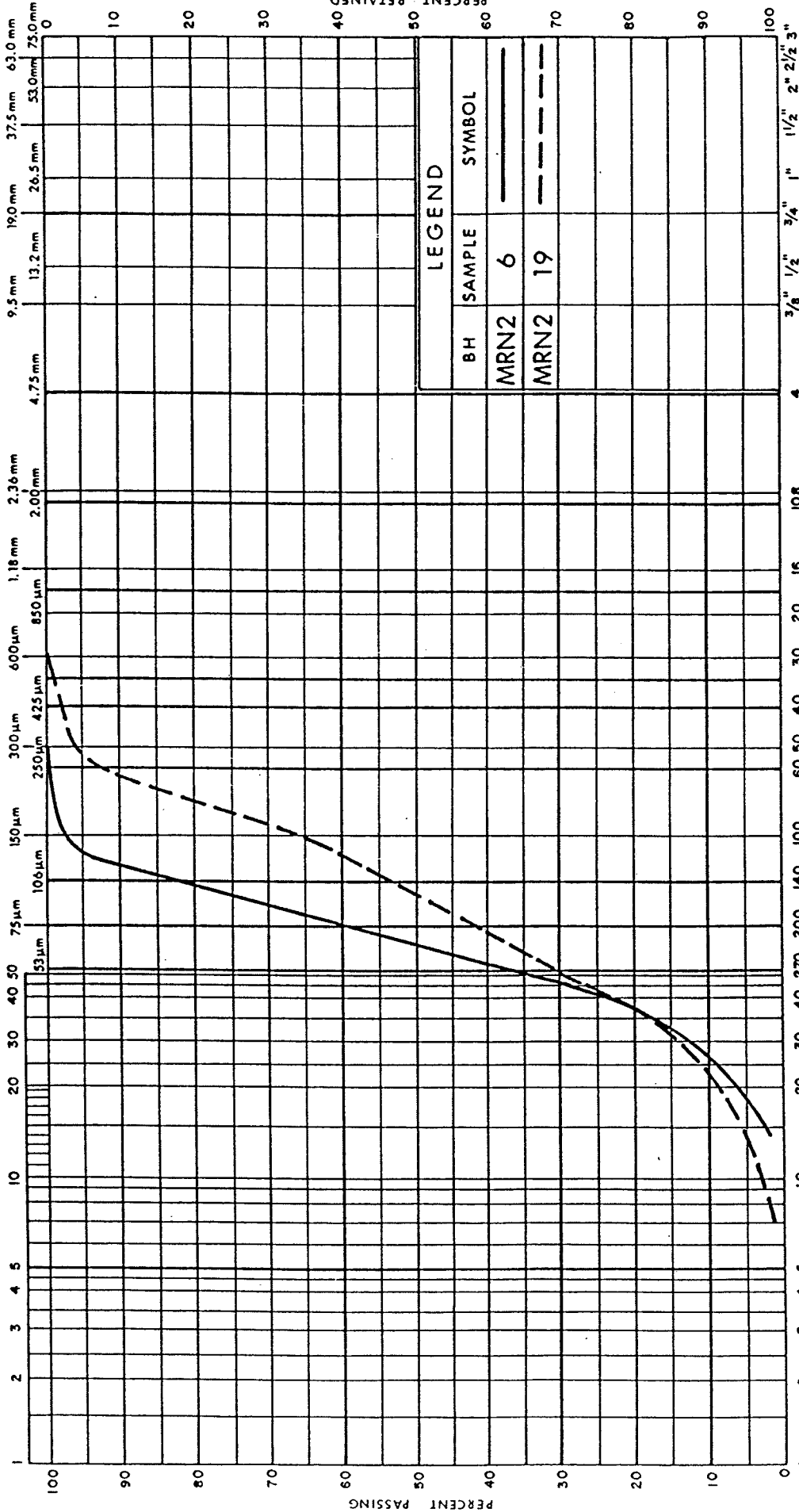
MINISTRY SIEVE DESIGNATION (Metric)



UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE IN MICROMETERS



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

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

“ The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 276. 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

| | Gamma C kN/m3 | Phi deg | Piezo Surf. |
|-----------------|---------------------|------------|----------------|
| Water | 9.81 | 0 | 0 |
| Backfill | 21 | 0 | 1 |
| Rockfill | 20 | 0 | 1 |
| Organic Silt | 19 | 0 | 1 |
| Silt/Silty Clay | 20 | 0 | 1 |
| Fine Sand | 21 | 0 | 1 |
| Hard Bottom | (Infinitely Strong) | | |

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy11 - Katrine
 December 2003
 North Crossing - NBL - South Bank
 Setback from May 2003 Water Level: 6.5m

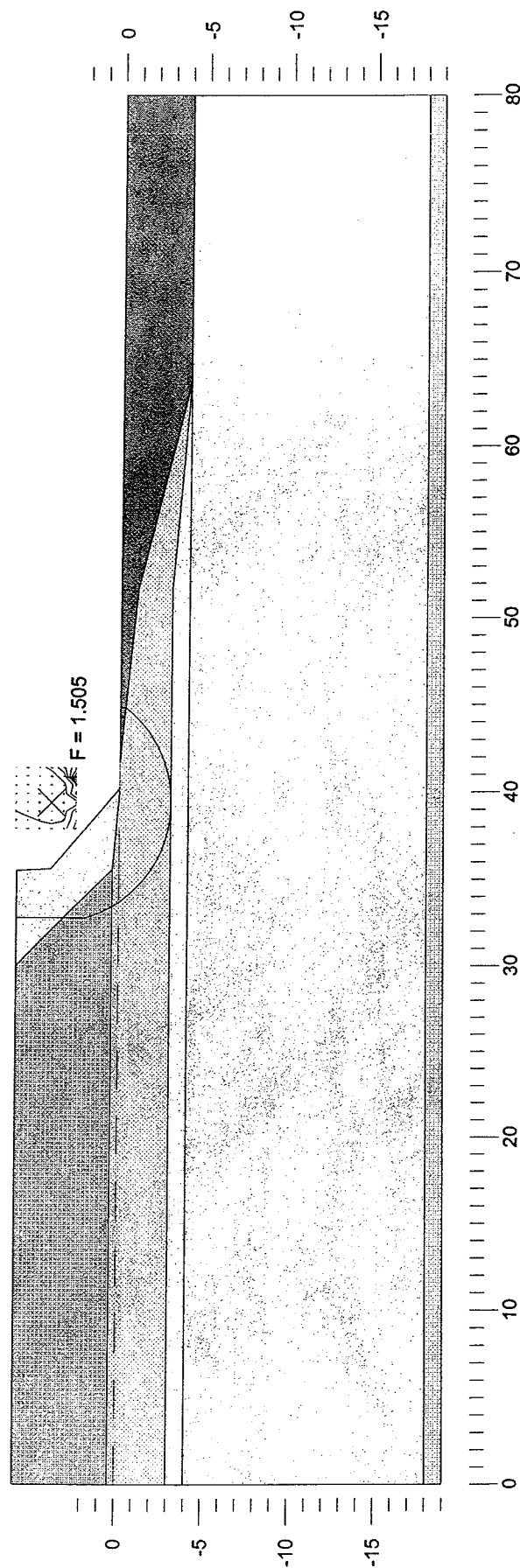


Figure 1

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

Thurber Engineering Ltd. - Toronto
19-1423-16
Hwy11 - Katrine
June 21, 2004
North Crossing - NBL - South Bank
lateral stability

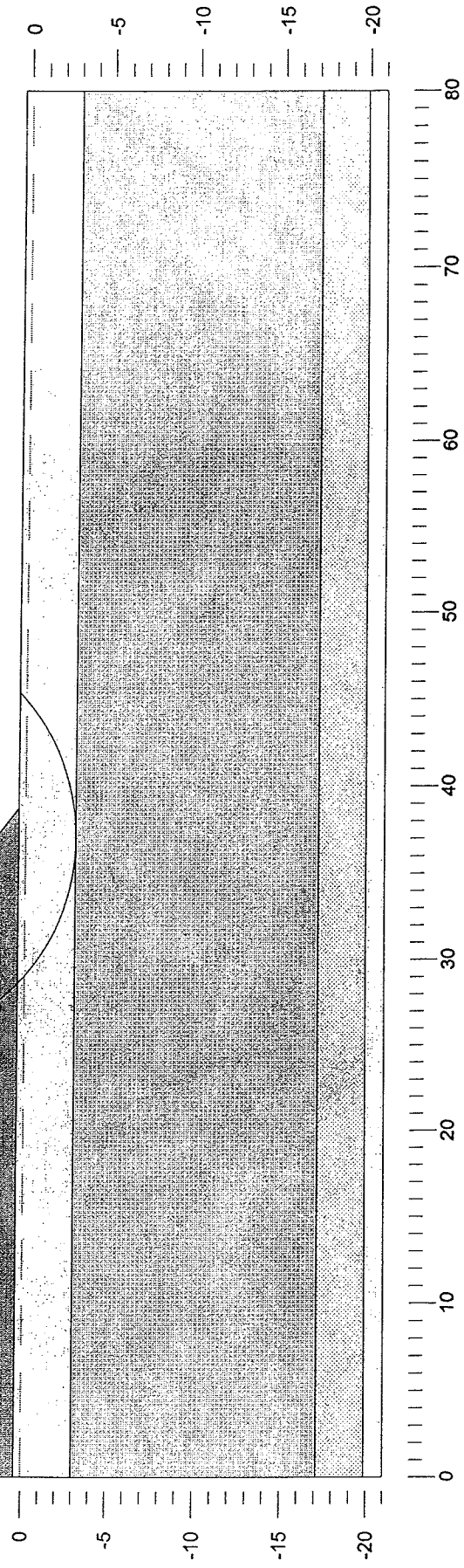
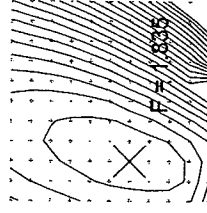


Figure F2

Thurber Engineering Ltd. - Toronto

19-1423-16

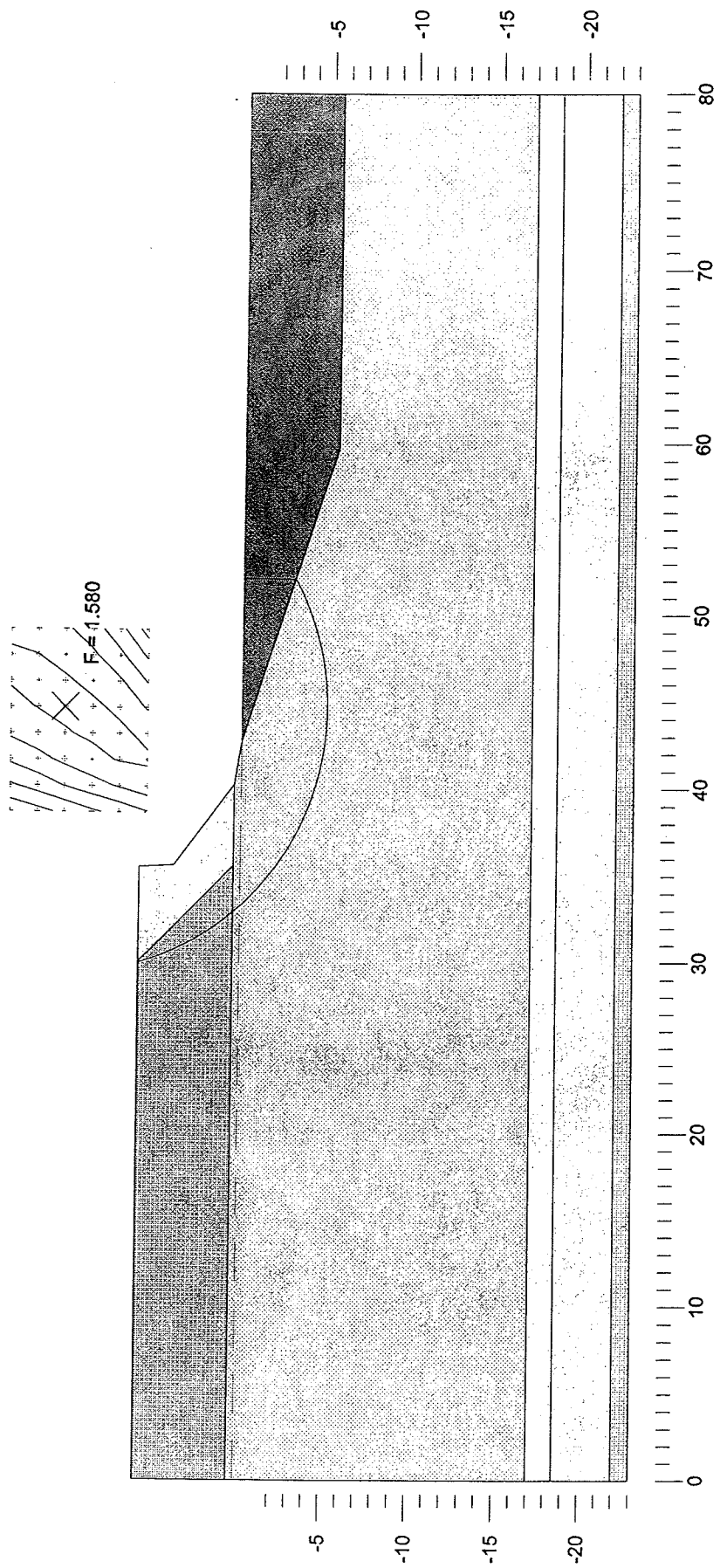
Hwy11 - Katrine

December 2003

North Crossing - NBL - North Bank

Setback from May 2003 Water Level: 7.25m

| | Gamma C | Phi | Piezo |
|-----------------|---------------------|-----|-------|
| | kN/m3 | deg | Surf. |
| Water | 9.81 | 0 | 0 |
| Backfill | 21 | 30 | 1 |
| Rockfill | 20 | 42 | 1 |
| Fine Sand | 19 | 28 | 1 |
| Silt | 20 | 30 | 1 |
| Gravel and Sand | 21 | 32 | 1 |
| Hard Bottom | (Infinitely Strong) | | |



| | Gamma C | Phi | Piezo |
|-----------------|---------------------|-----|-------|
| | kN/m ³ | 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) | | |

Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy11 - Katrine
 July 22 2004
 North Crossing - NBL - North Bank
 Lateral stability

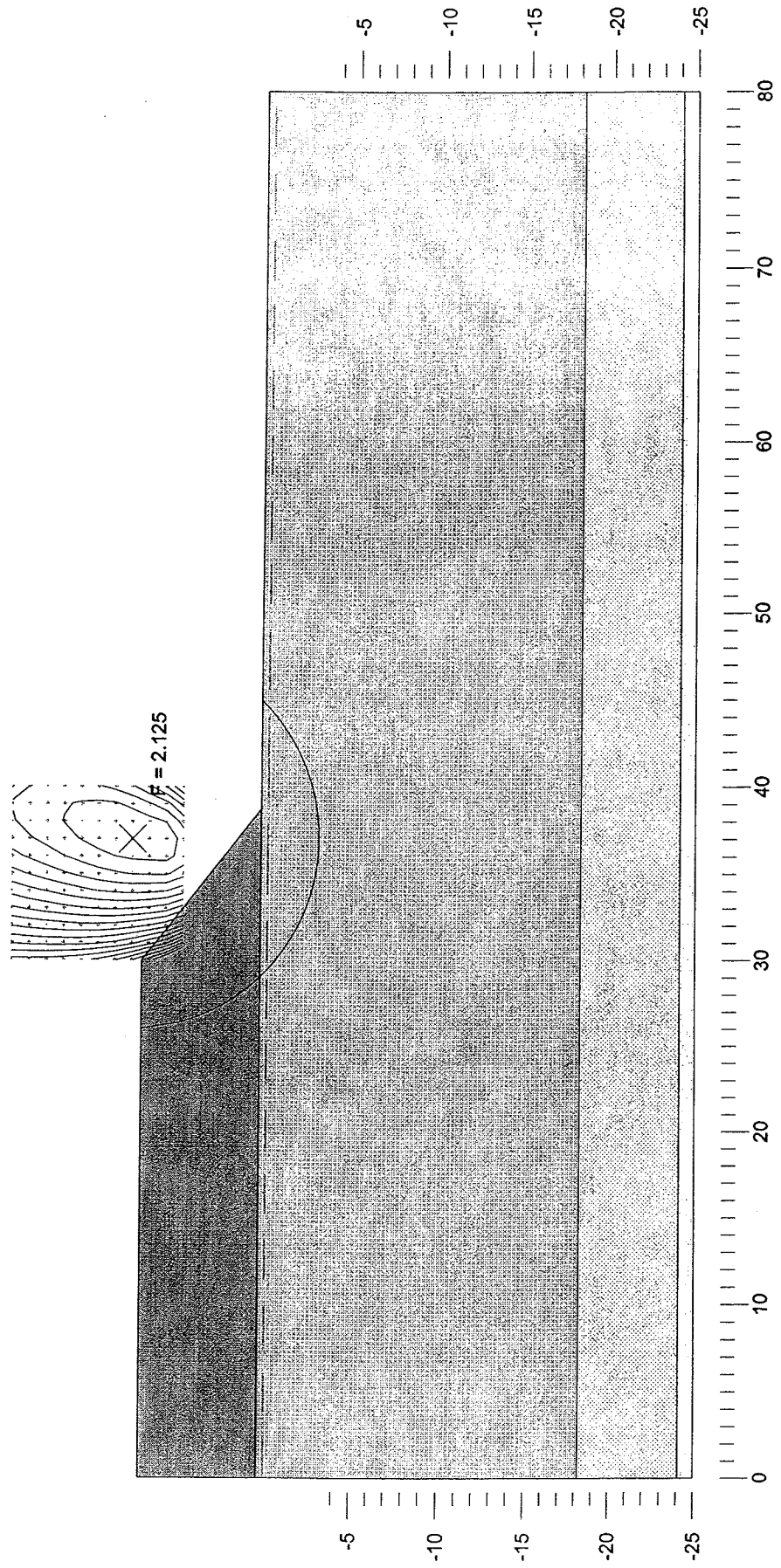


Figure 4

Appendix G

Drawings

