

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
FROG RAPIDS  
HIGHWAY 72, SIOUX LOOKOUT  
G.W.P. 452-00-00, SITE 41S-5**

**Geocres Number: 52J-7**

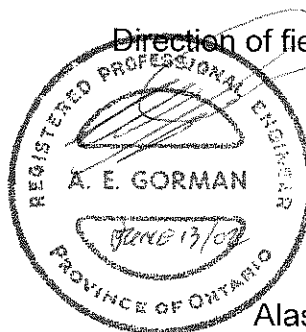
Report

to

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## **1 INTRODUCTION**

This report presents the results of the foundation investigation carried out by Thurber Engineering Ltd. (Thurber) at Frog Rapids on Highway 72.

The investigation was carried out at the site of an existing four-span structure consisting of two, through truss main spans and two girder approach spans. The main spans are supported on piers founded in the river and the abutments are believed to be supported on spread footings founded in rock fill. North and south approach causeways have been constructed into the narrows.

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, a stratigraphic profile and a written description of the subsurface conditions.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation under Ministry of Transportation (MTO) Agreement 6005-A-000126.

## **2 SITE DESCRIPTION**

### **2.1 Site Location and Surrounds**

The site lies on Highway 72 approximately 1 km south of Sioux Lookout and approximately 58 km north of the intersection of Highway 72 and Highway 17. The site lies between Station 2+400 and Station 2+810, approximately, C/L Highway 72.

Locally, the site may be described as spanning the narrows between Pelican Lake and Abram Lake. The narrows have been partially filled by approach embankments, leaving a total bridge span of approximately 105 m. The fill



material visible on the exposed sides of the embankments consists of rock fill.

The only development in the immediate vicinity of the bridge consists of a fishing lodge on the north shore of Pelican Lake on the west side of the highway and a one storey house on the east of the highway approximately 100 m further north.

## **2.2 Physiography**

Physiographically, the area exhibits typical Canadian Shield characteristics. The bedrock in the area is igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The overburden generally consists of shallow glacial drift and outwash sands though clay and comparatively deep organic deposits can be encountered locally. Bare rock ridges commonly outcrop through the overburden.

Drainage immediately at the bridge site is expected to be good, with water levels being controlled directly by the adjacent lake levels.

Apart from localized clearings, the area is covered by boreal forest.

## **3 INVESTIGATION PROCEDURES**

The site investigation was carried out in two stages between December 18 and December 23, 2001, and between February 5 and February 14, 2002. The drilling was conducted by Paddock Drilling Ltd. of Brandon, Manitoba, using a crew and equipment mobilized from Thunder Bay, Ontario.

### **3.1 First Stage of Investigation**

In the first stage of investigation, three foundation investigation boreholes were drilled at both the north and south abutments of the existing bridge, as follows:

Borehole Number	Northing	Easting	Depth (m)	Location
BH 1	5548540	380121	16.9	South abutment
BH 3	5548643	380168	9.1	North abutment
BH 4	5548641	380174	9.2	North abutment

The equipment mobilized to site consisted of an Acker MP5 drill rig and the necessary drill tools and support equipment to complete the anticipated investigation.

Cased boreholes were advanced through the approach embankment rock fill using a 90 mm Stratex system of down-hole hammer drill and casing advancer. When bedrock was encountered, the hammer drill was withdrawn from the hole and investigation continued using NQ coring techniques. Each borehole was cored 3 m into bedrock, when encountered.

These boreholes were drilled through the asphalt pavement at distances of 2 to 5 m beyond the ends of the existing structure. These locations were considered suitable to confirm the anticipated condition of rock fill overlying bedrock.

Using this technique, two boreholes (BH 3 and BH 4) were successfully completed at the north abutment where bedrock was encountered at 0.2 to 0.9 m below the base of the rock fill. However, at the south abutment bedrock was not encountered immediately below the rock fill as expected and the one borehole drilled (BH 1) was advanced to a depth of 16.9 m. At that depth, the borehole had advanced beyond the available length of casing and problems were encountered due to sand washing into the down-the-hole hammer when drilling was paused to add additional lengths of drill rod. Consequently, the borehole had to be terminated at a depth of 16.9 m, while drilling in a layer of boulders.

In addition to the boreholes at the foundation locations, four solid stem auger boreholes were drilled to provide information for both analysis of the

embankment stability under a grade raise and for pavement design purposes.

### 3.2 Second Stage of Investigation

In the second stage of investigation, three foundation investigation boreholes were drilled at the south abutment and the centre pier foundation of the existing bridge, as follows:

Borehole Number	Northing	Easting	Depth (m)	Location
BH 2	5548546	380131	18.7	South abutment
BH 5	5548594	380145	12.7*	Centre pier, west side
BH 6	5548593	380153	14.2*	Centre pier, east side

\* Total depth from bridge deck.

In addition to the foundation boreholes described above, eight more boreholes were drilled to explore the nature and composition of the approach fills.

In view of the subsurface data obtained in the first stage of investigation, it was decided to advance the borehole at the south abutment using the diamond coring technique. It was anticipated that this technique would allow penetration of boulders and also allow SPT sampling through the drill string when required. In fact, SPT sampling was only possible at two depths, while the borehole was penetrating a layer of sand and gravel. At greater depths, the borehole encountered boulders in a sand and gravel matrix and constant problems were experienced with blocking and jamming of the core barrel as gravel and short pieces of core turned inside the core tube. Retrieval of the inner core tube invariably left gravel and core pieces in the bottom of the drill string where they blocked the passage of the SPT spoon. Consequently no SPT sampling was possible in this lower layer.

By a depth of 18.7 m in BH 2 there was no effective progress between one attempted core run and the next and the decision was made to terminate the borehole.

Coring equipment was also used for the investigation at the centre pier, where two boreholes were drilled from the bridge deck. The concrete deck was cored to permit the drill string to pass through to the underlying soil or rock fill. On completion of drilling the core holes in the deck were patched using rapid hardening concrete mix.

The drill rig used in the second stage of investigation was an Acker SX mounted on a rubber tired skidder tractor.

### **3.3 Field Supervision**

The drilling and coring procedures were supervised on a full time basis by a member of Thurber's engineering staff who recorded the material being encountered and the drill progress. The inspector also carried out the initial logging of the recovered rock core, recorded its location in the borehole, percent recovery and RQD and processed it for transport to Thurber's Toronto area laboratory.

In view of the site conditions, i.e. rock fill and coarse grained soils in a lake, it was decided that piezometers were not required at this site.

Traffic control during drilling was provided by Elk Construction of Ignace, Ontario, in accordance with the Temporary Conditions Manual, Book 7.

Boreholes in the road were backfilled with drill cuttings on completion and topped with cold mix asphalt. Holes cored in the concrete bridge deck were grouted on completion of the sampled borehole using a rapid-setting concrete mix.

The result of the drilling and coring are summarized on the borehole logs in Appendix A.



## **4 LABORATORY TESTING**

Gradation analyses was undertaken for selected samples recovered from the boreholes and the test results are shown in Figure B1, Appendix B.

Examination of the bedrock showed it to be fairly massive, granitic gneiss and no testing was deemed to be necessary.

## **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

The subsurface conditions encountered in the boreholes drilled for the structure foundations and the proposed grade raise for Highway 72 are described in the following paragraphs.

### **5.1 North Abutment**

The subsurface conditions at the north abutment are presented in the logs of Boreholes BH 3 and BH 4.

Each borehole encountered pavement structure extending to a depth of 600 mm below the road surface. The pavement consisted of 180 to 250 mm of asphaltic concrete overlying granular fill.

Below the granular fill, each borehole encountered rock fill extending to depths of 5.2 to 5.9 m below the road surface, corresponding to Elevation 356.7 under the SBL and Elevation 357.3 under the NBL. The rock fill consisted of fragments of hard, granitic gneiss rock up to approximately 600 mm in size. The rock was probably derived from nearby rock cuts on Highway 72.

Below the rock fill, each borehole encountered a layer of silt and gravel 230 to 860 mm thick.

Bedrock was encountered below the silt and gravel at Elevation 356.5 under the SBL and 356.4 under the NBL. The rock consists of hard, granitic

gneiss Precambrian rock. The core recovery was 100% and the RQD ranged from 16% at the bedrock surface to 50 to 74% below the surface.

## **5.2 Centre Pier**

Two core holes were drilled from the bridge deck through the foundation of the centre pier: Borehole BH 5 under the SBL and Borehole BH 6 under the NBL.

Under the SBL, i.e. the west end of the pier footing, the top of sand and gravel (and possibly some broken rock and construction debris) was encountered at Elevation 355.5 and the top of concrete at Elevation 355.0, the latter elevation corresponding to 8.68 m below the bridge deck. The top of sound bedrock was encountered at Elevation 353.0. However, the bottom of concrete was measured to be at Elevation 353.4 and there is apparently a gap between the base of the concrete and the top of the bedrock. This gap is apparently filled with cobbles and sand, based on the evidence of a piece of rock adhering to the bottom of the concrete and some sand recovered in the core barrel. The drill operator also noticed a pronounced drop in the drill string at this depth, indicating the presence of a void.

There are no known construction records that document how the footing was constructed but it is possible that the footing formwork was not effectively unwatered, making cleaning of the base difficult and resulting in some cobbles and sand and gravel being left in the base.

A piece of steel plate approximately 15 mm thick was recovered at the top of the first core run. It was not possible to determine if this was embedded in the concrete or lying loose on top.

Under the NBL, i.e. the east end of the footing (BH 6), the material on top of the footing was encountered at Elevation 355.5 and the top of concrete at 355.2, the latter elevation corresponding to 8.43 m below the bridge deck. The top of sound bedrock was encountered at Elevation 350.7. However, at Elevation 352.7 the borehole cut through a steel plate with a possible

300 mm gap under it. In this borehole, the concrete was found to be in tight contact with the bedrock, though not bonded to it.

These results show a difference in elevation of the top of bedrock of 2.2 m across the width of the structure.

The core recovery in the bedrock was 100% and the RQD was approximately 31 to 33%.

On completion of drilling, Borehole 6 was grouted using cement grout. Borehole 5 could not be grouted because the casing moved and the hole was lost.

### **5.3 South Abutment**

The subsurface conditions at the south abutment are presented in Boreholes BH 1 and BH 2.

In the initial investigation, BH 1 was drilled approximately 5 m south of the south end of the existing structure and in the SBL. In the later investigation, BH 2 was drilled at the proposed abutment location and in the NBL. Below the rock fill, both boreholes encountered essentially the same stratigraphy and Borehole BH 2 will be described as it is the closer to the actual foundation location.

Drilling started from the existing bridge deck at Elevation 364.62 and the top of the rock fill in the forward slope was encountered at Elevation 361.6. The rock fill consisted of fragments of hard, granitic gneiss up to approximately 600 mm in size which was probably derived from nearby rock cuts on Highway 72. At Elevation 356.7, the material encountered was identified as changing from rock fill to boulders in a sand and gravel matrix. This soil type continued to Elevation 353.7 where it changed to a layer of medium to coarse sand and gravel.

The sand and gravel layer was found to be approximately 3.7 m thick and to extend down to Elevation 350.0. Two SPT tests were carried out within this layer and gave results of 16 and 7 blows for 0.3 m of penetration, as shown

on the borehole log in Appendix A. However, considering the gradation of the sand and the depth of overburden above the sampling locations, these values may not reflect the true insitu density of the soil. It is believed that they have been influenced by disturbance due to water flow in the soil prior to sampling.

At Elevation 350.0, the borehole encountered a lower layer of boulders and cobbles in a sand and gravel matrix. The borehole was terminated in this layer at Elevation 345.9, a depth of 18.7 m below the bridge deck.

The cobbles and boulders are inferred to be up to 200 to 300 mm in size based on the largest pieces of core retrieved from this layer.

#### **5.4 Groundwater**

Groundwater level were not measured in the boreholes and no piezometers were installed. Given the nature of the soil, the groundwater level may be taken as the same as the lake level at any particular time.

At the time of drilling, the water level in the narrows was approximately Elevation 357.

The water level surveyed on October 23, 2001, was Elevation 357.1, normal water level is Elevation 357.7 and high water level is Elevation 359.1.

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## **6 INTRODUCTION**

The existing bridge is a four-span structure consisting of two through truss main spans and two steel girder approach spans. It is understood that the proposed replacement will consist of a two span steel girder bridge. The new bridge will be supported on new abutments to be constructed a short distance in front of the existing abutments and on the existing centre pier, which will be rehabilitated as part of the project. The foundations for the new abutments will be constructed in or through the rock fill in front of the existing abutments.

It is anticipated that in order to maintain the required navigational clearance under the bridge, the finished deck will be higher than the existing deck. The higher deck elevation will necessitate a grade raise of up to 0.75 m in the pavement profile of the approaches.

An investigation was conducted into the stratigraphy in the vicinity of the existing abutments and under the existing centre pier foundation. An investigation was also conducted into the composition of the approach fills. The results of the investigation are described in the earlier portions of this report, which also constitute the Foundation Investigation Report for the project.

The following sections describe the recommended foundation alternatives and corresponding geotechnical design parameters.

## 7 RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

A number of foundation alternatives have been considered for this project and these are described in the following sections together with comments regarding issues specific to each foundation element.

### 7.1 Spread Footing on Bedrock

A spread footing may be designed to bear on sound bedrock based on a factored geotechnical resistance at ULS of 2 000 kPa. The SLS condition does not govern for footings bearing on sound bedrock.

#### 7.1.1 North Abutment

The top of bedrock is expected to be at least 1 m below the water level at all times, based on the elevation data given in the factual section of the report. Constructing a footing to bear on the bedrock will require unwatering of the footing excavation to allow inspection of the base and placing the concrete in dry conditions. The means of unwatering should be left to the contractor, but probably will require some form of perimeter cutoff and pumping from within the excavation.

Alternatively, mass concrete fill may be poured on the bedrock and the footing be designed to bear on the mass concrete at a more convenient elevation, say Elevation 359. It is considered acceptable to place the mass concrete fill below water by tremie methods provided the contract can be written to require the contractor to demonstrate that the base of the excavation is in sound rock and that all earth material has been removed from the required bearing surface. The base of the mass concrete fill should project beyond the footing by a distance equal to 50% of the thickness of the mass concrete.

While this foundation system is geotechnically satisfactory, it is recognized that, for staged construction, roadway support will be required in the rock fill.

### 7.1.2 South Abutment

A spread footing on bedrock is not feasible at the south abutment.

## 7.2 Spread Footing on Granular Fill Pad

An alternative system to allow founding the footing at a higher elevation is to construct an engineered granular fill pad within the existing rock fill to support the footing. Assuming a granular pad constructed as described elsewhere in this report, the footing may be designed for maximum, vertical, geotechnical resistances as follows:

- factored ULS = 600 kPa
- SLS = 350 kPa

The above figures are considered to be the maximum permissible resistance at the top of the engineered fill pad. The loads from the fill pad will have to be resisted by the rock fill for which no placement data is available. In view of the lack of information, the vertical bearing resistance of the rock fill should be limited to 200 kPa at factored ULS and 120 kPa at SLS. The thickness of the granular pad must be sufficient to distribute the load so that the applied vertical stress on the rock fill does not exceed 200 kPa. In all cases, the pad should be at least 2m thick.

If this option is adopted, the foundations engineer should be closely involved in the development of the footing arrangement.

The granular fill pad should be constructed as described later and it must be totally wrapped in geotextile filter fabric and have adequate rip rap cover for scour protection.

Sliding resistance may be evaluated on the basis of an unfactored coefficient of 0.47 for a concrete footing poured against compacted Granular "A" or Granular "B" Type II.

While geotechnically feasible, this scheme also requires roadway protection to be constructed in the rock fill to accommodate staged construction.

It is not considered feasible to construct a granular pad starting from below the water level. Accordingly, the practical base elevation for the granular pad is restricted to Elevation 357 or higher. To maintain a minimum thickness of 2 m, the lowest permissible founding elevation for an abutment footing will be Elevation 359. If structural consideration require the founding elevation to be lower than 359, then this option is not feasible.

### **7.3 Steel Piles at North Abutment**

The north abutment foundation may be supported on steel piles founded on bedrock. H-section piles founded in sockets drilled into the bedrock are recommended.

A steel pile grouted into a bedrock socket can be designed on the basis of the factored structural resistance of the pile at ULS. For a pile founded in sound bedrock, the SLS case will not govern.

It will not be feasible to drive the piles through the existing rock fill in the forward slope of the approach embankment and another method of installation will be necessary. The Contractor should provide details of the installation method, subject to approval by the Ministry. However, a typical sequence of installation might be as described below:

1. Advance a suitable diameter steel liner through the rock fill and underlying soil and seat into the bedrock. The liner diameter should be specified to accommodate the design flexure of the selected pile section and have allowance for some "drift" as the liner is installed.
2. Various methods of advancing the liner would be acceptable, for example:
  - fit liner with a carbide drill shoe and drill it into place then clean out using down the hole hammer or other suitable means
  - fit liner with a carbide shoe and drill in using rotary drill with simultaneous casing advancement (well drilling rig)
  - use a driven casing following churn drilling



(The contractor may elect to use another system, subject to Ministry review).

3. After the liner is seated in bedrock, advance a socket into the bedrock to receive the H-pile. The socket may be advanced by a suitable drilling method such as down the hole hammer or rotary drilling.
4. The diameter of the socket should be large enough to accommodate the pile, allow for centring and allow for grouting.

Since it will not be possible to inspect the bedrock as the socket is being advanced, a minimum socket depth of 1 m is recommended. Deeper sockets may be used if required by structural considerations.

The required depth of socket into bedrock may be a function of the type of abutment designed for the structure in that minimum free lengths of pile are required for integral abutment design.

#### **7.4 Steel Piles at South Abutment**

The south abutment foundation may be supported on steel piles. As described below, an HP 310X174 section is recommended to withstand anticipated hard driving and to provide a larger end bearing area.

Due to the limitations of the drilling program, stratigraphic data is available only down to a depth of 18.7 m below the existing bridge deck, or to Elevation 345.9 at BH 2.

It will not be feasible to drive the piles through the existing rock fill in the forward slope of the approach embankment or the possible bouldery layer immediately below the fill and another method of installation will be necessary. The Contractor should provide details of the installation method, subject to approval by the Ministry. However, a typical sequence of installation might be as described below:

1. Advance a suitable diameter steel liner through the rock fill and underlying boulders, stopping when the liner has penetrated through

the boulders and into the underlying sand or when the tip of the liner is at Elevation 354.0. The liner diameter should be specified to accommodate the design flexure of the selected pile section and have allowance for some "drift" as the liner is installed.

2. Various methods of advancing the liner would be acceptable, for example:

- fit liner with a carbide drill shoe and drill it into place then clean out using down the hole hammer or other suitable means
- fit liner with a carbide shoe and drill in using rotary drill with simultaneous casing advancement (well drilling rig)
- use a driven casing following churn drilling

(The contractor may elect to use another system, subject to Ministry review).

3. After the liner is in position place the pile inside the liner and drive as described later under "Control of Pile Driving".

#### 7.4.1 Axial Pile Resistance

The axial geotechnical resistance of an HP 310X174 pile was analyzed using the shear strength, effective stress, approach. The first analysis conducted was for a pile that would be founded within the depth of exploration in Borehole BH 02-2, using the following assumptions:

Underside of abutment stem/top of pile	Elevation 361
Pile section	HP 310X174
Total length of pile	11 to 12 m (0 to 1 m penetration into the lower boulders)
Length of shaft ignored (from top of pile)	7 m (length of liner)
Angle of friction of soil on shaft	30° (sand)
Angle of friction at tip	41° (cobbles and boulders in sand and gravel)

Based on the above assumptions, the analysis gives the ultimate vertical resistance of the pile as **3 100 kN**.

Applying a resistance factor of 0.4 and assuming settlements should be limited to less than 25 mm gives the following values to be used in design:

- factored geotechnical resistance at **ULS 1 250 kN**
- resistance at **SLS 1 000 kN**

This analysis has been conducted for an 11 m long pile, i.e. assuming only nominal penetration into the lower boulder layer before the piles meet refusal. In practice, some or all of the piles may penetrate to a greater depth.

#### 7.4.2 Control of Pile Driving

Driving of the HP310 X 174 piles should be carried out using a pile driver with a minimum rated energy of 95 kJ per blow.

Control of pile driving in the field should be in accordance with Note 2 of the Structural Manual.

Piles should be driven at least to Elevation 350 and then driving should be controlled using the Hiley Formula with an ultimate resistance of 2500 kN.

### 7.5 Conventional vs Integral Abutment

If a conventional or a semi-integral abutment design is selected, no pile flexibility is required. Accordingly the full length of the space between the pile and the liner (and the full depth of the rock socket at the north abutment) can be grouted up to the underside of the abutment stem.

If an integral abutment design is selected, the depth of rock socket or length of liner, must be sufficient to accommodate the required free length of pile for flexibility.

From a foundation design perspective, the flexible length of pile can be left free inside the liner, assuming that the liner is left in place (recommended). The space can also be filled with uniform sand as described in the Integral Abutment Report if this is advantageous from a structural perspective. The

sand should be of uniform grain size and meet the gradation requirement given in the Integral Abutment Bridges report.

It is anticipated that at least the lower portion of the annular space around the pile will be permanently full of water or saturated sand. See the section "Frost Protection" later in this report.

#### 7.5.1 North Abutment

Whichever abutment type is selected, the socket should be finished as free of drill cuttings as is practicable. The pile should be placed in hard contact with the bedrock in the bottom of the socket, centred on the abutment and then grouted using 30 Mpa concrete.

For a semi-integral abutment design, the depth of concrete at the bottom of the socket should be limited as follows:

- 300 mm if a pin joint is assumed
- at least 500 mm if fixity is required

#### 7.5.2 South Abutment

If an integral abutment design is selected, the flexibility of the pile must be maintained. In this case, it is recommended that no concrete be placed in the liner, except as a means of centring the pile as described below.

It is anticipated that driven piles may tend to drift from location as they are driven through some of the strata identified at this site. To assist in maintaining the location, it is acceptable to place a plug of concrete around the base of the pile, inside the liner, prior to driving. The length of the concrete plug should be approximately 1.0 m and the pile must be wrapped in a material that will:

- Prevent the concrete from adhering to the pile
- Nor hinder the pile driving
- Allow for some lateral movement of the pile under service loads

A suitable material would be 25 mm thick expanded polystyrene secured to the pile prior to placing the pile in the liner.

### 7.5.3 Pile Tip Protection

Piles driven at the south abutment should be provided with tip protection for driving through boulders and to improve contact with bedrock in the event bedrock is encountered. Protection should take the form of Titus Steel Pile Points, e.g. Point No: HPP-R-12 for the HP 310X174 piles.

### 7.5.4 Lateral Pile Deflection

In computing the lateral deflection behaviour of the piles at the south abutment, the following values of the horizontal modulus of subgrade reaction ( $k_s$ ) may be used:

Material	$k_s$ (kN/m <sup>3</sup> )
Rock fill	80,000
Sand	40,000

Calculations carried out by Thurber show that effective fixity of the piles at the south abutment is achieved at a depth of 9 m below the top of the pile, assuming a 7 m free length of the pile.

## 7.6 Centre Pier

As reported in the Foundation Investigation section of the report, there is a possibility of discontinuous gaps between the existing concrete footing and the underlying bedrock. The extent of these gaps cannot be determined on the basis of the investigation completed at the centre pier. However, it is recommended that grout injection be used to fill any voids below the existing footing and possibly consolidate the material that is present.

This course of action is considered warranted because:

- the pier will remain in service
- the loads from the new structure will be greater than from the existing structure
- the west end of the footing will likely experience higher loading during staged construction.

A grouting program should be designed that includes a number of injection holes and observation holes. It is recommended that these holes be drilled through the footing and into the bedrock and that the progress of drilling be monitored for evidence of voids between the concrete and the bedrock. Any evidence of voids should be used during construction to delineate voids and to possibly modify the sequence of grouting.

A grid pattern of holes across the entire exposed top of the concrete footing is recommended to explore for possible voids and to allow for them to be grouted.

The hardened grout must have sufficient strength to withstand the loads imposed by the superstructure and substructure, taking account of the worst loading condition.

It is not possible to predict the volume of grout take and in practice it could vary from practically nil to several cubic metres, depending on the volume of voids and volume of grout loss beyond the immediate footprint of the footing. The contract should be structured accordingly.

Discharge of grout into a waterway is not environmentally acceptable and a system of monitoring and/or positive containment of the grout is required. Continuous inspection by a diver during the grouting operations may be appropriate.

## 7.7 Engineered Granular Fill Pad

Engineered granular fill pads should be designed to replace as great a depth of the existing rock fill as is practicable. Factors that may limit the depth of excavation and therefore the depth of replacement include:

- Limitations imposed by staging
- Roadway protection requirements related to the support of the sides of the excavation
- The water level in the narrows at the time of excavation

It is not practical to construct a compacted granular fill below the level of the water at the time of construction. For design purposes, the water level in the narrows may be taken to be Elevation 357.7.

The material used to construct the engineered fill pads should consist of OPSS Granular "A". The fill should be placed in lifts not exceeding 250 mm, loose thickness, and then be compacted to 100% of the SPMDD at a moisture content within 2% of optimum.

A typical section of an engineered fill pad is shown in Figure 1.

For this site, where the engineered fill will be placed over rock fill and in a location exposed to scour, specific procedures will be required as described below.

### 7.7.1 Preparation of Base

The base on which the engineered fill will be placed will consist of the existing rock fill. Excavation to the design level for the base of the granular fill (Elevation 357.7) must be carried out in a manner that will minimize the disturbance to the base and this may require equipment other than a conventional backhoe.

The Contractor should select his method of excavation based on his equipment and experience and submit his excavation proposal to the Ministry for comment.

After excavation, the exposed base must be thoroughly chinked and then be recompacted as thoroughly as is practical.

#### 7.7.2 Prevention of Loss of Fines

It is important that the engineered fill that will support foundation loads retains its design configuration and gradation characteristics and remains in a dense state through the life of the structure. Accordingly, steps must be taken to prevent loss of material either from erosion or loss into voids in the rock fill.

Prevention of loss of material from within the engineered fill mass may be accomplished by wrapping the entire mass of fill in a non-woven geotextile filter cloth. Specifications for the filter cloth are:

- Non-woven
- Class II
- Elongation % at break 70%
- F.O.S. 50 to 100  $\mu$ m

The filter cloth must be placed across the entire chinked rock fill base, up all sides and be wrapped around the top and anchored under the concrete footing to totally enclose the fill. The filter cloth must be installed according to the manufacturer's instructions and with adequate overlap to ensure the continuity of the filter during and after construction and throughout the service life of the structure.

#### 7.7.3 Erosion Protection

The forward slopes at the abutments will be exposed to erosive forces from the water flow through the rapids and it is important the engineered fill be protected from erosion.



Prevention of erosion by flowing water should be accomplished through the provision of adequate rip rap protection. The size of rip rap and the thickness of the rip rap layer should be determined in consultation with a river hydrologist.

## 8 ROADWAY PROTECTION

During staged construction, one lane of traffic must be maintained. Implicit in maintaining traffic is the protection of half the width of the existing abutment footing. If the spread footing option is selected for the new abutments, construction of the foundations will require a significant amount of excavation that will undermine the roadway and the existing footing that must be kept in service.

The roadway protection scheme should be the responsibility of the Contractor and should be designed by a Professional Engineer specializing in that field, subject to comment by the Ministry.

Typical roadway protection schemes that could be considered include:

- some form of soldier pile and lagging or contiguous caisson wall
- shotcrete and tiebacks

The installation of soldier piles or caissons would be difficult and expensive as each one would have to be drilled in through the rock fill. The toes of the drilled holes would terminate below water in permeable soil, making concreting difficult. The task of excavating neatly between soldier piles to install lagging would also be very difficult.

For the shotcrete system, it is envisaged that a near vertical face could be excavated in stages, shotcreted and tied back to provide support. Typically, the rock fill could be excavated in 500 mm lifts using very careful excavation techniques to avoid undermining the roadway. As each lift is exposed, it should be shotcreted and tied back prior to proceeding to the next lift. On this site it may be most practical to drill horizontally through the rock fill and anchor to plates on the opposite side of the embankment.

The roadway protection should be designed to support horizontal loads determined on the basis of the parameters given in the following section.

## 9 EARTH PRESSURE

The lateral earth pressures to be used in design should be computed in accordance with Section 6-7 of the OHBDC .

Granular backfill should be placed behind the abutment walls and wing walls to conform to the minimum requirements illustrated in OPSD 3501.00. The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular A or Granular B Type I.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table below.

$\gamma$  = unit weight of soil = 23 kN/m<sup>3</sup> for Granular A  
21.2 kN/m<sup>3</sup> for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)			
	OPSS Granular A $\phi' = 35^\circ$		OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.43	-	0.50	-
Wing Walls (Unrestrained Wall)	0.27	0.40	0.33	0.55

For the design of roadway protection in rock fill, a coefficient of active earth pressure of 0.20 may be used.

If an integral abutment design is used, the abutments will be cast integrally with the deck and therefore the abutment walls should be treated as restrained. If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained. The above also assumes a horizontal ground surface behind the abutment walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

Design lateral pressures at any depth should not be less than 16 kPa to account for compaction induced forces.

## 10 GRADE RAISE

It is understood that the maximum grade raise will be no more than 0.75 m.

The existing embankment consists of an earth fill core within a rock fill shell, as determined in the course of the investigation. Based on the findings in the boreholes, rock fill comprises the full width of the embankment for approximately 20 m beyond the north abutment and 10 m beyond the south abutment. Further from the structure, the boreholes indicate that the core of the embankment is composed mainly of earth fill.

Projecting the existing side slopes up to the new grade results in a platform that is too narrow to accommodate the planned new roadway. The options to accommodate the required new platform are:

- widen the entire embankment, including encroaching into the lake
- achieve the grade raise on the existing platform using some form of retaining structure

Since encroachment into the lake is not acceptable, the grade raise must be accomplished using some form of retaining structure. In view of the rock fill shell

and the low height of the grade raise, the most practical solutions would appear to be small cantilevered concrete walls, a retained soil system (RSS) or a proprietary bin wall system.

The concrete walls could be cast in place or supplied as precast sections. In either case, the wall should be L-shaped cantilever retaining wall with the vertical leg retaining the fill and the horizontal leg returning under the fill to provide resistance to sliding and overturning. Horizontal forces may be computed using the earth pressure parameters given in the previous section. Sliding resistance may be checked using the following unfactored friction coefficients:

- precast concrete 0.40
- cast in place concrete 0.55

If an RSS is selected, it should be a proprietary system from the Designated Suppliers List. The wall should be specified to be high performance and high appearance.

The bin wall may be proprietary system from an approved supplier. Materials suitable to resist the local environment, including the effects of road salt should be specified.

Where granular fill may be placed over existing rock fill, either in a bin wall, an RSS or as bedding for precast wall sections, the exposed surface of the rock fill must be carefully chinked and the chinked surface covered by a synthetic filter fabric. Specifications for the filter cloth are:

- Non-woven
- Class II
- Elongation % at break 70%
- F.O.S. 50 to 100  $\mu$ m

## 11 EMBANKMENT STABILITY

Visual examination of the approach embankments did not reveal any evidence of instability and Thurber has not been made aware of any past stability problems. The stability of the embankment after construction of the grade raise has been analyzed using Gslope® by Mitre Software.

The following parameters were assumed for the stability analysis.

Material	Friction Angle	Cohesion	Unit Weight
Rock fill	47°	0 kPa	21
Sand and gravel	35°	0 kPa	21
Earth Fill	33°	0 kPa	19
Native soil	42°	0 kPa	22

### 11.1 North Approach

The north approach embankment is approximately 80 m long.

The section selected for stability analysis is at Stn 2+656, where it appears there is the deepest earth fill and a relatively high slope on the west side. It was assumed that the approach embankment consisted of earth fill with a rock fill shell approximately 2.5 m thick.

Using the assumed parameters, stability analysis of this portion of the approach yields the following results:

- factor of safety in existing configuration 1.38
- factor of safety after 0.75 m grade raise 1.32

These safety factors are considered to be acceptable.

## 11.2 South Approach

The north approach embankment is approximately 30 m long.

Borehole BH 1 encountered 600 mm of pavement structure over rock fill and Borehole BH 8, drilled 11 m to the south, encountered 2.1 m of pavement structure and sand before encountering refusal on rock fill or bedrock. Refusal in the latter borehole was confirmed at 2.1 m by augering a second probe hole approximately 1 m to the north.

In the south approach, only the geotechnical borehole drilled approximately 2 m west of BH 8, in the SBL shoulder, penetrated earth fill. This hole was augered to a depth of 6.1 m without encountering refusal.

The data obtained is interpreted as indicating that the highest and steepest portion of the south approach, i.e. everything north of Stn. 2+500, consists of rock fill overlying non-cohesive soils.

Stability analysis of this portion of the approach yields the following results:

- factor of safety in existing configuration 1.40
- factor of safety after 0.75 m grade raise 1.38

The reduction in factor of safety is not considered to be significant and it is concluded that a grade raise of up to 0.75 m will not destabilize this portion of the approach.

In the vicinity of the pavement borehole at 2+493, 3.5 Lt, a rock fill shell approximately 2.5 m thick over the earth fill was assumed for the analysis. It was also assumed that the pavement structure was 1.5 m thick.

Based on analysis of the selected section representing these conditions, the following results were obtained:

- factor of safety in existing configuration 1.66
- factor of safety after 0.75 m grade raise 1.57

Thus, it can be seen that there will be a small decrease in the calculated factor of safety. The conditions used for this analysis are considered to represent the probable worst conditions in the south approach. The final value of 1.57 for the factor of safety is considered to be acceptable.

## 12 SETTLEMENT

A grade raise of up to 0.75 m is anticipated in the final design, which will increase the loading on the existing embankment fill.

The findings from the boreholes indicate that rock fill comprises the full width of the embankment for approximately 20 m beyond the north abutment and 10 m beyond the south abutment. Further from the structure, the boreholes indicate that the core of the embankment is composed mainly of earth fill.

Planning for the grade raise should be carried out taking into account the recommendations of the Pavement Design Report, issued separately.

Based on the soil stratigraphy encountered in the approach fills, the assessed settlements under the grade increase are small. The anticipated maximum settlement under the added loading from the grade raise is in the order of 15 to 20 mm.

It is recommended that the fill for the grade raise be placed as early in the construction program as is feasible within the staged program. This should allow sufficient time for the settlements to be essentially completed prior to paving.

### 13 FROST PROTECTION

The design depth of frost penetration for this project is 2.5 m.

If spread footings bearing on earth or engineered granular fill are used, then the base of the footing should be provided with at least 2.5 m of soil cover as frost protection.

For the case of piled foundations where the underside of the abutment stem or pile cap will be perched in the rock fill above water level, frost protection is not considered necessary from the point of view of preventing frost jacking under the concrete. However, in the case of an integral abutment bridge, where flexibility of the pile is important, it is recommended that sufficient frost protection be provided to prevent the water freezing around the pile.

Where 2.5 m of soil cover cannot be provided, the equivalent protection may be provided using synthetic insulation such as extruded rigid polystyrene, where 25 mm of extruded rigid polystyrene is equivalent to 600 mm of earth cover. It should also be noted that rock fill does not provide frost protection equivalent to the same thickness of earth fill. Circulation of air in the voids present in rock fill can lead to greater vertical penetration of frost into rock fill.

A suitable arrangement for insulation would consist of extruded rigid polystyrene insulation placed under the adjacent slope and extending to 2.5 m beyond the foundation element. The thickness of insulation should be chosen on the basis of the thickness of soil cover being replaced.



## 14 CONSTRUCTION CONCERNS

Construction concerns on this site relate to the anticipated difficulties a contractor may have in implementing the foundation recommendations, especially on a relatively remote site.

### 14.1 Spread Footing

The following items represent points of concern for the construction of spread footings on engineered fill:

- Roadway Protection - a major issue for the construction of spread footings is the provision of adequate roadway protection that can be installed without undermining the remainder of the abutment footing or the traveled roadway. This will be a particularly difficult and expensive operation for the contractor and must be carefully planned and executed.
- Excavation to Construct Engineered Fill - this excavation will proceed in conjunction with the provision of roadway protection and must not undermine the portion of footing to be retained in service or the traveled roadway. The excavation must also be carried out in a manner to minimize the disturbance of the exposed rock fill subgrade. Excavation using a conventional backhoe may not be satisfactory and other equipment such as a clamshell or grab excavator may be necessary.
- Protection of the Engineered Fill - the engineered fill must be protected from loss by surface erosion or the migration of the granular fill into voids in the rock fill. Protection should be accomplished by thoroughly chinking the rock fill subgrade and completely wrapping the fill in a strong geotextile filter. Construction of the fill must be planned and executed in a manner that will ensure overlaps in the filter fabric are maintained and not compromised by construction activities.

The construction sequence should permit placement of rip rap protection up to the level of the top of the engineered fill as soon as the fill has been completed.

## **14.2 Piled Foundations**

Construction concerns related to pile installation include:

- Positioning and “drift” of the liner to be advanced through the rock fill
- Positioning of the pile within the liner and “drift” during driving
- Cleaning of drill cuttings from the bottom of rock sockets at the north abutment
- Penetration of the piles to the required minimum depth and, conversely, the development of sufficient resistance if the piles penetrate further than anticipated

## **15 CONSTRUCTION INSPECTION AND MONITORING**

During construction, all foundation installation, excavation and fill construction activities should be monitored by geotechnical personnel to confirm that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

- - End of Text - -

# STATEMENT OF GENERAL CONDITIONS

## **1. STANDARD OF CARE**

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

## **2. COMPLETE REPORT**

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

## **3. BASIS OF REPORT**

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

## **4. USE OF THE REPORT**

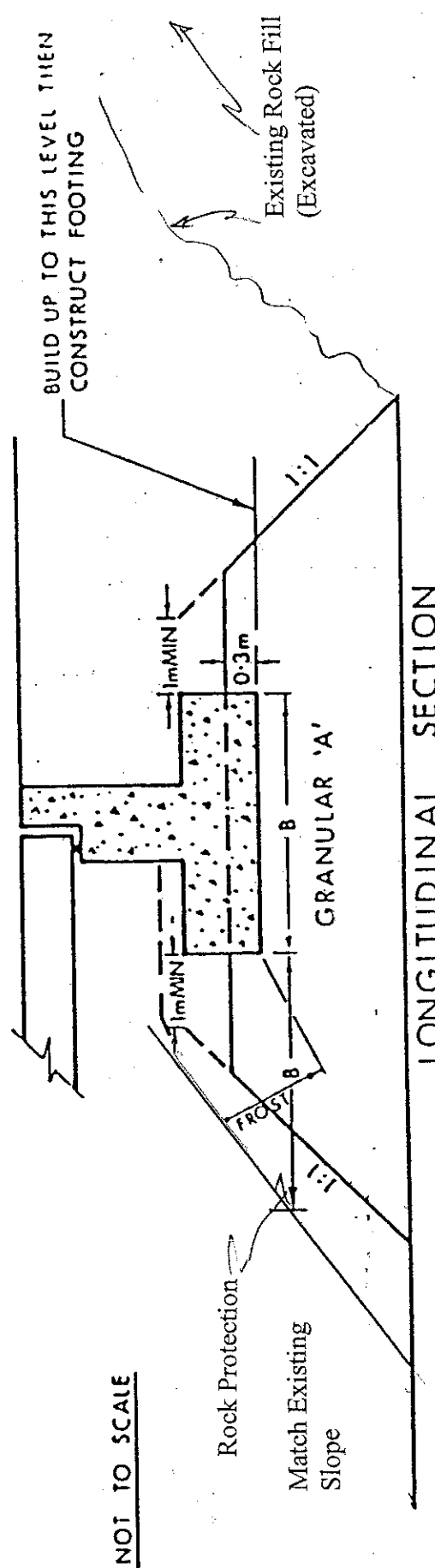
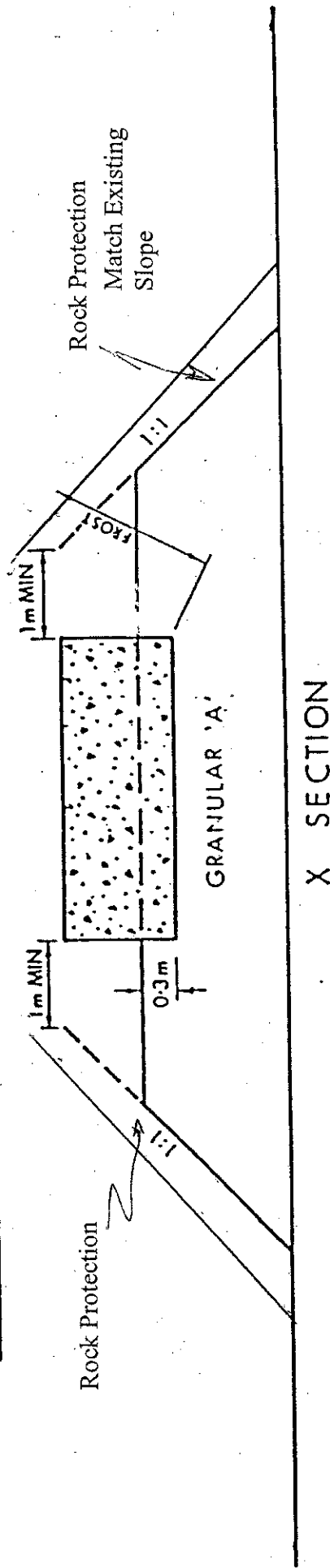
The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

## **5. INTERPRETATION OF THE REPORT**

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

# ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



NOT TO SCALE

## NOTES

- 1) Excavate existing rock fill as described in the report
- 2) Chink exposed surface and cover with geotextile
- 3) Place Granular "A" fill as directed, protecting with geotextile filter cloth
- 4) Place rock protection as specified
- 5) Place remainder of fill

**Schematic Illustration Only**

## SYMBOLS AND TERMS USED ON TEST HOLE LOGS

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <sup>(1)</sup> "N" VALUE
Very Soft	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR	 Shelby Tube	A - Casing
SAMPLE TYPE	 SPT	 Grab/Auger sample
	 No Recovery	 Core

- MC – Moisture Content (% by Weight) as determined by sample

 Water Level

C <sub>vane</sub>	Shear Strength Determination by Field Insitu Vane
C <sub>pen</sub>	Shear Strength Determination by Pocket Penetrometer
C <sub>lab</sub>	Shear Strength Determination using a Laboratory Vane Apparatus
C <sub>U</sub>	Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test – refers to the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

# UNIFIED SOILS CLASSIFICATION

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

# RECORD OF BOREHOLE No BH-1

1 OF 2

METRIC

W.P. 452-00-00 LOCATION N 5548539.839 E 380120.974 ORIGINATED BY MF  
DIST Kenora HWY 72 BOREHOLE TYPE Down Hole Hammer Drill COMPILED BY WM  
DATUM Geodetic DATE 19.12.01 - 21.12.01 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
364.8							20	40	60	80	100		
364.0	ASPHALT (225mm)						20	40	60	80	100		
0.2	GRANULAR FILL						20	40	60	80	100		
364.2	ROCK FILL						20	40	60	80	100		
0.6	( up to 600 mm in size )						20	40	60	80	100		

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

ORIGINATED BY MF

COMPILED BY WM

CHECKED BY      AEG

+ 3, × 3: Numbers refer to Sensitivity



METRIC

W.P.	452-00-00	LOCATION	N 5548545.483 E 380130.703	ORIGINATED BY	MF
DIST	Kenora	HWY	72	BOREHOLE TYPE	NQ Core Barrel
DATUM	Geodetic	DATE	07.02.02 - 14.02.02	COMPILED BY	WM
				CHECKED BY	AEG

[illegible]

Continued Next Page

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

(%) STRAIN AT FAILURE

**METRIC**[illegible]

# RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 452-00-00 LOCATION N 5548643.066 E 380168.112 ORIGINATED BY MF  
 DIST Kenora HWY 72 BOREHOLE TYPE Air Hammer / Core Barrel COMPILED BY WM  
 DATUM Geodetic DATE 18.12.01 - 18.12.01 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	W <sub>P</sub>	W	W <sub>L</sub>			
362.6							20 40 60 80 100							
362.9	ASPHALT (250mm)													
0.3	GRANULAR FILL													
362.0	ROCKFILL													
0.6	( up to 600 mm in size )													
356.7														
5.9	GRAVEL													
356.5	BEDROCK- granitic gneiss, grey													
6.1	CORE #1 (RQD=16.1%, TCR=100%)		1	CORE										
	CORE#2 (RQD=74.2%, TCR=100%)		2	CORE										
353.4	CORE#3 (RQD=53.6%, TCR=100%)		3	CORE										
9.1	END OF THE BOREHOLE AT 9.14m. BOREHOLE DRY ON COMPLETION. BOREHOLE BACKFILLED WITH GRAVEL.													

# RECORD OF BOREHOLE No BH-4

1 OF 1

METRIC

W.P. 452-00-00 LOCATION N 5548641.293 E 380174.001 ORIGINATED BY MF  
 DIST Kenora HWY 72 BOREHOLE TYPE Air Hammer / Core Barrel COMPILED BY WM  
 DATUM Geodetic DATE 22.12.01 - 22.12.01 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
362.5								20	40	60	80	100		
360.0	ASPHALT (180mm)													
0.2	GRANULAR FILL													
361.9	ROCKFILL													
0.6	( up to 600 mm in size )													

## METRIC

+ 3, × 3: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No BH-7

1 OF 1

METRIC

W.P. 452-00-00 LOCATION N 5548678.012 E 380185.668 ORIGINATED BY MF  
 DIST Kenora HWY 72 BOREHOLE TYPE 100mm SOLID STEM AUGERS COMPILED BY WM  
 DATUM Geodetic DATE 11.02.02 - 11.02.02 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
362.0								20	40	60	80	100					
361.8	ASPHALT (100mm)																
0.1	SAND and GRAVEL, brown, dry																17 65 18
361.2																	
0.8	SILT, clayey, some sand, trace gravel, hard, brown: (FILL.)		1	SS	50		361										
360.4																	
1.6	SAND, trace to some gravel, dense to loose, brown: (SP)		2	SS	35		360										24 70 6
			3	SS	6		359										
			4	SS	4		358										17 79 4
357.6			5	SS	4												
4.4	END OF THE BOREHOLE AT 4.42m. BOREHOLE OPEN TO 4.42m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																

# RECORD OF BOREHOLE No BH-8

1 OF 1

METRIC

W.P. 452-00-00 LOCATION N 5548529.005 E 380117.415 ORIGINATED BY MF  
 DIST Kenora HWY 72 BOREHOLE TYPE 100mm SOLID STEM AUGERS COMPILED BY WM  
 DATUM Geodetic DATE 11.02.02 - 11.02.02 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
365.2														
366.0	ASPHALT (175mm)													
0.2	SAND and GRAVEL, compact, brown: (FILL)		1	SS	24		365							15 74 12
363.8							364							20 72 9
1.5	SAND, coarse grained, trace to some gravel, compact, brown		2	SS	18									
363.1														
2.1	END OF THE BOREHOLE AT 2.13m ON ASSUMED ROCKFILL OR BOULDER. REFUSAL AT 2.13m. BOREHOLE OPEN TO 2.13m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.													

+ 3, X 3; Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE





# RECORD OF BOREHOLE No BH-9

1 OF 1

METRIC

W.P. 452-00-00 LOCATION N 5548503.594 E 380103.579 ORIGINATED BY MF  
 DIST Kenora HWY 72 BOREHOLE TYPE 100mm SOLID STEM AUGERS COMPILED BY WM  
 DATUM Geodetic DATE 11.02.02 - 11.02.02 CHECKED BY AEG

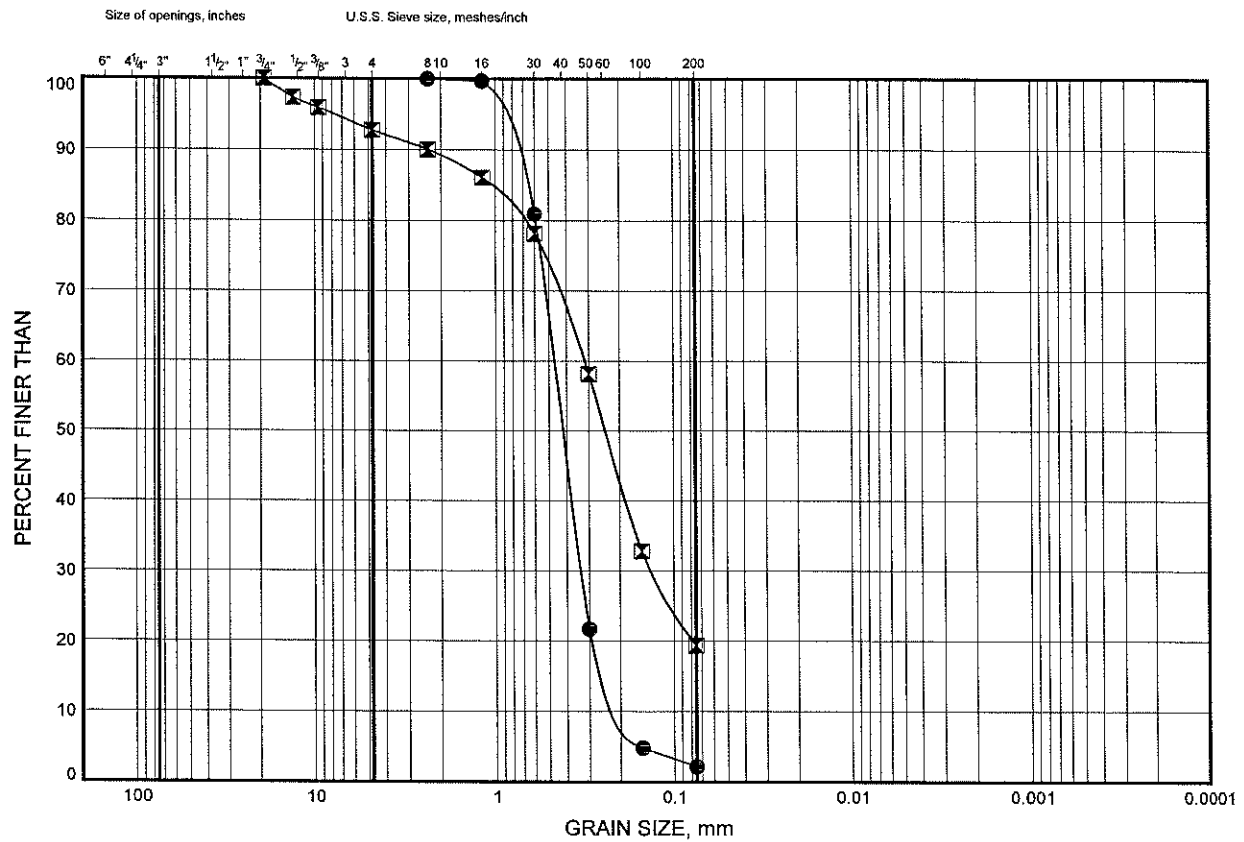
SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa												
366.4								20	40	60	80	100						
366.3	ASPHALT (125mm)																	
0.1	SAND and GRAVEL, brown: (FILL)																7 73 19	
365.6																		
0.8	END OF THE BOREHOLE AT 0.76m. REFUSAL AT 0.76m ON ASSUMED BEDROCK. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																	

**APPENDIX B**

**LABORATORY TESTING**

# Frog Rapids Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

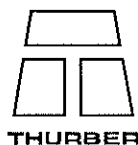


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BH-2	13.72	350.90
×	BH-9	0.46	365.92

SAND, Trace Gravel, Trace to Some Silt

Date June 2002  
Project 452-00-00

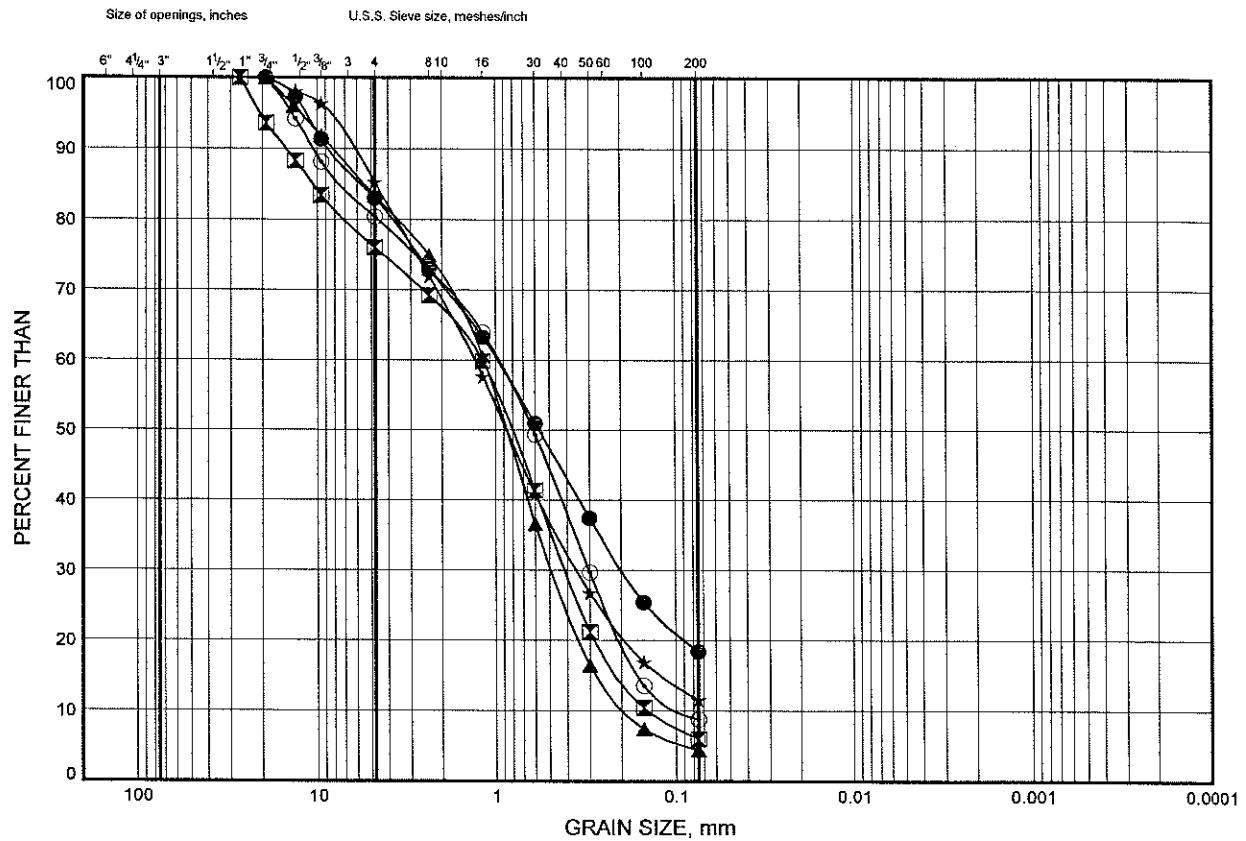


Prep'd WM  
Chkd. AEG

# Frog Rapids Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE B2



**APPENDIX C**

**PAVEMENT BOREHOLE LOGS**

2+344      2.8 LT Centreline  
Note:      Centre of Southbound lane

0	-	230	Asph
230	-	910	Br Gr(y) F Sa some Si - Si(y), moist
		910	NFP BR

2+463      2.2 RT Centreline  
Note:      Centre of Northbound lane

0	-	255	Asph
255	-	610	Br Gr(y) F Sa some Si - Si(y), moist
610	-	1.52	Br F Sa some Gr some Si - Si(y), moist

2+464      5.2 LT Centreline  
Note:      Shoulder of Southbound lane

0	-	125	Asph	
125	-	760	Br Gr(y) F Sa some Si - Si(y), moist	02TEL01
		760	NFP Poss RF Poss BR	

Sample 02TEL01 (460)

% Passing 4.75 mm	93 %
% Passing 75 µm	19 %

2+493 - 1.6 LT Centreline  
Note: Centre of Southbound Lane

0 - 175 Asph  
175 - 1.5 Br Sa and Gr  
1.5 - 2.1 Br Sa some Gr  
2.1 NFP Poss RF

02TEL02, 02TEL03

Sample 02TEL02 (460)

% Passing 4.75 mm 85 %  
% Passing 75 µm 12 %

Sample 02TEL03 (1.07)

% Passing 4.75 mm 80 %  
% Passing 75 µm 9 %

2+493 4.3 LT Centreline  
Note: Shoulder of Southbound lane

0 - 115 Asph  
115 - 3.96 Br Gr(y) F Sa some Si - Si(y), moist  
3.96 - 4.27 Br Cl(y) Si some Sa Tr Gr, moist  
4.27 - 6.10 Br Gr(y) F Sa some Si - Si(y), moist

2+627 4.0 LT Centreline  
Note: Shoulder of Southbound lane

0 - 125 Asph  
125 - 610 Br Gr(y) F Sa some Si - Si(y), moist  
610 - 1.37 Poss RF  
1.37 NFP Poss RF

2+636        4.2 LT Centreline  
Note:        Shoulder of Southbound lane

0	-	70	Asph
70	-	610	Br Gr(y) F Sa some Si - Si(y), moist
610	-	1.22	Poss RF
		1.22	NFP Poss RF

2+636        4.2 RT Centreline  
Note:        Shoulder of Northbound lane

0	-	75	Asph
75	-	610	Br Gr(y) F Sa some Si - Si(y), moist
		610	NFP Poss RF

2+657 -        1.9 LT Centreline  
Note:        Centre Southbound lane

0	-	100	Asph
100	-	800	Br Sa and Gr
800	-	1.6	Cl(y) Si Tr Sa
1.6	-	4.4	Br Sa some Gr

02TEL04

02TEL05, 02TEL06

Sample 02TEL04 (460)

% Passing 4.75 mm	83 %
% Passing 75 µm	19 %

Sample 02TEL05 (1.83)

% Passing 4.75 mm	76 %
% Passing 75 µm	6 %



Sample 02TEL06 (3.35)

% Passing 4.75 mm            83 %  
% Passing 75 µm            5 %

2+657            4.4 LT Centreline  
Note:            Shoulder of Southbound lane

0	-	75	Asph
75	-	1.07	Br Gr(y) F Sa some Si – Si(y), moist
1.07	-	2.29	Br Cl(y) Si some Sa Tr Gr, moist
2.29	-	4.57	Br F Sa some Si some Gr OCC Cl OCC Cob, moist
4.57	-	6.40	Br Co Sa some Gr Tr Si, moist
6.40	-	7.01	Poss RF
		7.01	NFP Poss BR

2+671            4.5 LT Centreline  
Note:            Shoulder of Southbound lane

0	-	70	Asph
70	-	910	Br Gr(y) F Sa some Si – Si(y), moist
910	-	1.22	Blk Sa W Tps some Gr OCC Cl, moist
1.22	-	1.68	Br Cl(y) Si some Sa Tr Gr, moist
1.68	-	5.38	Br F Sa some Si some Gr OCC Cl OCC Cob, moist
		5.38	NFP Poss BR

2+671            2.6 LT Centreline  
Note:            Centre of Southbound lane

0	-	230	Asph
230	-	740	Br Gr(y) F Sa some Si – Si(y), moist
740	-	1.52	Br Si(y) Cl, moist

2+691            3.8 LT Centreline  
Note:            Shoulder of Southbound lane

0	-	70	Asph
70	-	1.07	Br Gr(y) F Sa some Si – Si(y), moist
1.07	-	1.52	Blk Sa W Tps some Gr OCC Cl, moist
1.52	-	5.03	Br F Sa Tr Gr, moist
		5.03	Poss BR

2+777            2.4 RT Centreline  
Note:            Centre of Northbound lane

0	-	255	Asph
255	-	610	Br Gr(y) F Sa some Si – Si(y), moist
610	-	910	Br F Sa some Gr some Si – Si(y), moist
910	-	920	Blk Tps, moist
920	-	1.52	Cl, moist
1.52	-	3.05	Si, wet

DISTRICT 61  
CONT. No. 2002-6009  
WP No. 452-00-00



HIGHWAY 72/FROG RAPIDS  
BRIDGE REHABILITATION

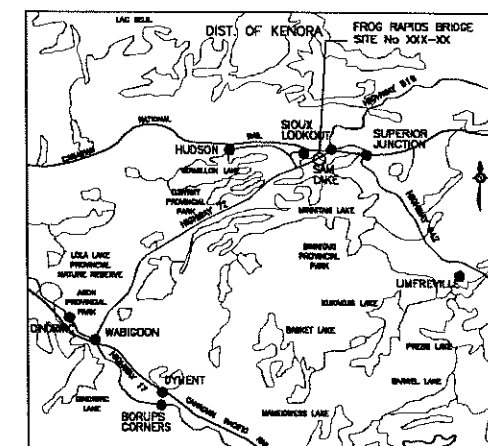
SHEET

40

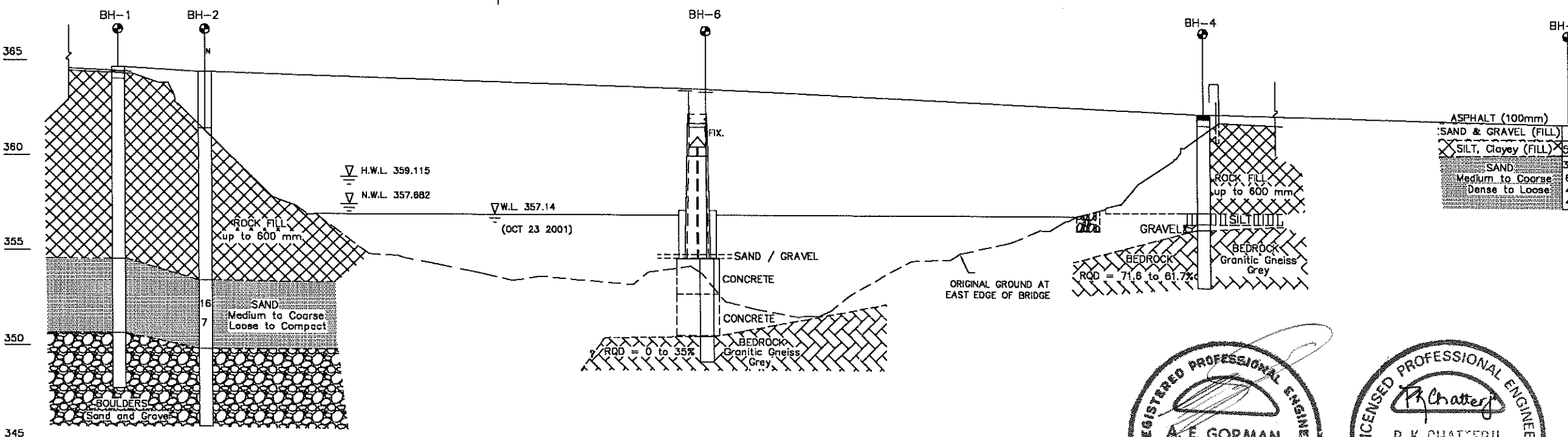
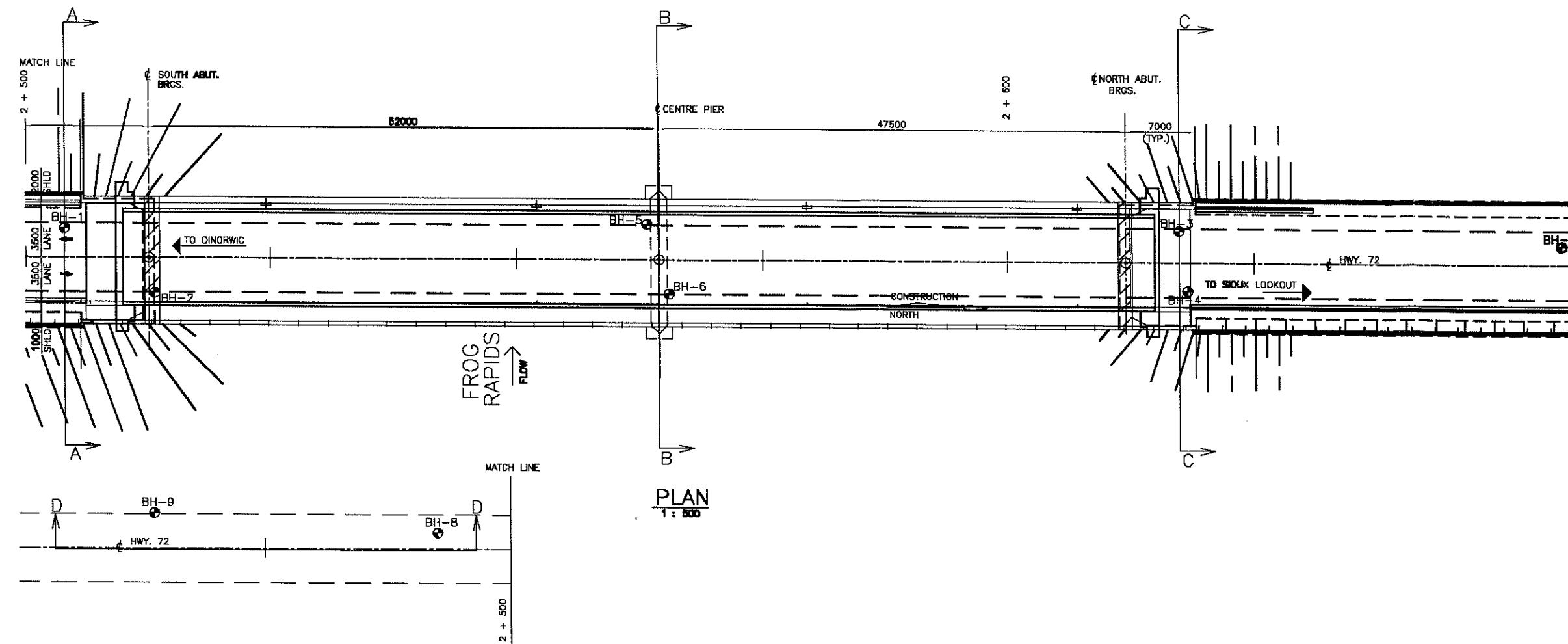
BOREHOLE LOCATION, SOIL STRATA

MRC McCORMICK RANKIN  
CORPORATION

Thurber Engineering Limited



KEY PLAN



LEGEND				
●	Bore Hole			
N	Blows/0.3m (Std Pen Test, 475J/blow)			
No	ELEVATION	NORTHING	EASTING	
1	364.83800	5548539.839	380120.974	
2	364.61600	5548545.483	380130.703	
3	362.57200	5548643.066	380168.112	
4	362.52800	5548641.293	380174.001	
5	363.69000	5548594.093	380145.174	
6	363.64600	5548593.166	380152.512	
7	361.98700	5548678.012	380185.668	
8	365.21900	5548529.005	380117.415	
9	366.38300	5548503.594	380103.579	

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



SOIL PROFILE


V. Scale - 1:250

H. Scale - 1:500

GEODETIC DATUM

REVISIONS	DESCRIPTION
DESIGN AEG CHK	CODE OHOB 1991 LOAD OHOB DATE MAY/2002
DRAWN WM CHK PKC	SITE 415-5 STRUCT SCHEME DWG 3


DISTRICT 61  
CONT. No. 2002-6009  
WP No. 452-00-00




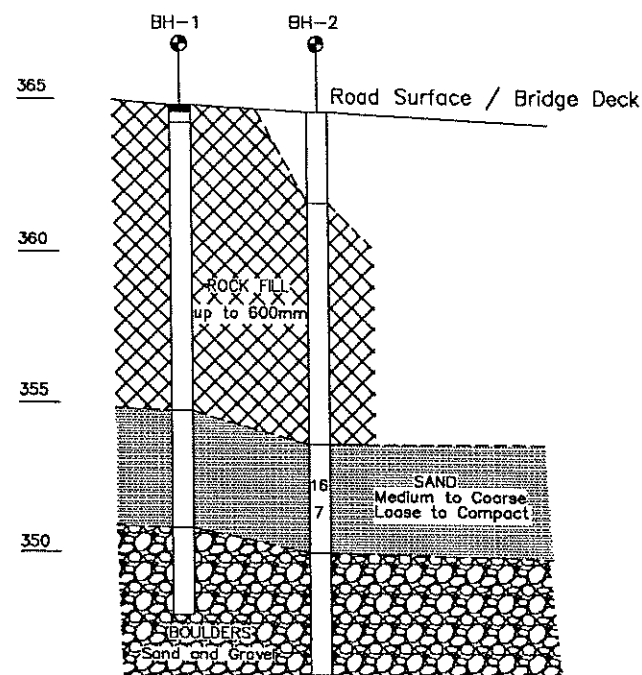
HIGHWAY 72/FROG RAPIDS  
BRIDGE REHABILITATION

SHEET  
41

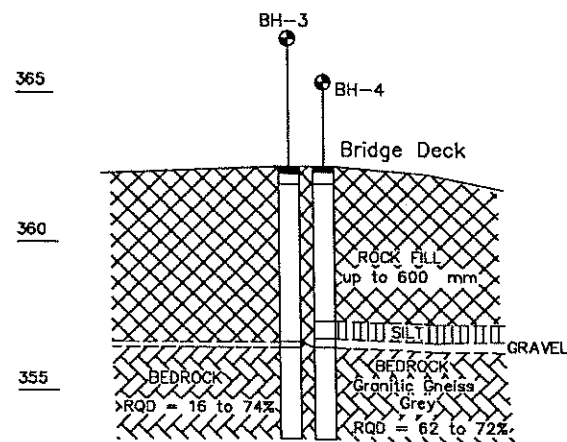
BOREHOLE LOCATION, SOIL STRATA


McCORMICK RANKIN  
CORPORATION

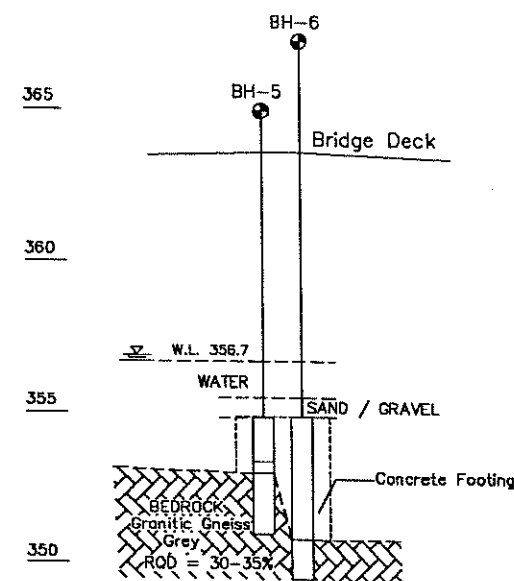

Thurber Engineering Limite



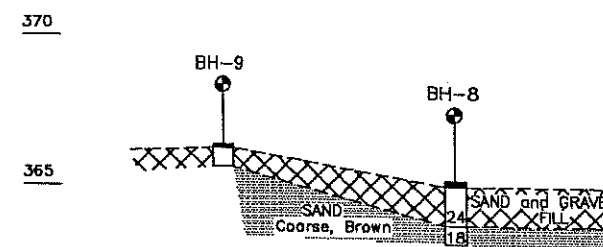
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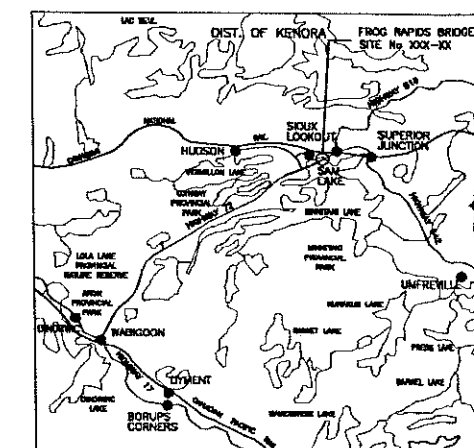
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SECTION: B-B



SECTION: D-D

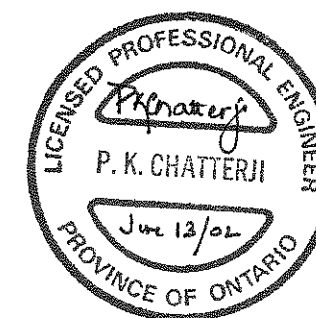
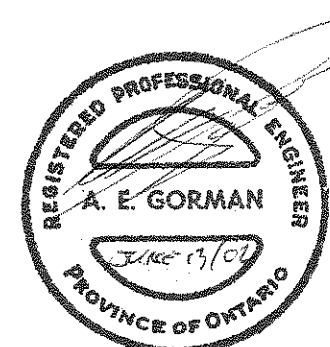


KEY PLAN

LEGEND			
● Bore Hole			
N Blows/0.3m (Std Pen Test, 475J/blow)			
No	ELEVATION	NORTHING	EASTING
1	364.83800	5548539.839	380120.974
2	364.61600	5548545.483	380130.703
3	362.57200	5548643.066	380168.112
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6	363.64600	5548593.166	380152.512
7	361.98700	5548678.012	380185.668
8	365.21900	5548529.785	380117.415
9	366.38300	5548503.594	380103.579

# SOIL CROSS-SECTION

V. Scale -- 1:250  
 H. Scale -- 1:500



METRIC  
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 AND/OR MILLIMETRES  
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DESIGN	AEG/CHK	CODE DHOB 1991 LOAD DHOB DATE MAY/02
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