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Golder Associates

REPORT
ON

FOUNDATION INVESTIGATION AND DESIGN FOR TEMPORARY BRIDGE OVER PORCUPINE RIVER W.P. 313-85-00, SITE 39E-083 HIGHWAY 101 MTO DISTRICT 53, NEW LISKEARD, ONTARIO

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PART A

FIELD INVESTIGATION
TEMPORARY BRIDGE
OVER PORCUPINE RIVER
W.P. 313-85-00, SITE 39E-083
HIGHWAY 101
MTO DISTRICT 53, NEW LISKEARD, ONTARIO

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the site of the proposed temporary detour structure ("Bailey" type bridge) for the Highway 101 crossing of the Porcupine River. The temporary bridge is required for traffic to cross the Porcupine River during the construction for the replacement of the existing Highway 101 bridge.

The subsurface information obtained during our investigation carried out for the replacement of the existing Porcupine River Bridge in 1997 and contained in the report titled "Foundation Investigation, Preliminary Design for Replacement of Porcupine River Bridge, W.P. 313-85-00, Site 39E-083, Highway 101, MTO District 53, New Liskeard, Ontario", March 1998, was compiled and used in this report. It is understood that the proposed detour Bailey bridge, to be constructed on the north side of the existing bridge, is a single span structure, wide enough to provide two way traffic.

The purpose of the investigation was to determine the subsurface conditions at the site of the proposed detour bridge by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the foundation aspects of the design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect the design of the proposed detour bridge and approach embankments.

The terms of reference and the scope of work are outlined in our letter 971-1191, dated August 24, 1999.

2.0 SITE DESCRIPTION

The site is located to the north of the existing bridge carrying Highway 101 over the Porcupine River in the Township of Matheson, District of Cochrane, east of Timmins (MTO District 53, New Liskeard). The width of the Porcupine River varies from about 20 m at the existing bridge crossing to about 10 m on the north side of the bridge. The river valley is relatively steep with several bedrock outcrops visible on both sides of the river. The water level in Porcupine River was measured at Elevation 274.6 m in December 1997.

The existing Porcupine River bridge is a four span reinforced concrete girder structure about 60 m in length, built in 1939. A retaining wall has been constructed to support the embankment fill on the north side of the east abutment. Gabion baskets have been placed at the ends of the bridge deck in all four quadrants. The current road grade / bridge deck is at about Elevation 287 m, some 12 m above the water level in the river. The existing approach embankments are 3 m to 4 m in height.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on October 20 and 21, 1999. During this time, a total of eight boreholes were put down at the site; four on each side of the river. Boreholes 99-1 through 99-4 were drilled on the west side of the river, while Boreholes 99-5 through 99-8 were put down on the east side of the river. The investigation was carried out using portable electric powered drilling equipment, supplied and operated by Colbar Resources of Sudbury.

In the boreholes, soil samples were obtained at regular intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with Standard Penetration Test (SPT) procedures. Groundwater conditions in the open boreholes were observed throughout the drilling operations and on completion of drilling.

The field work was supervised on a full-time basis by a member of our technical staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in containers and transported back to our laboratory for further examination. Index and classification tests were carried out on selected samples.

The as-drilled borehole locations were located by our field personnel relative to existing site features. The borehole elevations were surveyed by our personnel relative to the temporary benchmark provided by McCormick Rankin (Survey Point 2805 located on the north-west corner of the central pier footing - Elevation 275.274 m). It is understood that this elevation is referenced to Geodetic Datum. The locations of the boreholes are shown on Drawing 0117700, attached.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geological information, the site is located in the physiographic region known as the Canadian Precambrian Shield. The shield terrain comprises large expanses of intrusive rocks such as gneisses and gneissic or massive granite rocks. The rocks are geologically complex with considerable folding, intrusive activity, regional metamorphism and faulting. Pleistocene lacustrine / marine deposits have been laid down in depressions and are associated with the Glacial Lake Barlow-Ojibway. The local physiography is characterized by overburden consisting mainly of glaciolacustrine and glaciomarine deepwater silty clay deposits and an irregular, variable bedrock surface with frequent rock outcrops. Recent natural deposits of sand, silt and gravel can be found within the flood plains of the existing rivers.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected samples, are given on the attached Record of Borehole sheets, following the text of this report. For reference, Record of Borehole sheets for boreholes put down during the previous 1997 investigation carried out for the main structure replacement have been attached in Appendix A, following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered in the boreholes put down during the current investigation generally consist of surficial layers of topsoil underlain by a glaciolacustrine deposit of silty clay. Fill material underlies the topsoil at some borehole locations. Thin layers of sand and gravel and sandy silt, were encountered beneath the silty clay at two of the borehole locations. All of the boreholes put down during the current investigation encountered refusal to further penetration within the granular deposits or at the base of the silty clay deposit. Based on the results of the previous site investigation, the refusal to further penetration of the drill equipment is considered to be on andesite bedrock or on cobbles and / or boulders. The depth to refusal in the boreholes varied from approximately 1.0 m to 2.3 m below existing ground surface on the west side of the river and from 1.1 m to 3.8 m below ground surface on the east side of the river.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

All boreholes encountered topsoil at the ground surface. The thickness of the topsoil layer varied from approximately 90 mm to 150 mm at the borehole locations.

4.2.2 Fill Materials

Boreholes 99-1, 99-2, 99-4 and 99-5 encountered fill materials beneath the topsoil layer. The fill materials typically consist of brown clayey silt to silty clay containing trace to some sand, trace gravel and organics. The fill material is likely comprised of the native silty clay deposit which has been reworked during construction of the watermain located in the vicinity of Borehole 99-2 or the embankment fill (Boreholes 99-1, 99-4 and 99-5). Based on visual examination of the soil samples and measured Standard Penetration Test (SPT) 'N' values, which varied between 9 blows and 10 blows per 0.3 m penetration, the clayey silt / silty clay fill materials is typically stiff. The thickness of the fill materials, where encountered, ranged from approximately 0.3 m at Borehole 99-1 to 1.5 m at Borehole 99-2. The fill extends to the bedrock surface at the locations of Boreholes 99-4 and 99-5, at depths of 1.0 m to 1.2 m below ground surface. The measured water content on selected samples of the fill were 12 percent to 28 percent.

Boreholes put down through the existing roadway embankment during the previous investigation at the site encountered fill materials consisting of gravelly sand to sand and gravel, silty clay and / or rockfill beneath the pavement structure. The base of the embankment fill, where fully penetrated during the previous investigation was between 3.0 m and 4.1 m below road grade.

4.2.3 Silty Clay

Underlying the topsoil and fill materials in Boreholes 99-1, 99-3, and 99-6 through 99-8 is a lacustrine deposit of silty clay containing variable, but generally minor, quantities of sand, gravel and organic matter in the upper portion of the deposit. The thickness of the silty clay ranges from approximately 0.9 m to 3.7 m in these boreholes. The silty clay deposit is firm to very stiff with SPT 'N' values ranging from 7 blows to 17 blows per 0.3 m of penetration. However, visual examination of the samples collected indicates that this deposit has a variable consistency with the upper portions of this deposit typically being softer.

At Borehole 99-8, the silty clay deposit was distinctly irregularly layered, varying in color from light to dark brown. This irregular layering was also noted within the silty clay deposit encountered in boreholes put down during the previous investigation at the site.

Atterberg limits tests carried out on samples of this deposit indicate liquid limits ranging from about 34 percent to 56 percent and plasticity indices ranging from 15 percent to 33 percent. Measured water contents range from about 12 percent to 37 percent. A grain size distribution for one sample of the silty clay is shown on Figure 1.

Boreholes 99-3 and 99-6 through 99-8 encountered refusal to further auger penetration at the base of the silty clay deposit at depths ranging from approximately 1.1 m to 3.8 m below ground surface.

4.2.4 Granular Deposits

Approximately 0.1 m of sandy silt and 0.15 m of sand and gravel were encountered beneath the silty clay deposit at the locations of Boreholes 99-1 and 99-2, respectively, located on the west side of the river.

Granular deposits, ranging in thickness from approximately 0.9 m to 1.0 m, were also encountered beneath the silty clay deposit in Boreholes 97-1 and 97-3 put down during the previous investigation at the site. The composition of the granular deposit at these locations was highly variable ranging from sand and gravel with some silt and trace clay to boulders and cobbles with some silt, sand and gravel. This granular deposit extends to the bedrock surface at about Elevation 278.9 m in Borehole 97-1 and at about Elevation 274.3 m in Borehole 97-3.

4.2.5 Bedrock

All boreholes put down during the current investigation encountered effective refusal to auger penetration; no rock coring was carried out at the site during this investigation. Refusal was encountered between Elevation 279.9 m and Elevation 281.2 m in Boreholes 99-4 and 99-3 located at the west side of the river and between Elevation 278.5 m and Elevation 282.2 m in Boreholes 99-7 and 99-5, respectively, at the east side of the river. The depth to refusal in the boreholes varied from approximately 1.0 m to 2.3 m below existing ground surface on the west side of the river and from 1.1 m to 3.8 m depth on the east side of the river.

During the previous investigation, bedrock was encountered at variable elevations at / near the existing bridge site. In the boreholes put down through the approach embankments on the west and east sides of the existing bridge / river valley, the surface of the bedrock varied between about Elevation 278.9 m and 282.2 m. The bedrock surface generally slopes down toward the river channel and there are several bedrock outcrops exposed on the river valley slopes. Based on rock core samples collected from Boreholes 97-1, 97-3, 97-4 and 97-6, bedrock at the site consists of brownish-grey to grey, medium grained andesite with trace chlorite and talc. Frequent joints with iron staining were noted within the core samples retrieved.

4.3 Groundwater Conditions

All boreholes put down during the current investigation were dry during and following completion of drilling within the overburden. Groundwater was encountered within the sand and gravel deposit at Elevation 279.1 m during drilling of Borehole 97-1.

Based on observations made during the current and previous investigation, it is considered that the groundwater table is controlled by the granular deposits immediately overlying the bedrock, by the fractured rock and by the river water level. It should be noted that the groundwater level is subject to seasonal and river water level fluctuations.

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PART B

FOUNDATION DESIGN
TEMPORARY BRIDGE
OVER PORCUPINE RIVER
W.P. 313-85-00, SITE 39E-083
HIGHWAY 101
MTO DISTRICT 53, NEW LISKEARD, ONTARIO

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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the foundation aspects of design and construction of the proposed reinforced-soil abutments for the temporary detour "Bailey" bridge over the Porcupine River based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

It is understood that the proposed temporary detour "Bailey" bridge over the Porcupine River is to be constructed to the north of the existing structure to carry Highway 101 traffic over the Porcupine River during the construction of the replacement bridge at this site, designated as Site 39E-083. It is understood that the temporary structure will be a 61 m long, single span and wide enough to provide two way traffic. It is understood that consideration is being given to support of the bridge abutments on a mechanically stabilized / reinforced-soil mass wall system. The proposed reinforced-soil retaining walls will be located immediately to the north of the existing Highway 101 (approximately 13.8 m distance center-to-center).

The proposed horizontal alignment for the detour and sketches showing vertical alignment of the bridge abutments were provided to us by McCormick Rankin in their facsimile transmission dated August 19, 1999. Preliminary sketches of the reinforced-soil mass and abutment configurations indicate the following:

- The locations of the west and east abutments at Stations 12+380.2 and 12+441.2, respectively;
- The grade of the bridge at Elevation 287.3 m at the west abutment and at Elevation 287.7 m at the east abutment;

- The top and base of the abutment footings at Elevations 286.3 m and 285.6 m at the west abutment and Elevations 286.7 m and 286.0 m at the east abutment;
- The height of the wall / reinforced-soil mass will be as much as 4 m and 5.5 m at the west and east abutments, respectively.

5.2 Reinforced-Soil Bridge Abutments

5.2.1 General

It is understood that consideration is being given to the use of a reinforced-soil mass for the support of the bridge abutments. A reinforced-soil mass system consists of soil reinforced with metal, plastic or fabric strips or grids integrated with suitable fill which is placed and compacted in lifts. A facing material, typically consisting of pre-cast concrete blocks or panels mechanically fastened to the reinforcing strips or grids is used to form the face of the reinforced soil structure and to prevent loss of fill material. The detailed design of reinforced-soil mass systems is typically carried out by the supplier as these systems are generally proprietary.

The reinforced-soil mass system is to be constructed immediately to the north of the existing highway. Based on the current investigation, the subsoils within the site of the proposed detour structure consist of topsoil, fill and native silty clay deposit. The soils are underlain at relatively shallow depth by andesite bedrock.

The topsoil and variable fill materials are not considered suitable for the support of the reinforced-soil mass system carrying the abutments and should be completely stripped beneath the entire plan area of the footings and reinforced-soil mass under the abutments. In addition, the native silty clay deposit anticipated under the abutments is of variable thickness and loading would result in differential settlement along the width of the abutment footing. It is recommended that the silty clay deposit also be removed from under the reinforced-soil mass supporting the abutments. The approach embankments will also be constructed over variable thicknesses of compressible silty clay and / or fill materials. There will be, therefore, differential settlement between the abutments (founded on the underlying bedrock / granular deposits) and the approach embankments. In order to smooth out the transition with respect to differential settlement along the length of the reinforced-soil mass which is to be constructed perpendicular to the abutments on the north side of the detour, it is recommended that a transitional sub-excavation be carried out. The silty clay should be fully sub-excavated adjacent to the abutment; at the end of the wall, only sub-excavation of the topsoil would be required.

It may not be possible to entirely sub-excavate the native silty clay deposit beneath the proposed embankments within open cut temporary side slopes since the excavation must extend into the existing embankment. Consideration could be given to carrying out the excavation with the use of temporary shoring / road protection systems to support a vertical cut face.

5.2.2 East Abutment

The proposed road grade is about 6 m above the existing grade on the north side and about 1.5 m above the existing grade on the south side of the abutment. For the proposed abutment founding levels, the top of the reinforced-soil system at the abutments will vary between 0 m and 4.5 m above existing ground surface.

The following table summarizes the relevant boreholes at the east approach and the depth / elevation where refusal to further penetration was encountered during drilling:

Borehole Number	99-5	99-6	99-7	99-8
Depth to Auger Refusal (m)	1.1	1.3	2.0	3.8
Elevation of Auger Refusal (m)	282.2	279.8	278.5	279.7

Boreholes 99-5 and 99-6 were put down at the location of the proposed east abutment footing. Borehole 99-5 encountered a 1 m thick layer of silty clay fill extending to bedrock surface at about Elevation 282.2 m. In Borehole 99-6 about 1.2 m of native silty clay extends to the bedrock surface encountered at about 1.3 m depth, corresponding to about Elevation 279.8 m. A greater thickness of fill is anticipated within the southern portion of the footing where it extends into the existing embankment.

For design, the following founding elevations may be assumed for the reinforced-soil system placement for founding directly on the bedrock surface:

North wall	279.5 m
West wall / abutment footing	varying from 279.5 m at north wall to 282 m at south wall

The design founding level for the north wall assumes full excavation to the bedrock surface. The above founding elevations must be confirmed by site inspection during construction.

5.2.3 West Abutment

In summary, the soils encountered in Boreholes 99-1 through 99-4 put down at the location of the west abutment consist of relatively thin deposit of silty clay and fill material which are underlain by bedrock and / or granular soils. The following table summarizes the relevant boreholes at the west approach and the depth / elevation where refusal to further auger penetration was encountered during drilling:

Borehole Number	99-1	99-2	99-3	99-4
Depth to Auger Refusal (m)	2.3	1.7	1.0	1.2
Elevation of Auger Refusal (m)	281.2	280.0	281.2	279.9

The proposed road grade at the abutment is about 5 m above existing ground surface. For the proposed abutment founding levels, the top of the reinforced-soil mass at the abutment will be about 3 m to 3.5 m above existing ground surface.

For design, the following founding elevations may be assumed for the reinforced-soil system placement for founding directly on the bedrock or granular soils:

North wall	Varying from 281.5 m at west and to 280.5 m at east end
East wall / abutment footing	varying from 280.5 m at north wall to 281 m at center, to 280 m at south end
South wall	Varying from 280 m at east end to 279.5 m at west end

The design founding levels for the north and south walls assume full sub-excavation. The above founding elevations must be confirmed by site inspection during construction.

5.2.4 Design Interaction

Reinforced-soil or mechanically stabilized soil systems are proprietary. The design of these systems in terms of internal stability including type of reinforcement, length of spacing of reinforcement, type of facing units and fastenings to the facing units is carried out specifically by the supplier of the proprietary system. The design needs to take into account surcharge effects such as that caused by the abutment foundation sitting on top of the reinforced-soil system.

Therefore, the design bearing pressure for the abutment foundation placed on the reinforced-soil mass is controlled by the design and capability of the reinforced-soil mass system itself.

It will be necessary to have design interaction between the suppliers of the reinforced-soil systems and the structural engineer designing the bridge abutment foundation. The designer of the proprietary system specifies the bearing capacity that is required of the ground on which the reinforced-soil mass will sit. For the case of the system founded on bedrock or dense granular deposits, the geotechnical resistance (bearing capacity) at ULS provided by the ground would be greater than 1,000 kPa which is considered to be adequate to comply with and satisfy the proprietary system's requirements. This should be confirmed with the proprietary system designers.

5.3 Subgrade Preparation and Approach Embankments

The surficial topsoil, and variable fill materials are not considered suitable for the support of the retaining walls, reinforcing elements or approach embankment fills and should be stripped from below the proposed approach embankment areas.

The design assumptions with respect to founding elevations should be confirmed by inspection by qualified personnel in the field during construction. Where the excavation is not extended to the bedrock surface, the exposed subgrade soils should be proof-rolled prior to fill placement. Softened or loosened soils should be sub-excavated and backfilled with approved select fill.

Construction of the approach embankment above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or Select Subgrade Material (in accordance with OPSS 1010) depending on the material available to the project. The existing embankments should be benched in accordance with OPSD 208.01 to ensure that the new fill is keyed into the existing fill and to minimize differential settlement between the new and the existing embankment.

All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

5.4 Temporary Excavations

Temporary excavations required for the construction of reinforced-soil mass supporting the abutments will be up to about 5 m deep at the west abutment and will vary between about 2 m to 4 m in depth at the east abutment. These depths encompass the full reinforced mass width under the abutment footings.

Temporary excavations will extend through loose to dense granular fill, soft to firm cohesive fill materials and through firm to very stiff, native silty clay. At the location of the east abutment, groundwater seepage inflow into the excavations is expected to be minor, except during periods of sustained precipitation. Some water flow into the excavation should be anticipated for construction of the west embankment, where water bearing sand and gravel deposits may be encountered overlying fractured bedrock.

Where space permits, excavations, which will be open for a relatively short period of time, can be made in temporary unsupported cut with side slopes maintained not steeper than 1 horizontal to 1 vertical, provided that only minor groundwater inflow is encountered. Along the south walls of the reinforced-soil abutments, relatively deep excavations will be required and open cut temporary excavations would require undercutting of the existing highway embankments. To protect against instability of the embankment and / or movements of the existing bridge abutments, the excavation for the south wall in the vicinity of the existing embankments at each abutment should be carried out within a temporary support system. The support system could consist of soldier piles and lagging extending to bedrock surface. Support to the soldier pile and lagging wall system could be in the form of rakers or an anchorage system and would probably involve additional anchorage at the toe of the pile to provide lateral support.

The design of soldier pile and lagging walls, where the support to the wall is provided by anchors or rakers, should be based on a triangular earth pressure distribution using the design parameters given below. The raker or anchor loads themselves should be checked using a rectangular earth pressure distribution (apparent earth pressure) using the parameters given below. The wall and the raker / anchor support system must be designed to accommodate the loads applied from surcharge pressures from area, line or point loads as wells as the influence of sloping ground behind the system.

Unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

$$p = K \gamma H$$

where

H = is the height of the excavation at any point in metres
K = 0.3 for level ground behind excavation
= 0.5 for ground sloping at 2H:1V behind excavation
γ = soil unit weight = 20 kN/m^3

Unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows:

$$p = K \gamma H$$

where

H = is the height of the excavation at any point in metres
K = 0.3 for level ground behind excavation
γ = soil unit weight = 20 kN/m^3

For the above case, the sloping ground should be treated as a surcharge.

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. The coefficient of passive lateral earth pressure, K_p, for the socket within the bedrock may be taken as 6. The groundwater level should be assumed to be at the bedrock surface.

It should be noted that churn drilling / augering procedures will be extremely difficult in creating a pile socket within the bedrock at this site. Core drilling will likely be required to advance the hole for pile installation.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index = $(w_l - w_p)$
w_S shrinkage limit
I_L liquidity index = $(w - w_p) / I_p$
I_C consistency index = $(w_l - w) / I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = $\tan\delta$
c' effective cohesion
c_u, S_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 99-1

METRIC

PROJECT 991-1177

W.P. 313-85-00 LOCATION STA. 12+369.8, 18.5m Rt. OF CL EXISTING HWY.#101 ORIGINATED BY MSB

DIST 53 HWY 101 BOREHOLE TYPE PORTABLE POWER AUGER COMPILED BY KN

DATUM GEODETIC DATE 21.10.99 CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT / LIMITS	GR	SA	SI	CL
283.53										
0.00	Topsoil									
0.12 / 283.07	Clayey Silt, Brown (Fill)				283					
0.46	Silty Clay, trace sand, trace organics, Firm to very stiff, Brown	1	50 DO	7		w ≈ 29				
		2	50 DO	10	282	w_P ≈ 19, w ≈ 33, w_L ≈ 37	0	2	51	47
281.40		3	50 DO	20		w ≈ 30				
281.24 / 2.29	Sandy Silt, trace clay, trace gravel, Compact, Brown									
	END OF BOREHOLE. Refusal to auger penetration probably on bedrock									

Note:
Open hole dry on completion on drilling.

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 99-2

METRIC

PROJECT 991-1177
W.P. 313-85-00
LOCATION STA. 13+382.0, 23.5m Rt. OF CL EXISTING HWY.#101
ORIGINATED BY MSB
DIST 53 HWY 101
BOREHOLE TYPE PORTABLE POWER AUGER
COMPILED BY KN
DATUM GEODETIC
DATE 21.10.99
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)
281.63							
0.00	Topsoil						
0.10	Clayey Silt, trace to some sand, trace gravel, trace organics Firm to very stiff Brown Moist (Fill)					281	
		1	50 DO	9			~28
		2	50 DO	49		280	~12
280.05 1.68	Sand and Gravel, trace silt Very dense Brown Moist						
	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.						

Chart column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 99-3

METRIC

PROJECT 991-1177
W.P. 313-85-00
LOCATION STA. 12+380.8, 14.5m Rt. OF CL EXISTING HWY.#101
ORIGINATED BY MSB
DIST 53 HWY 101
BOREHOLE TYPE PORTABLE POWER AUGER
COMPILED BY KN
DATUM GEODETIC
DATE 21.10.99
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (20 40 60 80 100)	WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
282.28											
0.00 / 0.09	Topsoil										
	Silty Clay, trace sand and gravel Firm to stiff Brown						282				
			1	50 DO	14				o (~22)		
281.24 / 1.04	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.										

Legend: SHEAR STRENGTH: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 99-4

METRIC

PROJECT 991-1177			
W.P. 313-85-00	LOCATION STA. 12+380.8, 9m Rt. OF CL EXISTING HWY.#101		ORIGINATED BY MSB
DIST 53	HWY 101	BOREHOLE TYPE PORTABLE POWER AUGER	COMPILED BY KN
DATUM GEODETIC	DATE 21.10.99		CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
281.15											
0.00 / 0.09	Topsoil Silty Clay, trace sand and gravel Stiff Brown (Fill)						281				
			1	50 DO	10				○ (approx. 22)		
279.93 1.22	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.						280				

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 99-5

METRIC

PROJECT 991-1177
W.P. 313-85-00
LOCATION STA. 12+439.6, 14.5m Rt. OF CL EXISTING HWY.#101
ORIGINATED BY MSB
DIST 53 HWY 101
BOREHOLE TYPE PORTABLE POWER AUGER
COMPILED BY KN
DATUM GEODETIC
DATE 21.10.99
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
283.26											
0.00 / 0.09	Topsoil										
	Silty Clay, trace sand and gravel Stiff Brown (Fill)						283				
			1	50 DO	9				o (approx. 11)		
282.16 / 1.10	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.										

+³, ×³: Numbers refer to Sensitivity
O ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 99-6

METRIC

PROJECT 991-1177

W.P. 313-85-00 LOCATION STA. 12+439.6, 19.8m Rt. OF CL EXISTING HWY.#101 ORIGINATED BY MSB

DIST 53 HWY 101 BOREHOLE TYPE PORTABLE POWER AUGER COMPILED BY KN

DATUM GEODETIC DATE 21.10.99 CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
281.09							281				
0.00 / 0.09	Topsoil Silty Clay, some gravel, trace sand and organics Firm to stiff Brown	▨	1	50 DO	8				46		
279.78 / 1.31	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.						280				

Plot legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

$+^{3}, \times^{3}$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 99-7

METRIC

PROJECT 991-1177
W.P. 313-85-00
LOCATION STA. 12+445.6, 22.5m Rt. OF CL EXISTING HWY.#101
ORIGINATED BY MSB
DIST 53 HWY 101
BOREHOLE TYPE PORTABLE POWER AUGER
COMPILED BY KN
DATUM GEODETIC
DATE 20.10.99
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
280.55											
0.00	Topsoil										
0.12	Silty Clay, trace to some sand and gravel, trace organics Soft to firm Brown Moist						280				
			1	50 DO	9						
			2	50 DO	10		279				
278.51			3	50 DO	9/.15						
2.04	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 99-8

METRIC

PROJECT 991-1177
W.P. 313-85-00
LOCATION STA. 12+451.7, 19m Rt. OF CL EXISTING HWY.#101
ORIGINATED BY MSB
DIST 53 HWY 101
BOREHOLE TYPE PORTABLE POWER AUGER
COMPILED BY KN
DATUM GEODETIC
DATE 20.10.99
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
283.48						
0.00	Topsoil					
0.15	Silty Clay, trace sand, irregularly layered Moist Light to dark brown consistency				283	
		1	50 DO	10		
		2	50 DO	13	282	56
		3	50 DO	16		
		4	50 DO	14	281	
	-Cobbles encountered below 3m depth	5	50 DO	17	280	
279.67 3.81	END OF BOREHOLE Refusal to auger penetration probably on bedrock Note: Open hole dry on completion on drilling.					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

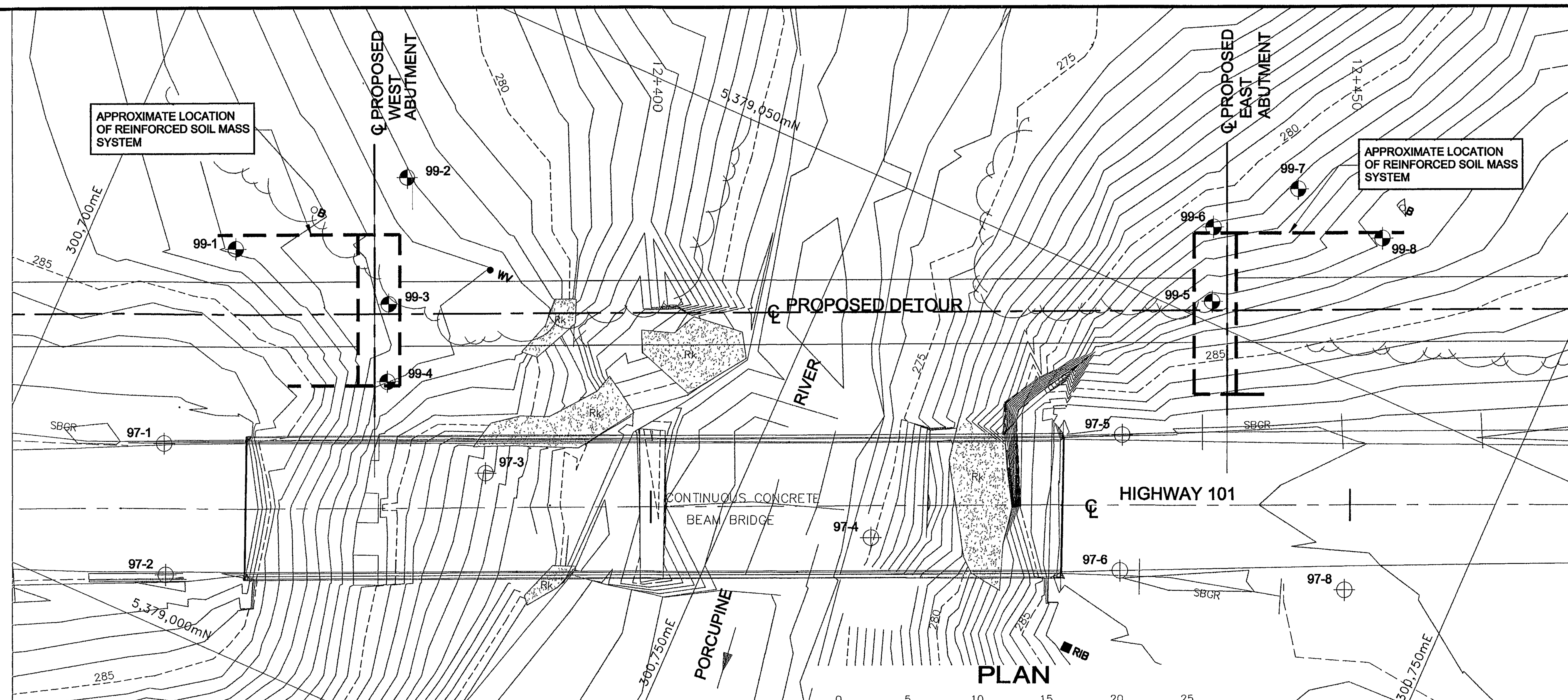
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 313-85-00

HIGHWAY No. 101 DETOUR AT PORCUPINE RIVER
BORE HOLE LOCATIONS & SOIL STRATA

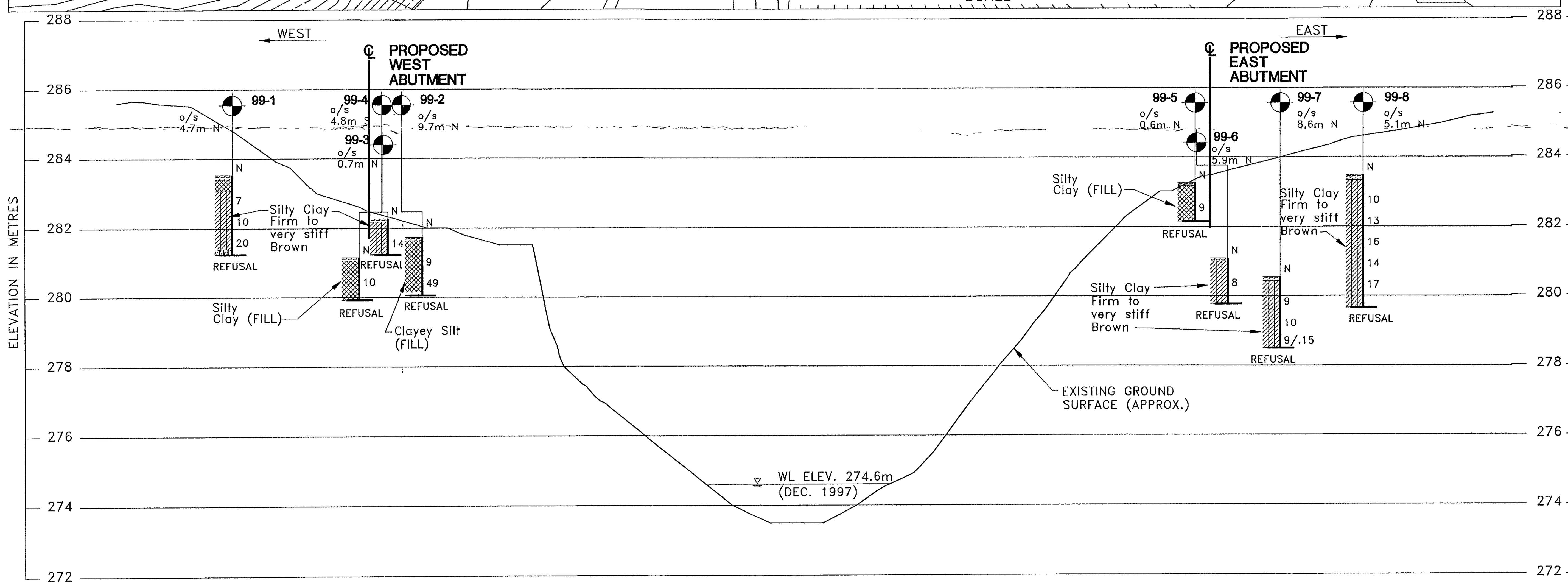
SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

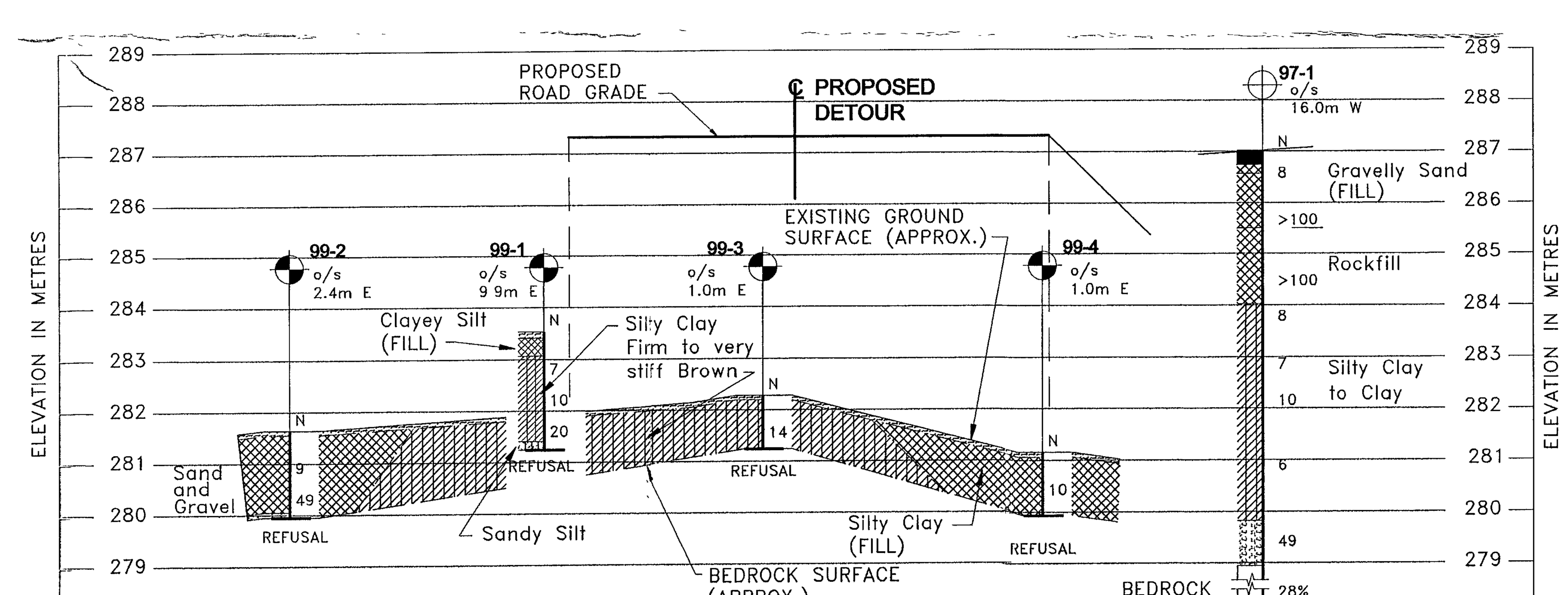
SCALE 0 5 10 15 20 25 METRES



PROFILE ALONG CENTRELINE OF DETOUR

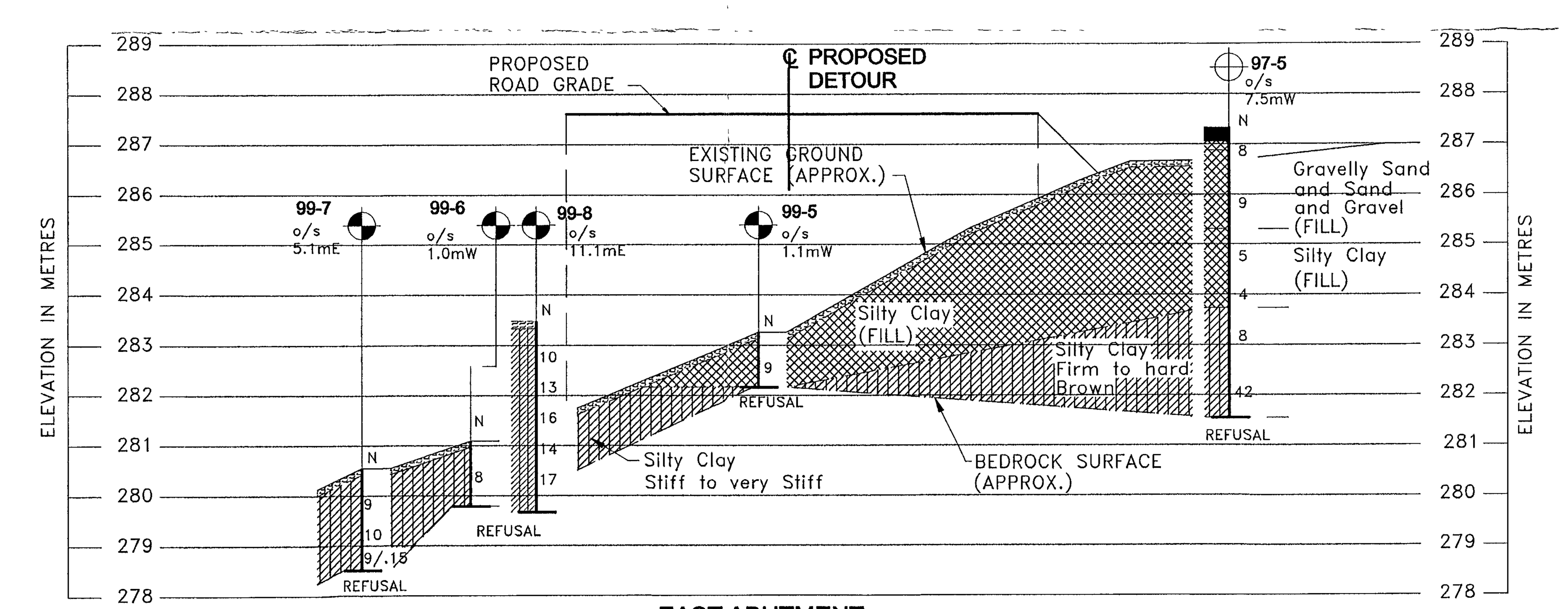
HORIZONTAL SCALE 0 5 10 15 20 25 METRES

VERTICAL SCALE 0 2 4 6 8 10 METRES



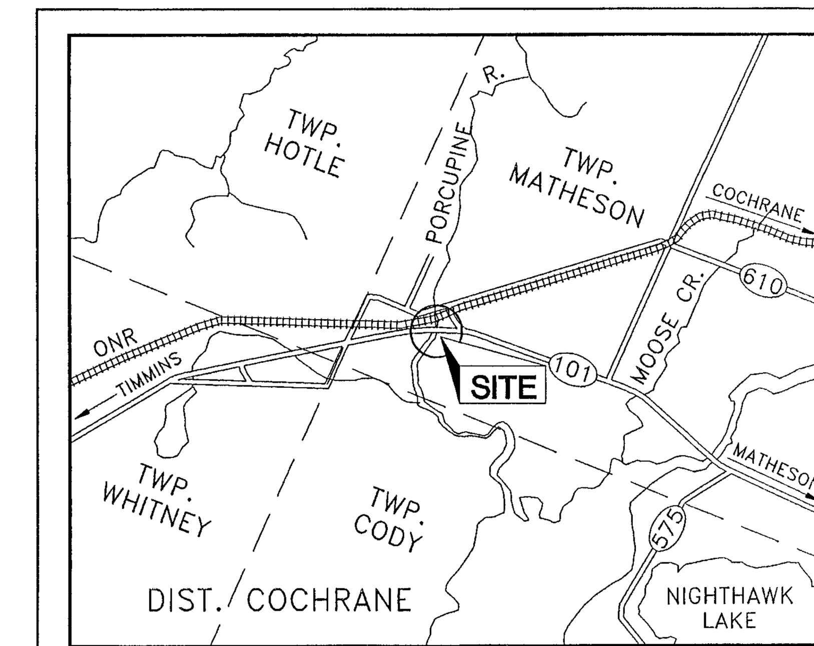
WEST ABUTMENT

SCALE 0 2 4 6 8 10 METRES



EAST ABUTMENT

SCALE 0 2 4 6 8 10 METRES



KEY PLAN

0.25 0 0.25 0.5 kilometre
SCALE, km

LEGEND

- Borehole 1997 Investigation
- Borehole 1999 investigation
- N — Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL at time of investigation 1997 08
- Rk — Bedrock Outcrop

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
97-1	287.02	5,379,012	300,711
97-2	286.95	5,379,004	300,715
97-3	276.55	5,379,019	300,733
97-4	275.21	5,379,026	300,760
97-5	287.33	5,379,040	300,774
97-6	286.96	5,379,031	300,777
97-7	287.10	5,379,007	300,696
97-8	287.24	5,379,037	300,793
99-1	283.53	5,379,027	300,710
99-2	281.63	5,379,036	300,719
99-3	282.28	5,379,028	300,722
99-4	281.15	5,379,023	300,724
99-5	283.26	5,379,051	300,776
99-6	281.09	5,379,056	300,774
99-7	280.55	5,379,061	300,778
99-8	283.48	5,379,061	300,785

NOTES

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

The foundation locations are shown for reference only and may differ from that on the structural drawings.

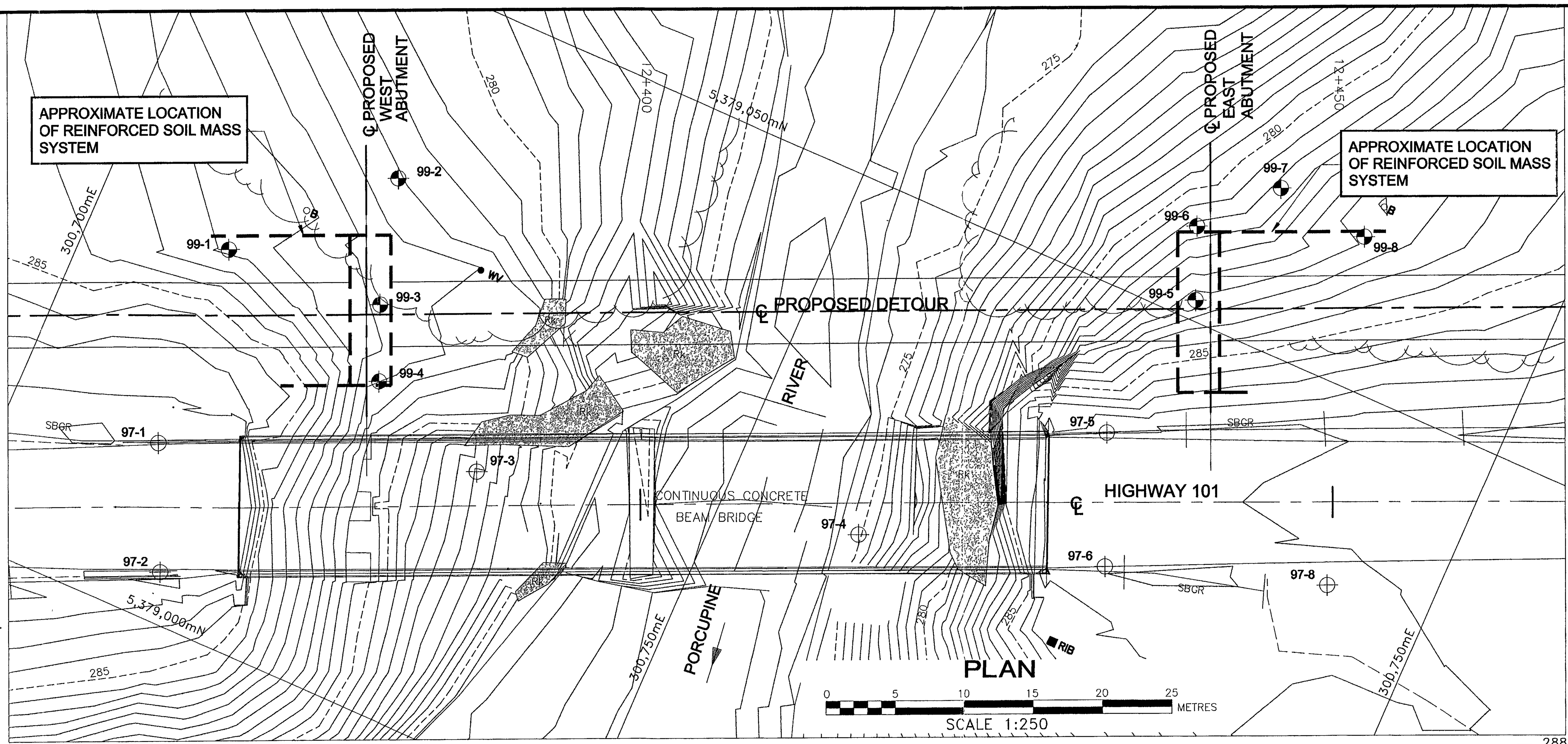
NO.	DATE	BY	REVISION

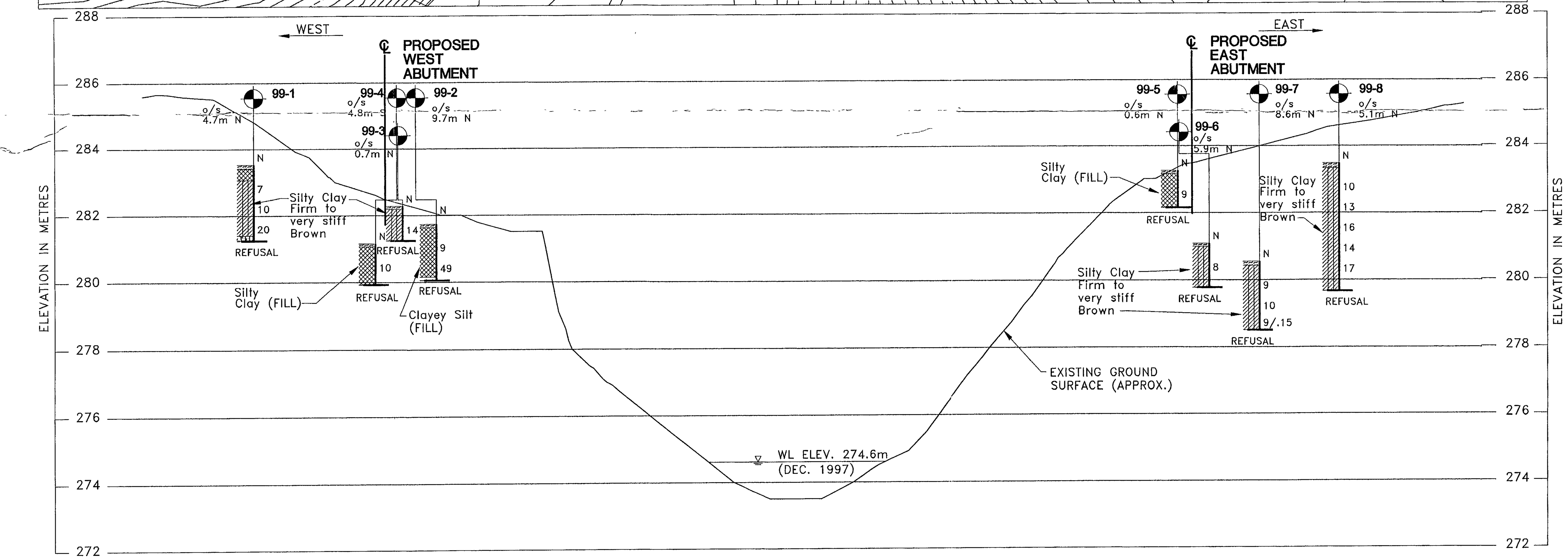
Geocres No.

HWY. No. 101		PROJECT NO.: 991-1177	53
SUBM'D. ASP	CHKD:	DATE: 1999 11 04	SITE 39E-083
DRAWN: JFC	CHKD. ASP	APPD.	DWG. 01177001

01177001.DWG

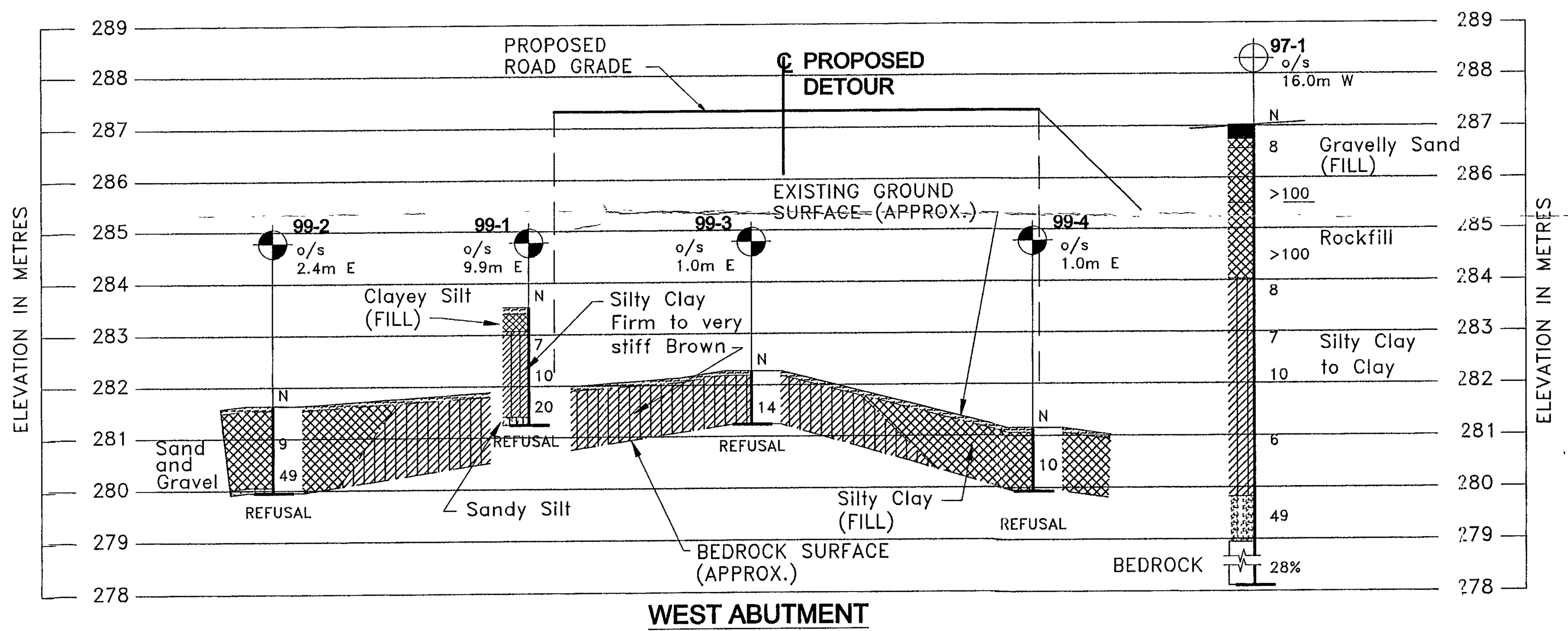
1" = 1" IMP (1:250 MS)





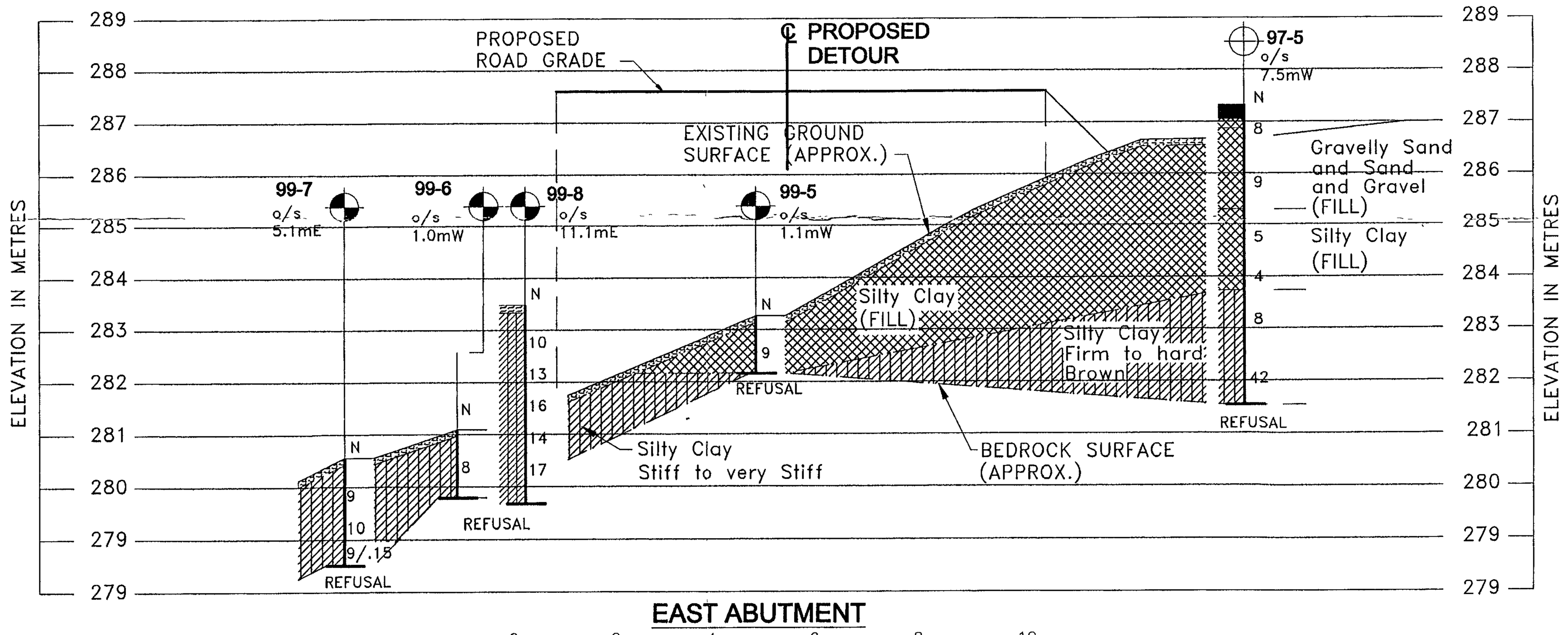
PROFILE ALONG CENTRELINE OF DETOUR

HORIZONTAL SCALE 1:250 VERTICAL SCALE 1:100



WEST ABUTMENT

SCALE 1:100



EAST ABUTMENT

SCALE 1:100

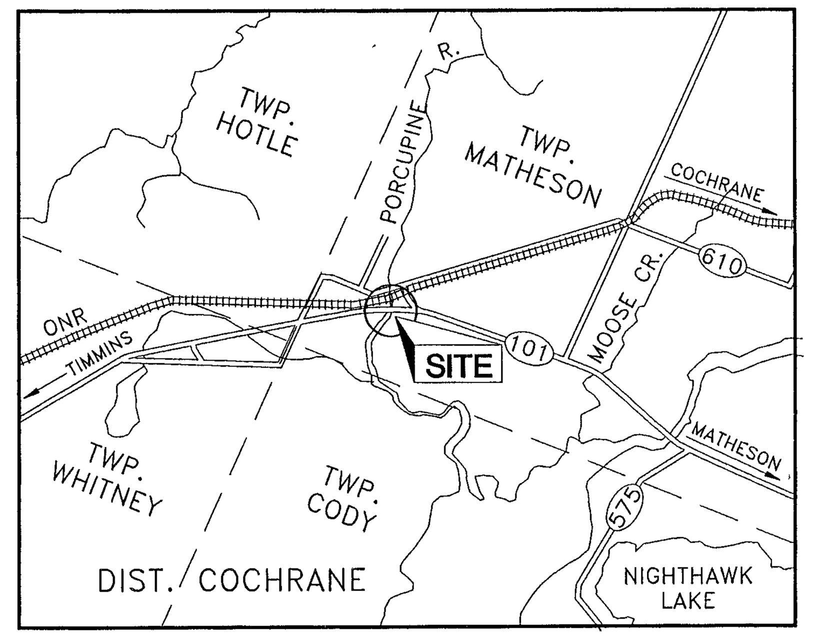
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 313-85-00

HIGHWAY No. 101 DETOUR AT PORCUPINE RIVER
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE, km

LEGEND

- Borehole 1997 Investigation
- Borehole 1999 investigation
- N — Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL at time of investigation 1997 08
- Rk — Bedrock Outcrop

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
97-1	287.02	5,379,012	300,711
97-2	286.95	5,379,004	300,715
97-3	276.55	5,379,019	300,733
97-4	275.21	5,379,026	300,760
97-5	287.33	5,379,040	300,774
97-6	286.96	5,379,031	300,777
97-7	287.10	5,379,007	300,696
97-8	287.24	5,379,037	300,793
99-1	283.53	5,379,027	300,710
99-2	281.63	5,379,036	300,719
99-3	282.28	5,379,028	300,722
99-4	281.15	5,379,023	300,724
99-5	283.26	5,379,051	300,776
99-6	281.09	5,379,056	300,774
99-7	280.55	5,379,061	300,778
99-8	283.48	5,379,061	300,785

NOTES

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

The foundation locations are shown for reference only and may differ from that on the structural drawings.

NO.	DATE	BY	REVISION

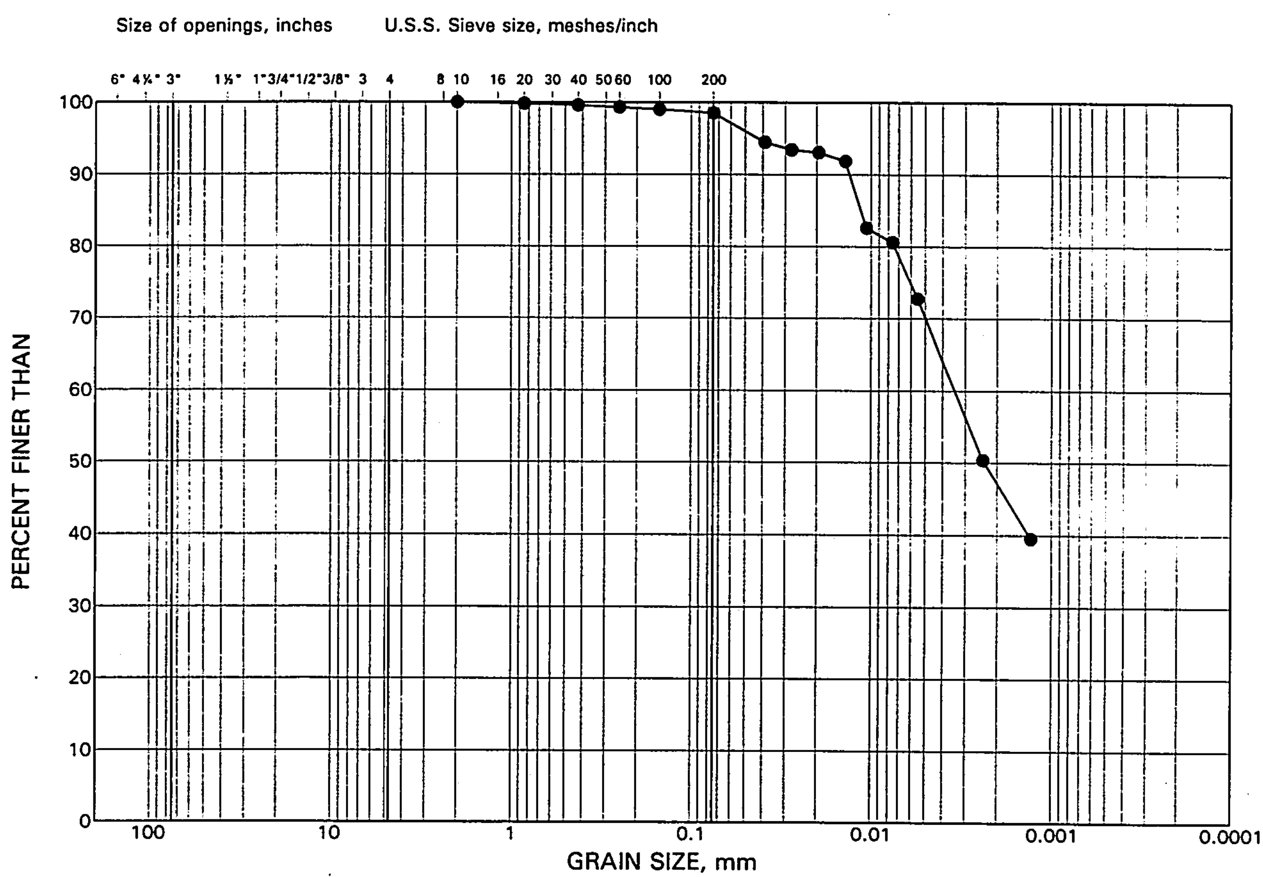
Geocres No.

HWY. No. 101		PROJECT NO.: 991-1177	53
SUBM'D. ASP	CHKD:	DATE: 1999 11 04	SITE 39E-083
DRAWN: JFC	CHKD. ASP	APPD.	DWG. 01177001

01177001.DWG
1" = 1" IMP

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 1



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	99-1	2	1.8

APPENDIX A

RECORD OF BOREHOLE SHEETS
GOLDER ASSOCIATES REPORT 971-1191

MPRB0001.BHS

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379012.10; E 300711.02

RECORD OF BOREHOLE 97-1

BORING DATE: DEC.11/97

SHEET 1 OF 2

DATUM: GEODETIC

PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
ROAD SURFACE		287.02 / 0.00				
ASPHALT		286.74 / 0.28				
Granular A (Road Base)		286.56 / 0.46				
Gravelly Sand, trace silt Loose Brown Moist (FILL)		285.50 / 1.52	1	50 DO	8	
Rockfill (inferred from resistance to augering - no sample recovery)		284.02 / 3.00	2	50 DO	> 100	
			3	50 DO	> 100	
Silty Clay to Clay, irregularly layered, trace sand Firm to stiff Brown Moist		279.82 / 7.20	4	50 DO	8	
			5	50 DO	7	MH
			6	50 DO	10	
			7	50 DO	6	
Sand and Gravel, some silt, trace clay Dense Brown Wet		278.94 / 8.08	8	50 DO	49	
REFUSAL TO AUGER PENETRATION BOREHOLE CONTINUED. FOR ROCK CORING DETAILS REFER TO RECORD OF DRILLHOLE, SEE SHEET 2.						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp, W, Wl — 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION

NOTE:
Water level at 7.9m depth during drilling in overburden.

CONTINUED ON NEXT PAGE

DATA INPUT: PS JAN.21/98

DEPTH SCALE 1 to 50

LOGGED: MB

CHECKED: AMP

Golder Associates

SOILM6

MPRB0001.BHR

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379012.10; E 300711.02

RECORD OF DRILLHOLE: 97-1

DRILLING DATE: DEC.11/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		CONTINUED FROM PREVIOUS PAGE														
7																
		For overburden stratigraphy, refer to Record of Borehole 97-1, sheet 1.														
8				278.94 8.08												
	NQ CORING	Brownish grey, medium grained ANDESITE, trace chlorite and talc on joints, some iron staining. (BEDROCK)			1								J,PL,R (x5) J,ST,R J,S,R Broken core			
9													J,I,R (x4) J,U,R J,PL,R (x3)			
10					2								Broken core J,PL,R (x5)			
11					3								J,PL,R (x5) J,PL,SM J,R,E J,U,R			
				275.20 11.82												
12		END OF DRILLHOLE														
13																
14																
15																
16																
17																

DATA INPUT: PS JAN.21/98

ROCKMV5

DEPTH SCALE:

1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

MPRB0002.BHS

W.P. 313-85-00

DIST. 53, SITE: 39E-083,HWY 101

LOCATION: N 5379003.63; E 300714.98

RECORD OF BOREHOLE 97-2

BORING DATE: DEC.9/97

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT (Wp – W – Wl)	PIEZOMETER OR STANDPIPE INSTALLATION
ROAD SURFACE	286.95 / 0.00					PROTECTIVE PIPE
ASPHALT	286.65 / 0.30					
Granular A (Road Base)	0.45					
Gravelly Sand to Sand and Gravel with possible rockfill/cobbles inferred from resistance to augering Very loose to dense Brown Moist (FILL)		1	50 DO	14	W ≈ 10	GROUT
		2	50 DO	36	W < 10	
		3	50 DO	7	W ≈ 10	
		4	50 DO	3		
	282.84 / 4.11	5	50 DO	> 100		BENTONITE SEAL
Silty Clay to Clay, irregularly layered, trace sand, trace gravel at depth Stiff to very stiff Brown Moist		6	50 DO	9	Wp ≈ 27, W ≈ 30, Wl ≈ 46	GRAVEL
		7	50 DO	22	W ≈ 26	
END OF BOREHOLE REFUSAL TO AUGER PENETRATION (PROBABLE BEDROCK).	280.40 / 6.55					

Other column headings: DEPTH SCALE METRES (0–10); STRATA PLOT; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (10 20 30 40); ADDITIONAL LAB. TESTING.

NOTE:
Borehole dry during drilling and on completion of drilling.

Piezometer dry on Dec.20,1997.

DATA INPUT: PS JAN.21/98

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

SOILM6

MPRB0003.BHR

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379019.42; E 300732.99

RECORD OF DRILLHOLE: 97-3

DRILLING DATE: DEC.10&11/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
0		GROUND SURFACE	276.55		
		TOPSOIL	0.00		
		Silty Clay	0.08		
1			275.33 / 1.22		
		Boulders and cobbles with some silt, sand and gravel.			
2			274.33 / 2.22		
	BQ CORING	Medium grained, grey ANDESITE, heavy iron staining, trace chlorite on joints. (BEDROCK)		1	Broken core; J,PL,SM
3				2	Broken core 0.6m; J,PL,SM; J,PL,SM; J,PL,SM; J,PL,R; J,PL,R(x2)
4				3	J,PL,SM (x2); J,PL,R; J,PL,SM; J,PL,R; J,U,R
				4	J,PL,SM; J,U,SM; J,U,SM; J,U,SM
5			271.28 / 5.27		J,PL,SM; J,PL,R
		END OF DRILLHOLE			
6					
7					
8					
9					
10					

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min.); FLUSH; COLOUR / % RETURN; RECOVERY: TOTAL CORE %, SOLID CORE %; R.Q.D. %; FRACT. INDEX PER 0.25; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY k, cm/sec; DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

DATA INPUT: PS JAN 21/98

ROCKMV5

DEPTH SCALE: 1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

MPRB0004.BHR

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379026.26; E 300760.03

RECORD OF DRILLHOLE: 97-4

DRILLING DATE: DEC.10/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY: TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS / TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
0		GROUND SURFACE		275.21										
	BQ CORING	Medium grained, grey ANDESITE, trace chlorite and talc on joints. Minor amounts of mica and quartzite sections. (BEDROCK)		0.00	1						Broken core to 0.9m depth			
1											J,ST,R; J,PL,SM; J,PL,SM; J,PL,SM (x2)			
											Broken core			
2					2						J,PL,R; J,PL,SM; J,PL,R; J,I,R; Broken core			
											J,PL,R J,PL,SM (x2); J,PL,R			
3					3						J,ST,R J,PL,R; J,PL,R; J,PL,R			
				271.61							J,PL,SM			
		END OF DRILLHOLE		3.60										
4														
5														
6														
7														
8														
9														
10														

DATA INPUT: PS JAN.21/98

ROCKMV5

DEPTH SCALE: 1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

W.P. 313-85-00
DIST. 39E-083,HWY 101
LOCATION: N 5379040.23; E 300773.68

RECORD OF BOREHOLE 97-5

BORING DATE: DEC.9/97

SHEET 1 OF 1
DATUM: GEODETIC
PROJECT: 971-1191

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp W Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	BOMBARDIER CME-55	ROAD SURFACE	287.33							
		ASPHALT	0.00 / 287.05							
		Granular A (Road Base)	0.28 / 286.87							
1		Gravelly Sand to Sand and Gravel, trace silt, trace clay Loose Brown Moist (FILL)	0.46	1	50 DO	8				
				2	50 DO	9				
2		Silty Clay, trace to some sand, trace gravel, trace organics Soft to firm Moist (FILL)	285.30 / 2.03	3	50 DO	5				
3				4	50 DO	4			MH	
4		Silty Clay, irregularly layered, trace sand, trace organics Firm to hard Brown Moist	283.73 / 3.60	5	50 DO	8				
5				6	50 DO	42				
6		END OF BOREHOLE REFUSAL TO AUGER PENETRATION (PROBABLE BEDROCK).	281.54 / 5.79							Note: Open borehole dry on completion of drilling.
7										
8										
9										
10										

DATA INPUT: PS JAN.21/98

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

W.P.	313-85-00
DIST.	39E-083,HWY 101
LOCATION:	N 5379031.37; E 300777.32

RECORD OF BOREHOLE 97-6

BORING DATE: DEC.10/97

DATUM: GEODETIC

PROJECT: 971-1191

Boring method: BOMBARDIER CME-55

Columns: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)); SAMPLES (NUMBER, TYPE, BLOWS/0.3m); DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp —O W— Wl; 10 20 30 40); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
ROAD SURFACE	286.96 / 0.00			
ASPHALT	286.68 / 0.28			
Granular A (Road Base)	286.50 / 0.46			
Gravelly Sand to Sand and Gravel, trace silt, occ. cobble, silty clay lenses Compact to very dense Moist Brown (FILL)		1	50 DO	11
		2	50 DO	10
		3	50 DO	14
		4	50 DO	67
Silty Clay, irregularly layered, trace sand, trace gravel, trace organics Stiff Brown Moist	283.26 / 3.70	5	50 DO	14
Sand, some gravel Very dense Brown Moist	282.39 / 4.57	6	50 DO	55/.15
REFUSAL TO AUGER PENETRATION BOREHOLE CONTINUED. FOR ROCK CORING DETAILS REFER TO RECORD OF DRILLHOLE, SHEET 2.	4.72			

NOTE:
Open borehole dry during drilling through overburden.

CONTINUED ON NEXT PAGE

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

DATA INPUT: PS JAN.21/98

SOILM6

MPRB0006.BHR

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379031.37; E 300777.32

RECORD OF DRILLHOLE: 97-6

DRILLING DATE: DEC.10/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		CONTINUED FROM PREVIOUS PAGE														
4		For overburden stratigraphy, refer to Record of Borehole 97-6, sheet 1.														
	NQ CORING	Brownish grey, medium grained ANDESITE, trace chlorite and talc on joints. (BEDROCK)		282.24 / 4.72	1								Broken core 0.6m			
5													J,PL,R (x3)			
													J,PL,R			
6													J,U,R			
													J,PL,R			
					2								J,PL,SM (x2)			
													J,ST,R			
													J,PL,R			
													Broken core			
7													J,PL,SM			
													J,PL,SM			
													J,ST,SM			
													J,U,SM			
													J,PL,SM			
													J,I,SM			
				279.04 / 7.92									J,PL,R (x4)			
8		END OF DRILLHOLE														
9																
10																
11																
12																
13																
14																

DATA INPUT: PS JAN.21/98

ROCKMV5

DEPTH SCALE:

1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

W.P. 313-85-00

DIST. 39E-083,HWY 101

LOCATION: N 5379006.56; E 300696.15

RECORD OF BOREHOLE 97-7

BORING DATE: DEC.11/97

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O^W— Wl	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		ROAD SURFACE		287.10							
0	BOMBARDIER CME-55	Granular A		0.00 / 286.82							
		Granular B		0.28 / 286.50							
		Rockfill (inferred from resistance to augering)		0.60 / 286.03							
1		REFUSAL TO AUGER PENETRATION END OF BOREHOLE		1.07							NOTE: Open borehole dry on completion of drilling operations.
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

MPRB0008.BHS

W.P. 313-85-00

DIST. 39E-083,HWY 101

LOCATION: N 5379036.55; E 300792.50

RECORD OF BOREHOLE 97-8

BORING DATE: DEC.10/97

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	BOMBARDIER CME-55	ROAD SURFACE		287.24							
		Granular A		0.00							
		Granular B		0.15 286.78							
1		Sand and Gravel Dense (FILL)		0.46	1	50 DO	33		O		
2		Rockfill (inferred from resistance to augering)		285.56 1.68	2	50 DO	> 100				
		REFUSAL TO AUGER PENETRATION END OF BOREHOLE		284.73 2.51							NOTE: Open borehole dry on completion of drilling operations.
3											
4											
5											
6											
7											
8											
9											
10											

DATA INPUT: PS JAN.21/98

SOILM6

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

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REPORT
ON

FOUNDATION INVESTIGATION
PRELIMINARY DESIGN
FOR REPLACEMENT OF PORCUPINE RIVER BRIDGE
W.P. 313-85-00, SITE 39E-083
HIGHWAY 101
MTO DISTRICT 53, NEW LISKEARD, ONTARIO

File 42A-50

Submitted to:

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Mississauga, Ontario

March 1998

971-1191-1

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Table 1 — Point Load Test Results; Rock Core Samples
Porcupine River Bridge

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology

Record of Borehole Sheets

Drawing M1191002 — Highway No. 101 Crossing of Porcupine River
Borehole Locations and Soil Strata

Figures 1 and 2 — Grain Size Distributions

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a geotechnical investigation at the site of the proposed replacement of the bridge carrying Highway 101 over Porcupine River. This project is part of MTO's on-going bridge upgrading program and includes preliminary design for replacing the existing bridge and upgrading the approaches within 350 m of the structure. The site of the project is designated as Site 39E-083.

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed bridge structure by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect the design of the proposed bridge and approach embankments.

The terms of reference and the scope of work are outlined in our proposal letter P71-1437, dated October 3, 1997. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, dated October 29, 1997.

2.0 SITE DESCRIPTION

The site is located at the crossing of the existing Highway 101 over Porcupine River in the Township of Matheson, District of Cochrane, east of Timmins (MTO District 53, New Liskeard). Porcupine River is about 20 m in width at the bridge crossing. The river valley is relatively steep with several bedrock outcrops on both sides of the river. The water level in Porcupine River was measured to be at Elevation 274.6 m at the time of our investigation.

The existing Porcupine River bridge is a four span reinforced concrete girder about 60 m in length, built in 1939. A retaining wall has been constructed to support the embankment fill on the north side of the east "abutment". Gabion baskets have been placed at the ends of the bridge deck in all four quadrants. The current road grade / bridge deck is at about Elevation 287 m, some 12.5 m above the water level in the river. The existing approach embankments are 3 m to 4 m in height.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between December 9 and December 11, 1997. During this time, a total of eight boreholes were put down at the site. Two boreholes were drilled at each of the abutments (97-1, 97-2, 97-5 and 97-6), one borehole was drilled at each of the two proposed pier locations (97-3 and 97-4), and two shallow boreholes (97-7 and 97-8) were drilled within the immediate approaches to the bridge. In addition, probeholes were put down to determine the overburden depths and bedrock outcrop extent at the proposed piers. The investigation was carried out using a bombardier mounted CME 55 drill rig supplied and operated by Marathon Drilling Inc. of Ottawa and portable electric drilling equipment supplied and operated by OGS Inc. of Stittsville. The portable drilling equipment was used to drill the boreholes at the locations of the proposed piers, where access to the CME 55 drill rig was not possible.

In the boreholes, samples of the overburden were obtained at regular intervals of depth using 50 mm outside diameter split spoon samplers in accordance with Standard Penetration Test (SPT) procedures. Bedrock coring was carried out in Boreholes 97-1, 97-3, 97-4 and 97-6. NQ size core samples were obtained from Boreholes 97-1 and 97-6 and BQ size core samples were obtained from Boreholes 97-3 and 97-4. Groundwater conditions in the open boreholes were observed throughout the drilling operations. One 50 mm piezometer was installed in Borehole 97-2 to permit monitoring of the groundwater conditions at the borehole location.

The field work was supervised on a full-time basis by a member of our technical staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil and rock samples were identified in the field, placed in containers and transported back to our laboratory for further examination. Index and classification tests were carried out on selected samples.

The as-drilled borehole locations were located by our field personnel relative to the piers of the existing bridge. The borehole elevations were surveyed by our personnel relative to the temporary benchmark provided by McCormick Rankin (Survey Point 2805 located on the north-west corner of the central pier footing - Elevation 275.274 m). We understand that this elevation is referenced to Geodetic Datum. The northing and easting co-ordinates of the boreholes are shown on the Record of Borehole sheets and on Drawing M1191002, attached.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geological information, the site is located in the physiographic region known as the Canadian Precambrian Shield. The shield terrain comprises large expanses of intrusive rocks such as gneisses and gneissic or massive granite rocks. The rocks are geologically complex with considerable folding, intrusive activity, regional metamorphism and faulting. Pleistocene lacustrine / marine deposits have been laid down in depressions and are associated with the Glacial Lake Barlow-Ojibway. The local physiography is characterized by overburden consisting mainly of glaciolacustrine and glaciomarine deepwater silty clay deposits and an irregular, variable bedrock surface with frequent rock outcrops. Recent natural deposits of sand, silt and gravel can be found within the flood plains of the existing rivers.

4.2 Site Stratigraphy

The detailed subsurface soil, rock and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected samples, are given on the attached Record of Borehole and Drillhole sheets, following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered in the boreholes consist of as much as 4.1 m of fill materials at the bridge approach embankments underlain by a glaciolacustrine deposit of silty clay. On the west side of the river, a layer of sand and gravel underlies the silty clay in one borehole; boulders, cobbles, sand and gravel were noted in the borehole located close to the river channel. The overburden is underlain by andesite bedrock. The surface of the bedrock as encountered in the boreholes varies. In the boreholes drilled through the road embankment on the west and east sides of the existing bridge / river valley, the surface of the bedrock (inferred and proven) was encountered between 6.5 m and 8.1 m depth and between 4.7 m and 5.8 m depth, respectively. Within the river valley, the bedrock surface dips toward the base of the river channel. Several bedrock outcrops are exposed on the river valley slopes; however, some 2.2 m of overburden was encountered in one borehole put down on the west side of the river in close proximity to the river channel.

The locations of the boreholes and stratigraphic section showing the inferred subsurface conditions at the proposed bridge site are shown on Drawing M1191002. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Pavement Structure and Fill Materials

All boreholes, except Boreholes 97-3 and 97-4, were put down through the road embankments. Boreholes 97-1, 97-2, 97-5 and 97-6 were drilled through the pavement structure. The pavement structure consists of 280 mm to 300 mm of asphalt underlain by about 450 mm to 460 mm of granular base. Boreholes 97-7 and 97-8, drilled outside the pavement structure, encountered 600 mm and 460 mm, respectively, of granular road base. The road base material is underlain by a gravelly sand to sand and gravel fill and / or rockfill. About 1.6 m of silty clay fill underlies the granular fill in Borehole 97-5 at about 2 m depth. The base of the fill material where fully penetrated was encountered between 3.0 m and 4.1 m depth (between Elevations 284.0 m and 282.8 m). The measured water content on the samples of the granular fill range from about 3 per cent to 18 per cent. The measured water content on one sample of the silty clay fill was 23 per cent. A grain size distribution for one sample of the silty clay fill is shown on Figure 1.

The presence of rockfill within the embankments was inferred based on resistance to augering with persistent grinding and bouncing of the augers observed during drilling at some locations. In Borehole 97-1, the augers were advanced through the rockfill from 1.5 m depth to the base of the fill at 3 m depth. Cobbles and rockfill were inferred from resistance to augering in Boreholes 97-2, 97-7 and 97-8.

The granular fill was typically loose to compact with SPT 'N' values generally ranging from 7 blows to 14 blows per 0.3 m of penetration. Higher SPT 'N' values of 36 to greater than 100 blows per 0.3 m were measured at some locations; these values are thought to be associated with the rockfill.

No fill material was encountered in Boreholes 97-3 and 97-4, which were put down at the proposed pier locations on the west and east sides of the river, respectively.

4.2.2 Irregularly Layered Silty Clay to Clay

Underlying the fill materials is a lacustrine deposit of irregularly layered silty clay to clay with trace sand. On the west side of the river valley in Boreholes 97-1 and 97-2, the silty clay to clay was encountered at depths of 3.0 m and 4.1 m (Elevations 284.0 m and 282.8 m). The thickness of the silty clay to clay ranged from approximately 2.4 m to 4.2 m in these boreholes. In Borehole 97-3, located at the base of the river valley, 1.2 m of silty clay was encountered underlying the topsoil. Atterberg limits tests carried out on samples of this deposit from the west side of the river indicate liquid limit ranging from about 30 per cent to 53 per cent and plasticity indices ranging from 12 per cent to 25 per cent. Generally, the silty clay to clay is firm to stiff with SPT 'N' values ranging from 6 blows to 10 blows per 0.3 m of penetration. Measured water contents range from about 29 per cent to 38 per cent. A grain size distribution for one sample of the silty clay is shown on Figure 2.

On the east side of the river, the silty clay deposit was encountered at depths of 3.6 m and 3.7 m (Elevations 283.7 m and 283.3 m) in Boreholes 97-5 and 97-6, respectively. The thickness of the silty clay deposit ranged from approximately 0.9 m to 2.2 m in these boreholes. Atterberg Limit tests carried out on the silty clay samples from the east side of the river indicate liquid limits ranging from about 24 per cent to 39 per cent and plasticity indices from 9 per cent to 14 per cent. The silty clay is firm to hard with SPT 'N' values ranging from 8 blows to 42 blows per 0.3 m of penetration. Measured water contents range from about 24 per cent to 33 per cent.

4.2.3 Granular Deposits

The irregularly layered silty clay deposit in Boreholes 97-1 and 97-3 is underlain by approximately 0.9 m to 1.0 m thick deposit consisting of varying proportions of granular materials. In Borehole 97-1, the deposit is classified as sand and gravel with some silt and trace clay. The sand and gravel is dense with a measured water content on one sample of 14 per cent. In Borehole 97-3, the deposit becomes coarser and consists of predominantly boulders and cobbles with some silt, sand

and gravel. This granular deposit extends to the bedrock surface at about Elevation 278.9 m in Borehole 97-1 and to about Elevation 274.3 m in Borehole 97-3.

4.2.4 Bedrock

The bedrock was encountered at various elevations across the bridge site. In the boreholes put down through the approach embankments on the west and east sides of the existing bridge / river valley, the surface of the bedrock was encountered between about Elevation 278.9 m and 282.2 m (between 4.7 m and 8.1 m depth). The bedrock surface dips toward the river channel and there are several bedrock outcrops exposed on the river valley slopes. Along the alignment of the proposed east pier, there is bedrock outcrop over the southern portion. Based on probing carried out within the northern portion, up to 200 mm of topsoil overlies bedrock. On the west side of the river, two bedrock outcrops were noted; one smaller outcrop at the edge of the river within the southern limit of the bridge, with height up to 1.5 m above the existing ground surface, and a larger outcrop extending along the northern limit of the bridge about 2.5 m to 3 m in height. In Borehole 97-3, located between the outcrops, some 2.2 m of overburden (1.2 m of silty clay and about 1 m of boulders and cobbles with some silt, sand and gravel) was encountered above the bedrock surface.

The bedrock surface as proven by rock coring or inferred from resistance to augering at the borehole locations is as follows:

Borehole Number	*Bedrock Depth (m)*	*Bedrock Elevation (m)*
97-1	8.08	278.94
97-2	6.55	280.40
97-3	2.22	274.33
97-4	0.00	275.21
97-5	5.79	281.54
97-6	4.72	282.24

Rock coring was carried out for a depth of 3.7 m, 3.1 m, 3.6 m and 3.2 m in Boreholes 97-1, 97-3, 97-4 and 97-6, respectively. The bedrock samples obtained consist of brownish grey to grey, medium grained andesite with trace chlorite and talc. Frequent joints with iron staining were noted within the core samples retrieved.

In Borehole 97-1, measured Rock Quality Designation (RQD) values of 13 per cent to 48 per cent were obtained on core samples. Measured RQD values on core samples from Boreholes 97-3, 97-4 and 97-6 were 14 per cent to 79 per cent, 0 per cent to 59 per cent, and 8 per cent to 19 per cent, respectively.

Point load testing was carried out on rock samples retrieved from the boreholes. The diametral Point Load Indices $Is_{(50)}$ measured on the andesite samples ranged from about 0.1 MPa to 1.3 MPa; an average uniaxial compressive strength for the intact rock of about 15 MPa is inferred from these values. The results of the point load tests carried out on the core samples obtained are summarized in Table 1, following the text of this report.

4.3 Groundwater Conditions

A piezometer was installed in Borehole 97-2. Details of the piezometer installation and of the water level measurements are shown on the attached Record of Borehole sheets. The piezometer in Borehole 97-2 with tip at Elevation 280.5 m within the silty clay deposit was dry on December 20, 1997. The water level in open Borehole 97-1 was observed at 7.9 m depth (Elevation 279.1 m) during drilling within the sand and gravel deposit. Boreholes 97-2, 97-5 to 97-8 were dry during drilling within the overburden. The water level in Porcupine River was measured to be at Elevation 274.6 m at the time of our investigation.

Based on these observations, it is considered that the groundwater table is controlled by the gravelly overburden immediately overlying the bedrock, by the fractured rock and by the river water level. It should be noted that the groundwater level is subject to seasonal and river water level fluctuations.

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the new Porcupine River bridge based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The works described in this report are associated with the proposed bridge designated as Site 39E-083 and the approaches within 20 m of the structure. It is understood that the new bridge to carry Highway 101 over Porcupine River will be a three span structure about 69 m in length and about 12 m in width, some 1 m wider on each side of the existing bridge. The final road grade will be 900 mm higher than the existing grade on the east side of the bridge and 100 mm higher on the west side of the bridge.

5.2 Bridge Foundations

In summary, the soils encountered in the boreholes at the bridge approach embankments consist of up to 4.1 m of fill materials underlain by glaciolacustrine silty clay. The overburden is underlain by andesite bedrock. The bedrock surface elevation as encountered in the boreholes is variable. In the boreholes drilled through the road embankment at the proposed abutment locations, the surface of the bedrock was encountered between 6.6 m and 8.1 m depth at the west abutment and between 4.7 m and 5.8 m depth at the east abutment. There are several bedrock outcrops present on the river valley slopes within the existing bridge footprint and along the proposed pier alignments. Based on the probing, the overburden cover is up to 200 mm thick in the vicinity of the east pier. On the west side of the river, the surface of the bedrock is undulating with rock outcrops on the north and south limits of the existing bridge and with depressions between the outcrops. About 2.2 m of overburden soils were encountered at the location of Borehole 97-3 in the vicinity of the west pier.

It is understood that the proposed east pier was shifted about 1.7 m to the west from the initially proposed alignment after completion of the investigation. The currently proposed alignment is at the edge of and extending into the river. Bedrock is exposed at the river edge on the south side of the east pier; the riverbed is thought to be at about Elevation 274 m. The borehole at the west pier was drilled about 2 m east of the proposed pier location due to steeply sloping ground at the pier centreline. Based on the subsurface information obtained, the bedrock surface at the borehole locations at and in the vicinity of the proposed foundation units is as follows:

Foundation Unit	*Borehole Number*	*Bedrock Depth Below Ground Surface (m)*	*Bedrock Surface Elevation (m)*
West Abutment	97-1	8.08	278.94
	97-2	6.55	280.40
West Pier *	97-3 (2 m east of pier centreline)	2.22	274.33
East Pier **	97-4 (2.5 m east of pier centreline)	0.00	275.21
	Probeholes	0.20	275.3
East Abutment	97-5	5.79	281.54
	97-6	4.72	282.24

* Bedrock is exposed at ground surface at about Elevation 278 m to 280 m at the northern limit of the proposed west pier footing.

** Bedrock is exposed at ground surface at about Elevations 274 m to 275 m over the southern portion of the proposed east pier footing.

Spread footings placed on the surface of the bedrock may be considered for the support of the abutments and the piers. For design, the following founding elevations may be assumed:

West Abutment	Stepped between Elevation 278.8 m and 280.4 m;
West Pier	Elevation 274 m; stepped up as required at the north and to key into rock outcrop;
East Pier	Stepped and maintained in bedrock 1 m below ground / riverbed surface (see explanation below); and
East Abutment	Stepped between Elevation 281.5 m and 282.2 m

It should be noted that Borehole 97-3 is located near the east limit of the west pier. The ground and bedrock surface is extremely variable at this pier location and the bedrock surface could be higher than that indicated above at the west limit of the pier.

Since bedrock is exposed at surface in the southern portion of the east pier footing and since the footing extends into the river channel, the depth of the footing will be governed by scour requirements and / or typical cover requirements. In addition, some protection against frost action is considered appropriate given the extent of fracturing of the bedrock in the upper 1 m or so at this pier location. For this reason, it is recommended that the footing be placed at least 1 m below the existing ground surface (where bedrock is exposed at ground surface) and at least 1 m below the river bed level (but within or on bedrock) where the footing extends into the river. There has been no probing completed along the north portion of the footing where it extends into the river; however, it has been assumed based on the site conditions that bedrock will not be at a depth greater than 1 m in this area. Provision should be made in the contract to deepen the excavation as may be required to ensure that the footing is placed on the bedrock surface. It is expected that the majority of the excavation to 1 m depth will be through fractured bedrock; it may be necessary, however, to excavate into less fractured bedrock at some locations using controlled drill and blast excavation techniques.

5.2.1 Factored Geotechnical Resistance

Spread footings placed on the surface of the bedrock or within the bedrock may be designed for a geotechnical resistance at Ultimate Limit States (ULS) of 1,000 kPa. This value is for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as appropriate. Given the highly fractured nature of the bedrock in the upper approximately 1 m or so at some locations, some settlement of footings placed on the bedrock surface may occur. For design, therefore, it is recommended that a value of 1,000 kPa may be assumed for the geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement. A characteristic or unfactored value for the coefficient of friction between the concrete and the fractured bedrock may be taken as 0.5 for assessment of sliding at SLS.

The design assumptions with respect to founding elevations should be confirmed by inspection by qualified personnel in the field during construction. All footing excavations should be inspected prior to placing concrete to ensure that the base has been adequately cleaned and that the bedrock conditions as exposed at the founding level are consistent with the design assumptions.

Alternatively, depending on the configuration of the abutments, consideration could be given to the use of caissons socketted into the bedrock. The caissons should be designed for shaft friction assuming contribution from the length of socket formed within the bedrock. Once the abutment configuration is established, an assessment of the contribution of shaft friction from the overburden can be made. An unfactored unit shaft friction value of 1,350 kPa may be used for the initial design of rock socket length. To obtain the factored geotechnical resistance, a resistance factor of 0.4 should be used. The caissons should have a minimum socket length of 3 m. Assuming a socket diameter of 1.2 m for caissons socketted 3 m into the bedrock, this results in a factored geotechnical resistance (axial capacity) at ULS of 6,100 kN. Settlement of caissons socketted at least 3 m into the bedrock is expected to be less than 25 mm. In general, SLS conditions are not applicable for geotechnical bedrock resistance assessment.

Given the proposed grade raise of 900 m on the east side and 100 mm on the west side for the approach embankments, the caissons would be subject to negative skin friction as a consequence of consolidation settlement of the silty clay deposit underlying the embankment fill. An unfactored value for negative skin friction on a single 1.2 m diameter caisson of 600 kN should be assumed for design. An appropriate load factor will need to be applied to this load onto the caisson as a result of negative skin friction.

For frost protection, the base of the caisson caps should be provided with at least 2.4 m of soil cover.

5.2.2 Resistance to Lateral Loads for Caissons

Where caissons are adopted at the abutments, the horizontal reaction to the caissons can be calculated from the expression:

$k_s = z \times n_h / d$,

where

k_s = coefficient of horizontal subgrade reaction (MPa/m)
d = caisson diameter (m)
n_h = constant of horizontal subgrade reaction (MPa/m)
z = depth below ground surface (m)

The constant of horizontal subgrade reaction depends on the soil type and soil density / consistency around the caisson shaft. For design, the values (or range of values) indicated in the table below may be assumed. All values quoted are unfactored geotechnical parameters.

Location	*Elevation (m) (approximately)*	*Soil Type*	*z x n_h (MPa)*
West Abutment	Ground Surface to 283.5	Fill: Gravelly Sand / Sand and Gravel / Rockfill, very loose to dense	z x 2.0 to z x 5.0
	283.5 to 280.0	Silty Clay to Clay, firm to stiff	2.0 to 3.0 (constant with depth)
	280.0 to 279.0	Sand and Gravel, dense	z x 6.0
	below 279.0	Andesite Bedrock	z x 12
East Abutment	Ground Surface to 283.0	Fill: Silty Clay, soft to firm and Gravelly Sand /Sand and Gravel, compact to dense	z x 1.0 to z x 2.0
	283.0 to 282.0	Silty Clay, stiff	3.0 (constant with depth)
	below 282.0	Andesite Bedrock	z x 12

Group action for lateral loading should be considered when the caisson spacing in the direction of loading is less than six to eight caisson diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

Caisson Spacing in Direction of Loading d = Caisson Diameter	*Subgrade Reaction Reduction Factor R*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.3 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments walls in accordance with OHBDC:

- Select free-draining non-frost susceptible granular fill meeting the specifications of OPSS Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 2.4 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3.

- For Case I, the pressures are based on the in-situ soils / embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming the in-situ soils and / or clean earth fill)	20 kN/m^3

Coefficients of lateral earth pressure:

'active'	0.33
'at rest'	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular A	**Granular B**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficient of Lateral Earth Pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

5.4 Excavations

Excavations for spread footing construction at the abutments could extend to about 5 m to 8 m depth below existing ground surface. Excavations will be through loose to dense granular and soft to firm cohesive fill materials and through firm to hard silty clay to clay. At the east abutment, the base of the excavation will be above the groundwater level. Some water flow into the excavation should be anticipated at the west abutment location, where the water bearing sand and gravel deposit was encountered overlying fractured bedrock. If space permits, the excavations can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical. Pumping from the sand and gravel deposit will be required to maintain the side slope stability and a dry excavation for concrete placement.

Alternatively, a vertical sided supported excavation could be considered for the abutment footings. Temporary support could consist of a braced soldier pile and lagging system. Soldier piles should be socketted into the bedrock below the proposed base of the excavation.

If deep foundations are adopted, excavations for the abutment pile caps (assumed to be at about Elevation 284 m) will extend some 3 m to 4 m below the ground surface. Excavations which will be open for a relatively short period of time can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical. Temporary liners will be required through the overburden and sealed into the bedrock for caisson construction to provide a cut off to water inflow through the sand and gravel and the upper fractured bedrock, to provide support to the sides and to maintain a dry excavation for inspection and concrete placement.

At the east pier, excavation for spread footing construction will be made through the riverbed materials and the bedrock and will extend to below the river water level. Considerable water inflow could occur through the fractured rock. A cofferdam will be required to divert the river around the proposed pier footing excavation. The water flow through the fractured bedrock will have to be controlled to permit footing construction in the "dry".

At the west pier, excavation for spread footing construction could be as much as 4.5 m deep below ground surface, based on the contour plan provided. The base of the excavation will be about 0.5 m below the river water level which was measured to be at Elevation 274.6 m in December 1997. The excavation at the west pier will extend through silty clay to clay and the underlying coarse granular deposit of sand, gravel cobbles and boulders. Temporary unsupported cuts through the overburden may be made with the side slopes indicated above provided that adequate groundwater control is implemented to control the flow anticipated from the sand and gravel and fractured bedrock. Where the excavation is extended through the bedrock, vertical sides will be suitable; however, some ravelling of the fractured bedrock should be expected. Typically, the upper 1 m of bedrock is fractured; however, where excavation is extended into less fractured bedrock, the use of controlled blast and drill excavation techniques will be required. Water inflow into the excavation should be anticipated through the granular deposit and the fractured portion of the bedrock. Some form of groundwater control will be required to permit excavation and spread footing construction in the "dry". Closed steel sheetpiling could be used as cut-off to groundwater flow and to form temporary support to the excavation; however, given the variable bedrock surface, it may not be possible to adequately key the sheetpiling into the bedrock. Pumping from sumps formed at the base of the excavation would likely also be required. Sumps should be maintained outside the footing area.

A mud coat should be placed on the prepared subgrade at the founding level immediately following cleaning and inspection.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

5.5 Approach Embankments

The proposed approach at the crossing will be widened by about 1 m on both sides of the embankment and it is understood that there will be a grade raise of 900 mm on the east side and 100 mm on the west side of the bridge. Given the nature of the subsoils, and the proposed raising and widening of the embankment over the existing grade, stability of the proposed embankments is not a concern with respect to deep seated failure through the founding soils. Some nominal long-term settlement of the east approach embankment should be expected due to consolidation of the silty clay to clay deposit underlying the embankment.

The magnitude of stress increase due to the nominal widening and raising of the embankment has not warranted detailed oedometer testing of the silty clay deposit. The consolidation parameters of the deposit were established based on the water content test results and compared to published values and tempered by our general experience with clay compressibility parameters. Based on these parameters, it is estimated that settlement due to increased load induced by embankment construction will be less than 25 mm. Given the magnitude of settlement, preloading and surcharging to induce/speed up the consolidation process are not considered necessary.

5.6 Subgrade Preparation and Embankment Construction

Topsoil and organic deposits should be stripped from below the proposed embankment widening areas and the exposed subgrade soils should be proof-rolled prior to fill placement. The existing embankment side slopes should be stripped and benched in accordance with OPSD 208.10 to ensure that the new fill is keyed into the existing fill and to minimize differential settlement between the new and the existing embankments.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or Select Subgrade Material (in accordance with OPSS 1010) depending on the material available to the project. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The permanent slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical. Vegetation cover should be established on all slopes to protect embankment fill against surficial erosion.

Alternatively, the approach embankments could be constructed using rockfill if available to the project. The permanent side slopes of the rockfill embankments should be maintained not steeper than 1.25 horizontal to 1 vertical, although the final slope would be governed by the existing embankment slope with respect to the proposed widening.

5.7 Detour

It is understood that a two lane detour is to be constructed along an alignment immediately to the north of the existing highway. Embankment heights of up to 4 m (slightly lower than the existing embankment) are required. It is understood that consideration is being given to the use of reinforced earth walls and Sierra slopes to maintain the detour embankment within the property limits.

The detour bridge may be supported on spread footings or caissons as outlined in Section 5.2 of this report. The geotechnical resistances as provided in Section 5.2 may be used for design of the foundations for the detour.

Stability of the proposed detour embankment to the east of the bridge is considered to be sufficient for the proposed configuration of vertical reinforced earth walls or 1:1 Sierra slopes. The leveling pad should be founded on the hard silty clay deposit anticipated at about 1 m depth below ground surface. To the west of the detour bridge, however, embankment will be constructed over the firm silty clay to clay deposit. The stability of the proposed vertical wall has been examined based on the completed borehole results. For the proposed embankment height of not more than 4 m, it is determined that the factor of safety against overall failure is greater than 1.5.

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WORD S/FINALDAT/1100/971-1191/1998/71191CR

TABLE 1

POINT LOAD TEST RESULTS; ROCK CORE SAMPLES
PORCUPINE RIVER BRIDGE

Borehole Number	*Sample Depth (m)*	*$Is_{(50)}$*	
		(MPa)	*Diametral (D) Axial (A)*
971-1	8.7	0.103	D
	9.4	0.362	D
	11.5	0.465	D
	10.0	0.745	A
	10.6	0.587	A
97-3	1.2	1.001	D
	5.3	1.216	D
	3.9	0.858	D
	1.8	1.318	A
	4.5	0.550	A
	5.2	0.541	A
97-4	1.4	0.644	D
	3.3	0.501	D
	2.4	0.274	A
	3.0	0.220	A
	1.7	0.274	A
97-6	6.1	0.482	D
	7.0	1.318	D
	5.5	1.014	D
	5.7	0.495	A
	7.5	0.240	A
	7.3	0.320	A

WORD S/FINALDAT/1100/971-1191/1998/71191CT1

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ - Parallel To	
$\perp$ - Perpendicular To	

RECORD OF BOREHOLE 97-1

W.P. 313-85-00
DIST. 53, SITE: 39E-083, HWY 101
LOCATION: N 5379012.10; E 300711.02

BORING DATE: DEC.11/97

DATUM: GEODETIC
PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) 10 20 30 40

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	ROAD SURFACE	287.02					
	ASPHALT	0.00					
	Granular A (Road Base)	286.74 0.28					
	Gravelly Sand, trace silt Loose Brown Moist (FILL)	286.56 0.46	1	50 DO	8		
	Rockfill (inferred from resistance to augering - no sample recovery)	285.50 1.52	2	50 DO	>100		
			3	50 DO	>100		
	Silty Clay to Clay, irregularly layered, trace sand Firm to stiff Brown Moist	284.02 3.00	4	50 DO	8		
			5	50 DO	7	MH	
			6	50 DO	10		
			7	50 DO	6		
	Sand and Gravel, some silt, trace clay Dense Brown Wet	279.82 7.20	8	50 DO	49		
	REFUSAL TO AUGER PENETRATION BOREHOLE CONTINUED. FOR ROCK CORING DETAILS REFER TO RECORD OF DRILLHOLE, SEE SHEET 2.	278.94 8.08					NOTE: Water level at 7.9m depth during drilling in overburden.
10	CONTINUED ON NEXT PAGE						

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379012.10; E 300711.02

RECORD OF DRILLHOLE: 97-1

DRILLING DATE: DEC.11/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

SHEET 2 OF 2

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
7		CONTINUED FROM PREVIOUS PAGE														
		For overburden stratigraphy, refer to Record of Borehole 97-1, sheet 1.														
8	NQ CORING	Brownish grey, medium grained ANDESITE, trace chlorite and talc on joints, some iron staining. (BEDROCK)		278.94 8.08	1								J,PL,R (x5) J,ST,R J,S,R Broken core J,I,R (x4) J,U,R J,PL,R (x3)			
9–11					2								Broken core J,PL,R (x5) J,PL,R (x5)			
					3								J,PL,SM J,R,E J,U,R			
12		END OF DRILLHOLE		275.20 11.82												
13																
14																
15																
16																
17																

DEPTH SCALE: 1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

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DATA INPUT: ?? JAN.21/98

MBORNV5

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W.P. 313-85-00

DIST. 53, SITE: 39E-083,HWY 101

LOCATION: N 5379003.63; E 300714.98

RECORD OF BOREHOLE 97-2

BORING DATE: DEC.9/97

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

WATER CONTENT, PERCENT — Wp W Wl — 10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DESCRIPTION	ELEV. / DEPTH (m)
ROAD SURFACE	286.95
ASPHALT	0.00
Granular A (Road Base)	286.65 / 0.30
Gravelly Sand to Sand and Gravel with possible rockfill/cobbles inferred from resistance to augering Very loose to dense Brown Moist (FILL)	0.45
Silty Clay to Clay, irregularly layered, trace sand, trace gravel at depth Stiff to very stiff Brown Moist	282.84 / 4.11
END OF BOREHOLE REFUSAL TO AUGER PENETRATION (PROBABLE BEDROCK).	280.40 / 6.55

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	50 DO	14
2	50 DO	36
3	50 DO	7
4	50 DO	3
5	50 DO	> 100
6	50 DO	9
7	50 DO	22

Installation: PROTECTIVE PIPE; GROUT; BENTONITE SEAL; GRAVEL

NOTE:
Borehole dry during drilling and on completion of drilling.

Piezometer dry on Dec.20,1997.

DATA INP... JAN.21/98

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

W.P. 313-85-00

DIST: 53, SITE: 39E-083, HWY 101

LOCATION: N 5379019.42; E 300732.99

RECORD OF DRILLHOLE: 97-3

DRILLING DATE: DEC.10&11/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

SHEET 1 OF 1

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
0		GROUND SURFACE		276.55												
		TOPSOIL		0.00												
		Silty Clay		0.08												
1		Boulders and cobbles with some silt, sand and gravel.		275.33 / 1.22												
2	BQ CORING	Medium grained, grey ANDESITE, heavy iron staining, trace chlorite on joints. (BEDROCK)		274.33 / 2.22	1								Broken core; J,PL,SM			
3					2								Broken core 0.6m; J,PL,SM; J,PL,SM; J,PL,SM; J,PL,R; J,PL,R(x2)			
4					3								J,PL,SM (x2); J,PL,R; J,PL,SM; J,PL,R; J,U,R; J,PL,SM			
5				271.28 / 5.27	4								J,U,SM; J,U,SM; J,U,SM; J,PL,SM; J,PL,R			
		END OF DRILLHOLE														
6																
7																
8																
9																
10																

DEPTH SCALE:

1 to 50

Golder Associates

LOGGED: MB

DATE:

CHECKED: AMP

RECORD OF DRILLHOLE: 97-4

W.P. 313-85-00
DIST. 53, SITE: 39E-083, HWY 101
LOCATION: N 5379026.26; E 300760.03

DRILLING DATE: DEC.10/97
DRILL RIG: BOMBARDIER CME-55
DRILLING CONTRACTOR: MARATHON DRILLING

SHEET 1 OF 1
DATUM: GEODETIC
PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
0		GROUND SURFACE		275.21												
	BQ CORING	Medium grained, grey ANDESITE, trace chlorite and talc on joints. Minor amounts of mica and quartzite sections. (BEDROCK)		0.00	1								Broken core to 0.9m depth			
1													J,ST,R J,PL,SM J,PL,SM J,PL,SM (x2)			
2					2								Broken core J,PL,R J,PL,SM J,PL,R J,I,R Broken core			
3					3								J,PL,R J,PL,SM (x2) J,PL,R J,ST,R J,PL,R J,PL,R J,PL,R			
				271.61									J,PL,SM			
		END OF DRILLHOLE		3.60												
4																
5																
6																
7																
8																
9																
10																

DEPTH SCALE:
1 to 50

Golder Associates

LOGGED: MB
DATE:
CHECKED: AMP

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DATA INPUT: ? JAN.21/98

RECORD OF BOREHOLE 97-5

W.P. 313-85-00
DIST. 39E-083,HWY 101
LOCATION: N 5379040.23; E 300773.68

BORING DATE: DEC.9/97

SHEET 1 OF 1
DATUM: GEODETIC
PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)
ROAD SURFACE		287.33
ASPHALT		0.00
Granular A (Road Base)		287.05 / 0.28
Gravelly Sand to Sand and Gravel, trace silt, trace clay, Loose, Brown, Moist (FILL)		286.87 / 0.46
Silty Clay, trace to some sand, trace gravel, trace organics, Soft to firm, Moist (FILL)		285.30 / 2.03
Silty Clay, irregularly layered, trace sand, trace organics, Firm to hard, Brown, Moist		283.73 / 3.60
END OF BOREHOLE REFUSAL TO AUGER PENETRATION (PROBABLE BEDROCK).		281.54 / 5.79

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 DO	8	
2	50 DO	9	
3	50 DO	5	
4	50 DO	4	MH
5	50 DO	8	
6	50 DO	42	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp W Wl — 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION

Note: Open borehole dry on completion of drilling.

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

MPRB0006.BHS

W.P. 313-85-00

DIST. 39E-083,HWY 101

LOCATION: N 5379031.37; E 300777.32

RECORD OF BOREHOLE 97-6

BORING DATE: DEC.10/97

SHEET 1 OF 2

DATUM: GEODETIC

PROJECT: 971-1191

BORING METHOD: BOMBARDIER CME-55

DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
ROAD SURFACE	286.96 / 0.00			
ASPHALT	286.68 / 0.28			
Granular A (Road Base)	286.50 / 0.46			
Gravelly Sand to Sand and Gravel, trace silt, occ. cobble, silty clay lenses Compact to very dense Moist Brown (FILL)	283.26 / 3.70	1	50 DO	11
		2	50 DO	10
		3	50 DO	14
		4	50 DO	67
Silty Clay, irregularly layered, trace sand, trace gravel, trace organics Stiff Brown Moist	282.39 / 4.57	5	50 DO	14
Sand, some gravel Very dense Brown Moist	4.72	6	50 DO	55/.15
REFUSAL TO AUGER PENETRATION BOREHOLE CONTINUED. FOR ROCK CORING DETAILS REFER TO RECORD OF DRILLHOLE, SHEET 2.				

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp —○W— Wl — 10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

NOTE: Open borehole dry during drilling through overburden.

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

CONTINUED ON NEXT PAGE

DATA INPUT: FJ JAN.21/98

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

GOLMC

W.P. 313-85-00

DIST. 53, SITE: 39E-083, HWY 101

LOCATION: N 5379031.37; E 300777.32

RECORD OF DRILLHOLE: 97-6

DRILLING DATE: DEC.10/97

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING

DATUM: GEODETIC

PROJECT: 971-1191

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min.)	FLUSH / COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
4		CONTINUED FROM PREVIOUS PAGE														
		For overburden stratigraphy, refer to Record of Borehole 97-6, sheet 1.														
	NQ CORING	Brownish grey, medium grained ANDESITE, trace chlorite and talc on joints. (BEDROCK)		282.24 / 4.72	1								Broken core 0.6m			
5													J,PL,R (x3)			
													J,PL,R			
6													J,U,R			
													J,PL,R			
					2								J,PL,SM (x2)			
													J,ST,R J,PL,R			
													Broken core			
7													J,PL,SM			
													J,PL,SM			
													J,ST,SM			
													J,U,SM			
													J,PL,SM			
													J,I,SM			
													J,PL,R (x4)			
8		END OF DRILLHOLE		279.04 / 7.92												
9																
10																
11																
12																
13																
14																

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DATA INPUT: PS JAN.21/98

DEPTH SCALE:
1 to 50

Golder Associates

LOGGED: MB
DATE:
CHECKED: AMP

W.P. 313-85-00

RECORD OF BOREHOLE 97-7

DIST. 39E-083,HWY 101

BORING DATE: DEC.11/97

DATUM: GEODETIC

LOCATION: N 5379006.56; E 300696.15

PROJECT: 971-1191

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	BOMBARDIER CME-55	ROAD SURFACE		287.10							
		Granular A		0.00							
		Granular B		286.82 0.28							
		Rockfill (inferred from resistance to augering)		286.50 0.60							
1		REFUSAL TO AUGER PENETRATION END OF BOREHOLE		286.03 1.07							NOTE: Open borehole dry on completion of drilling operations.
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

RECORD OF BOREHOLE 97-8

W.P. 313-85-00
DIST. 39E-083,HWY 101
LOCATION: N 5379036.55; E 300792.50

BORING DATE: DEC.10/97

SHEET 1 OF 1
DATUM: GEODETIC
PROJECT: 971-1191

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	BOMBARDIER CME-55	ROAD SURFACE		287.24							
0		Granular A		0.00							
		Granular B		0.15							
		Sand and Gravel Dense (FILL)		286.78 / 0.46							
1					1	50 DO	33		O (water content)		
		Rockfill (inferred from resistance to augering)		285.56 / 1.68	2	50 DO	> 100				
2											
		REFUSAL TO AUGER PENETRATION END OF BOREHOLE		284.73 / 2.51							NOTE: Open borehole dry on completion of drilling operations.
3											
4											
5											
6											
7											
8											
9											
10											

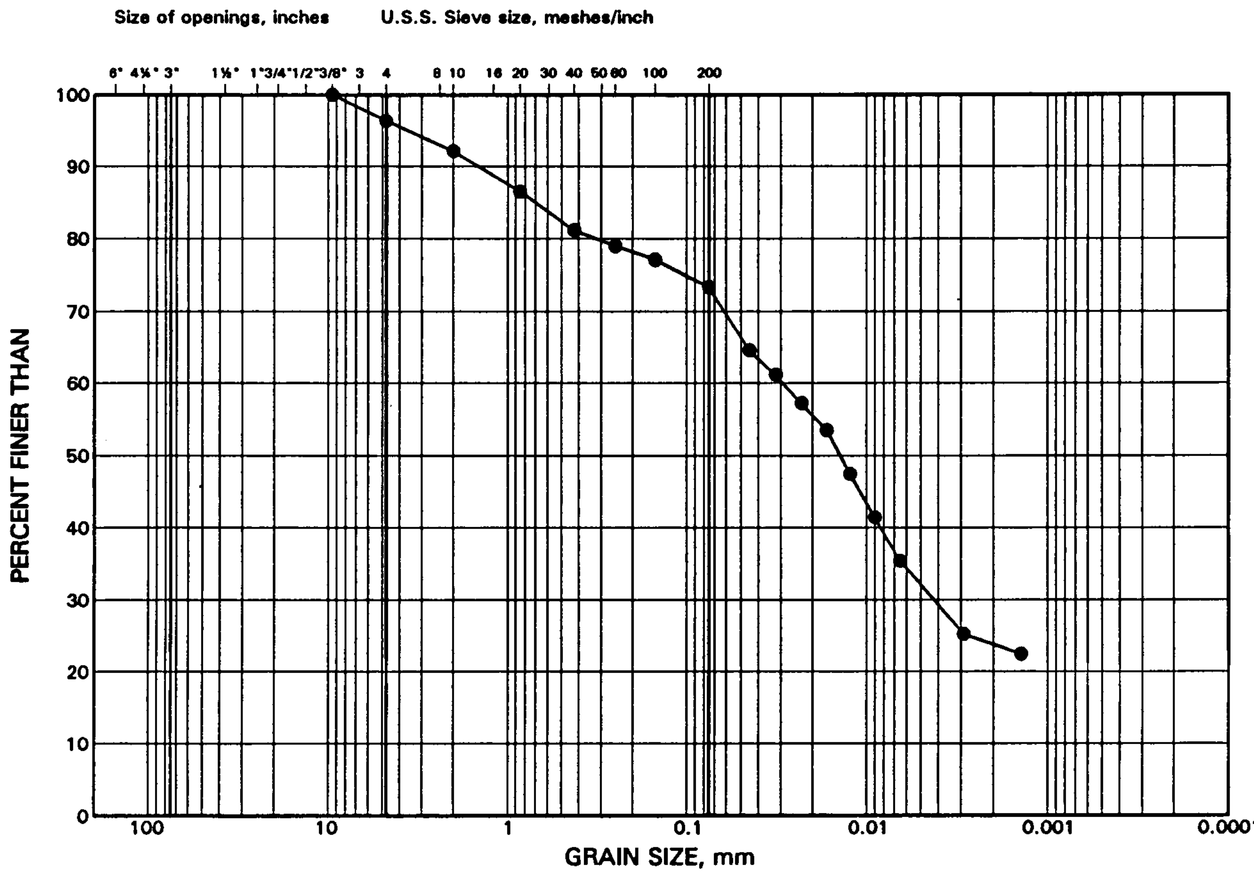
DEPTH SCALE
1 to 50

Golder Associates

LOGGED: MB
CHECKED: AMP

GRAIN SIZE DISTRIBUTION
Silty Clay (Fill)

FIGURE 1



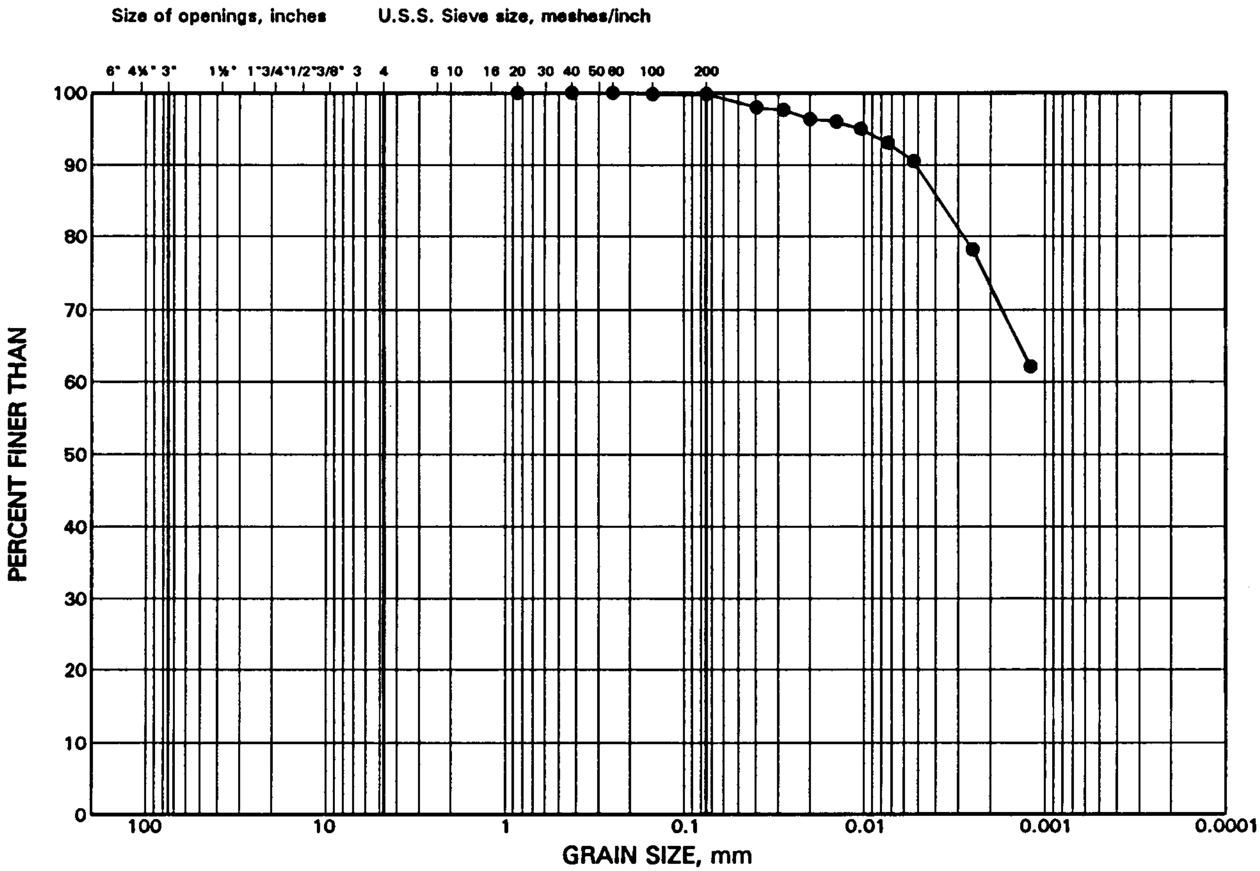
COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-5	4	284.0

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 2



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-1	5	283.0

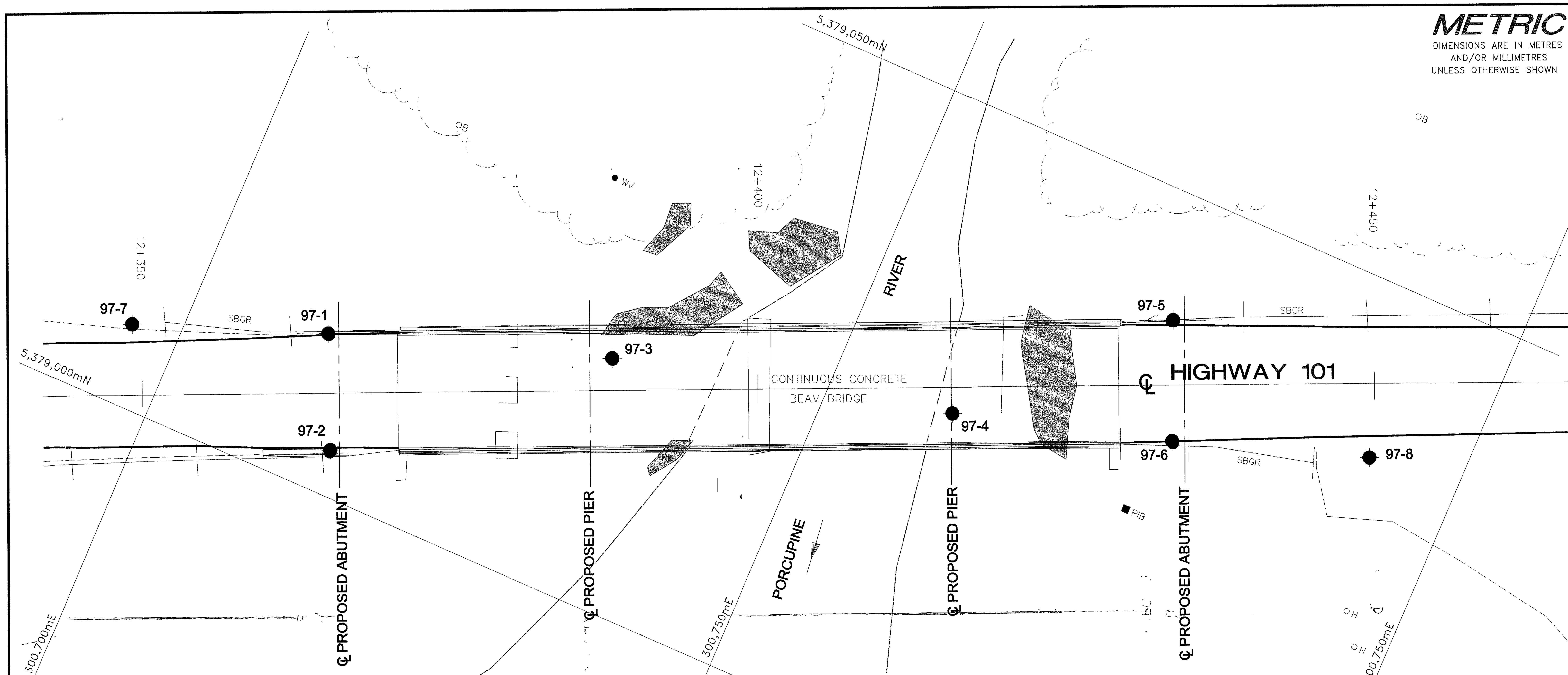
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 313-85-00

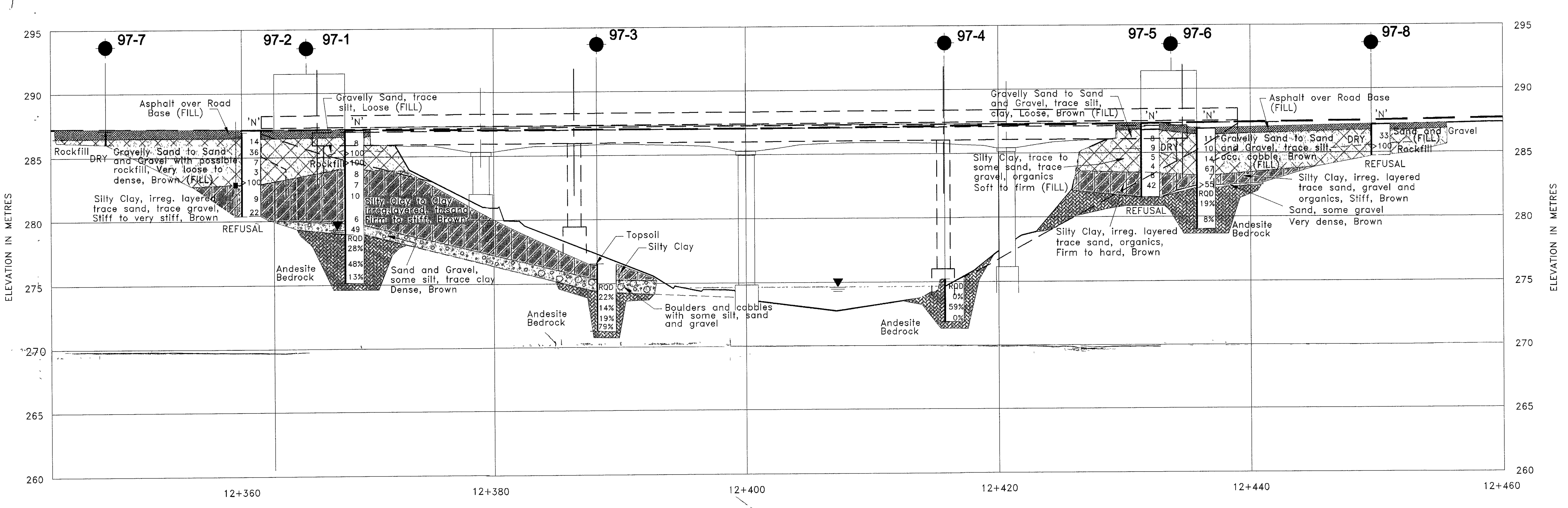
HIGHWAY No. 101 CROSSING OF PORCUPINE RIVER
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

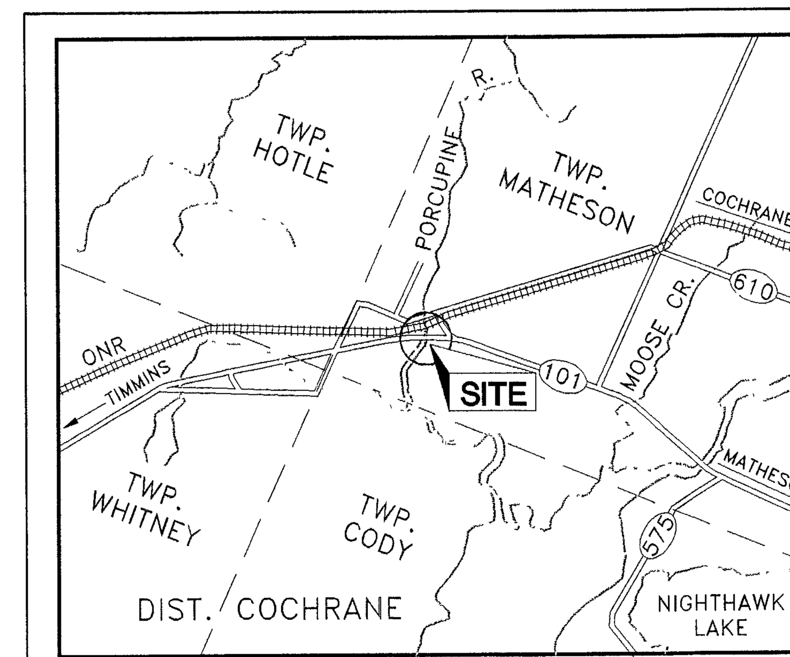
Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN



PROFILE



KEY PLAN

LEGEND

Symbol	Description
●	Bore Hole
⊕	Dynamic Cone Penetration Test (Cone)
◐	Bore Hole & Cone
N	Blows/0.3m (Std. Pen. Test, 475 j/blow)
Cone	Blows/0.3m (60° Cone, 475 j/blow)
▼	WL at time of investigation 1997 08
Rk	Bedrock Outcrop

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
97-1	287.02	5,379,012.10	300,711.02
97-2	286.95	5,379,003.63	300,714.98
97-3	276.55	5,379,019.42	300,732.99
97-4	275.21	5,379,026.26	300,760.03
97-5	287.33	5,379,040.23	300,773.68
97-6	286.96	5,379,031.37	300,777.32
97-7	287.10	5,379,006.56	300,696.15
97-8	287.24	5,379,036.55	300,792.50

NOTES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 101		PROJECT NO.: 971-1191	53
SUBM'D. AMP	CHKD:	DATE: 1998 01 19	SITE 39E-083
DRAWN: MHW	CHKD.	APPD.	DWG. M1191002

ACAD FILE M1191002.DWG

1:200

MEMORANDUM

TO: Ed Carriere - Golder Sudbury — August 28, 2000

FR: Fin Heffernan — 991-1177

RE: **TEMPORARY DETOUR BRIDGE**
PORCUPINE RIVER
HIGHWAY #101

BACKGROUND

We carried out the investigation for the main 3 span bridge replacement (971-1191-1) in 1997 using a CME 55 drill-rig, (Marathon). Report 971-1191-1 borehole logs and partial text were faxed to Sudbury on August 25. We core drilled at each foundation unit and at each abutment we put down a second hole which was taken to auger refusal.

In 1999 we carried out an investigation for a one span "Bailey Bridge" using a portable electric powered drill (Colbar Resources of Sudbury), Report 991-1177 is in Sudbury office.

SOIL CONDITIONS

Below the roadway fill (granular and rockfill), the 1997 investigation encountered a firm to stiff clay deposit, followed by a thin granular deposit and then by Andesite bedrock. The bedrock surface in Borehole 97-6at the east abutment was at Elevation 282.2 and refusal level in Borehole 97-5 was at Elevation 281.5. At the west abutment Borehole 97-1 encountered the bedrock slightly lower (Elevation 278.9) with refusal in Borehole 97-2 at Eevation 280.4

In the 1999 investigation for the detour bridge, boreholes at or near the west abutment (99-2, 99-3, 99-4) encountered refusal by the portable drill at Elevation 280 to 281. At the boreholes at or near the east abutment (Boreholes 99-5, 99-6 and 99-7) refusal was encountered at Elevation 278.5 to 282).

The bedrock outcrops at several locations at the bridge site and the bedrock surface of these outcrops is irregular.

FOUNDATION DESIGN

The foundation design selected for the detour bridge has the abutment footings founded on a reinforced earth mass or block which in turn is placed on a compacted Granular "A" fill which levels the irregular founding station (bedrock or dense granular deposit). Because of the sloping nature of the site, for stability reasons we did not want to found on the firm to stiff clay. The top surface of the Granular "A" pad is planned at Elevation 281.5 at the west abutment and Elevation 282.0 at the east abutment.

By memo in February, 2000 (copy attached) we gave the bearing value of the granular deposit as 800 kPa ULS and 300 kPa SLS (based on 1 inch settlement). I have checked with the reinforced earth personnel and the loading that their system imposes is 220 kPa ULS and 170 kPa SLS, that is well less than what was given to them. In addition the nature of the Bailey Bridge is that it is tolerant to settlement.

TEST PIT RESULTS AT DETOUR BRIDGE

a) **West Abutment Area**

A line of test pits, as shown on the plan area,were put down by the contractor along the front face of the west abutment from north of the abutment to just beyond the center of the abutment. Rock is identified in an irregular pattern between Elevations 280.1 and 281.9 within the north half of the abutment. These elevations correspond quite well with the refusal levels found in the 1999 investigation at the south end of the abutment area, to Borehole 99-4 met refusal at 279-9.

b) **East Abutment Area**

Test-pitting in the east abutment area encountered rock at Elevation 278 at the north end, which is 1.8 metres below the refusal level in Borehole 99-6. At the center of the abutment, rock was encountered at Elevation 276.3, which is some 6 metres below the refusal level in Borehole 99-5.

SITE PROGRAM

You should carefully log and photograph the three test pits in the east abutment area, especially the two within the abutment area. A sign should be placed in each photograph area indicating which test pit, i.e. E1 (for east abutment), E2, E3 and W4 (for west abutment) W5 etc. Good ammunition against a claim.

We are particularly interested in the bottom elevation of the clay/top surface of the granular deposit. When identified get site personnel to provide elevations to you for this interface. We would like to excavate to this interface and begin our Granular "A" pad at that elevation and bring it up to elevation 282.

You should log and photograph the test pits at the west abutment as well. The problem is somewhat different here as the bedrock at some locations is a little too high at the center of the abutment area (above the Granular "A" platform level of 281.5). The reinforced earth people will have to decide whether the high points in the rock are cut-off or whether the Granular "A" base is raised (with a possible shortening of the reinforced walls). As at the east abutment, please determine the elevation in the pits between the bottom of the clay/top of granular deposit. Where the rock appears low at the abutment we would want to place the Granular "A" on the granular deposit.

PERMANENT BRIDGE

Questions will probably be raised about the founding of the permanent bridge. We have a borehole at each foundation element, which has proved the bedrock by core drilling.

a) **West Abutment**

Founded on short H piles driven to surface at the bedrock. The piles are fitted with rock point because of the sloping bedrock at the site. The top of footing pile cap is Elevation 285. 185 and the refusal will be at about Elevation 279.

b) **East Abutment**
At the east abutment of the permanent bridge found bedrock at Elevation 282.2 by core drilling. Borehole 97-5 encountered refusal (with CME-55) at Elevation 281.5. The abutment will be founded on a spread footing with top of footing at Elevation 283.4. A concrete pad will be placed along the irregular bedrock surface to raise the level to the bottom of the footing elevation.

c) **West Pier**
Bedrock is exposed at about Elevation 278 to 280 at the northern limit of the west pier location. The borehole in the adjacent northern portion of the pier area proved bedrock at Elevation 274.3.

d) **East Pier**
Bedrock is exposed at about Elevation 274 to 275 over the southern portion of the east pier location. The borehole in that area proved bedrock at Elevation 275.2. Outside of irregular rock surface in the foundation area, there should be no unusual problems with the pier foundation.

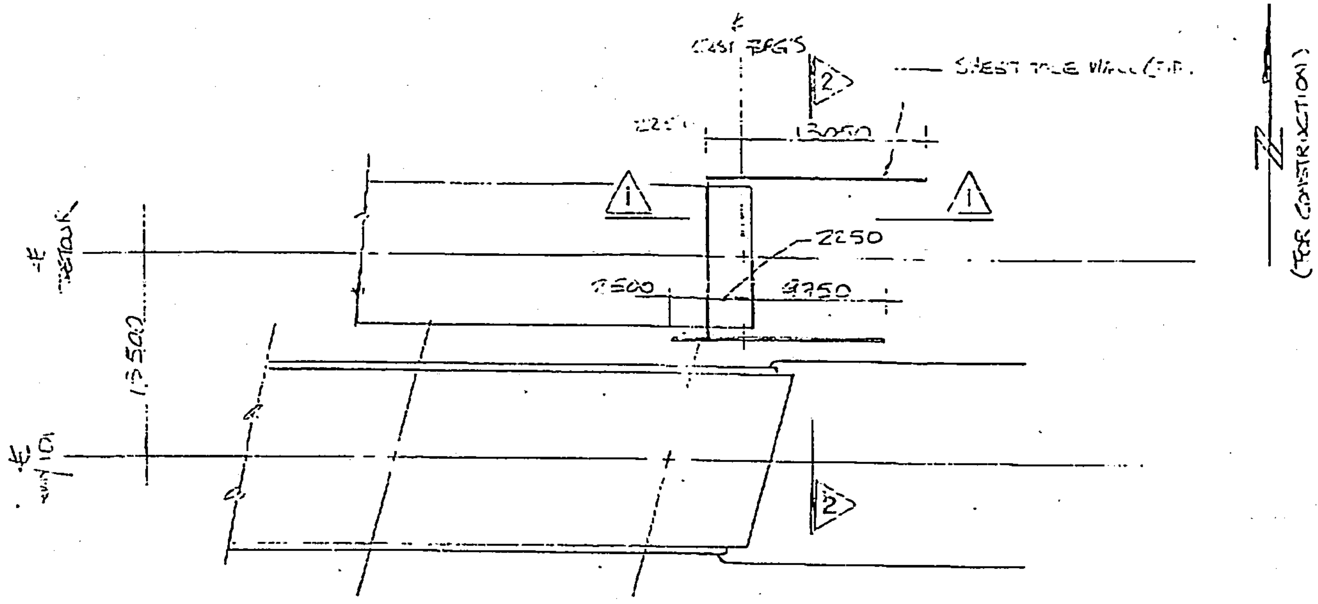
If you have any questions please call me. I will be back from a meeting between 3:30 p.m. and 4:00 p.m.

On Tuesday, please call me at the office after you have had a chance to look at all the test pits.

Regards,

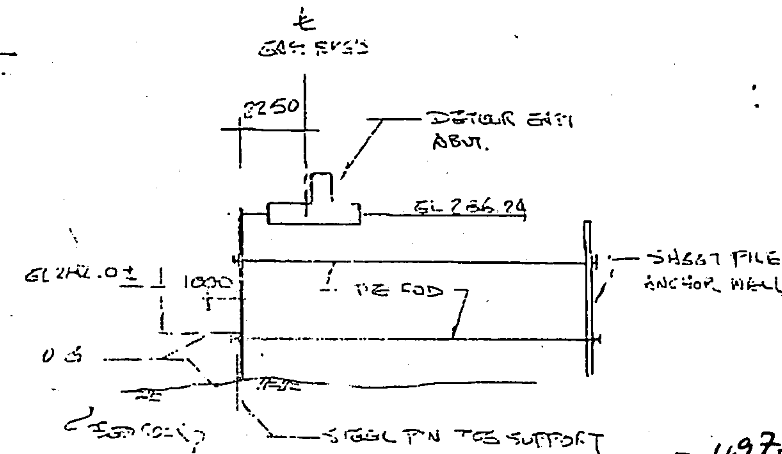
Fin

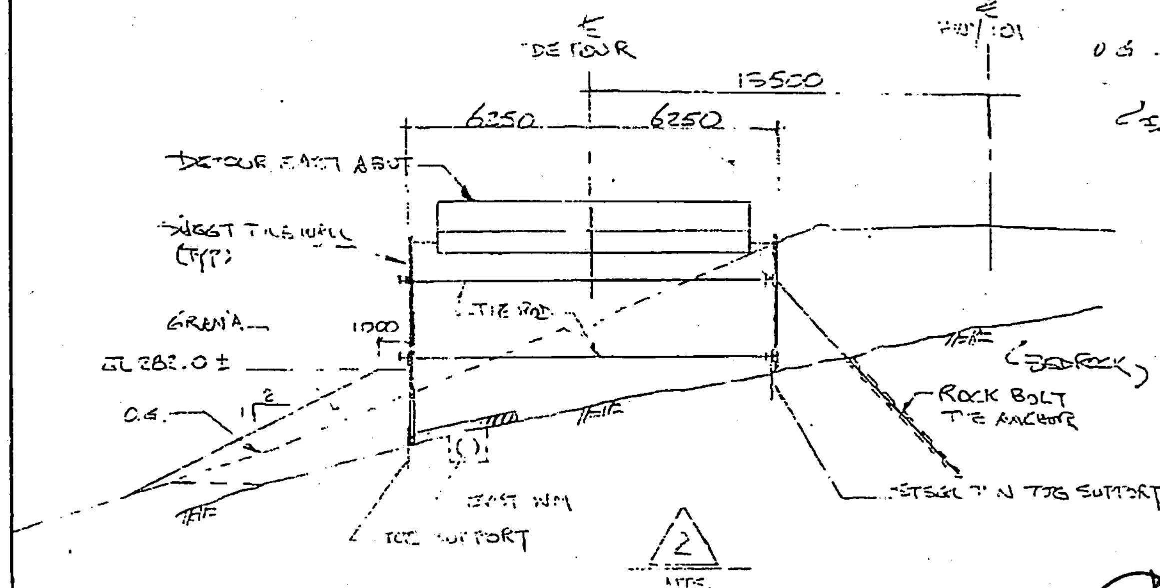
cc: Anne Poschmann



PARTIAL PLAN
NTS

NOTE:
EAST ABUTMENT AS SHOWN, WEST ABUTMENT SIMILAR





1
NTS

NOTES:

1. ALL STEEL SHEET PILES SHALL CONFORM TO THE REQUIREMENTS OF CAN/CSA G40.21 GR 300W
2. ALL STRUCTURAL STEEL SHALL CONFORM TO THE REQUIREMENTS OF CAN/CSA G40.21, GR 350W FOR W SECTIONS, GR 300W FOR REMAINING.
3. ALL TIE RODS SHALL BE DYWIDAG THREADBAR, MIN ULTIMATE TENSILE STRENGTH 1050 MPa

CONSTRUCTION SEQUENCE

1. EXCAVATE AND CONST. WM PROTECTION SLAB
2. INSTALL SOUTH SHEET PILE WALL AND TIE ANCHORS.
3. SUBEXCAVATE AT ABUTMENT LOCATION TO BEDROCK.
4. INSTALL NORTH AND WEST SHEET PILE WALLS, TIE
5. RODS AND ROCK BOLT TIE ANCHORS.
6. BACKFILL TO ELEV. 286.24
7. CONSTRUCT CONCRETE ABUTMENT

416-235-5240.

Attn:
Tae Kim.

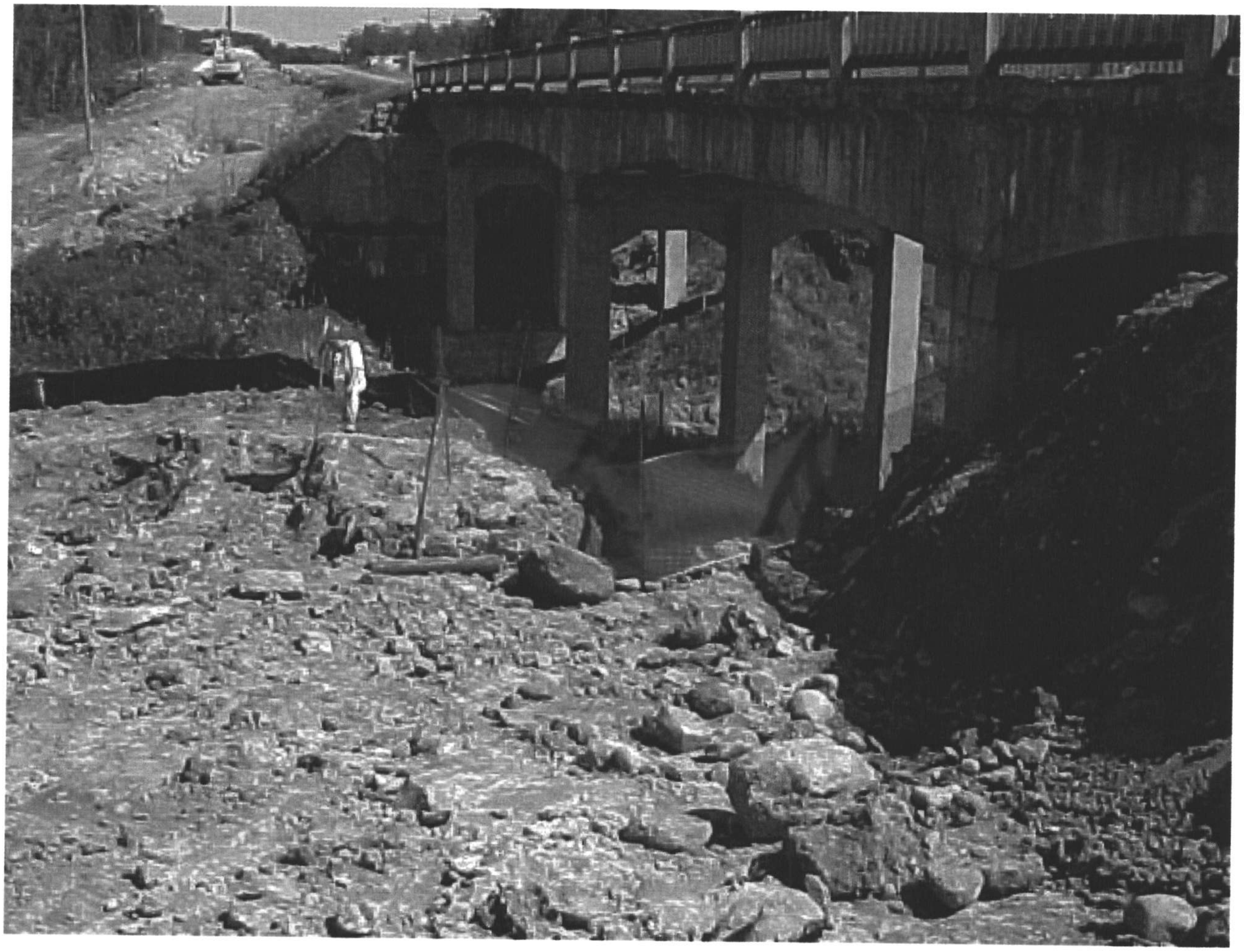
paul - 705-497-1207?
peter - 705-497-5?26

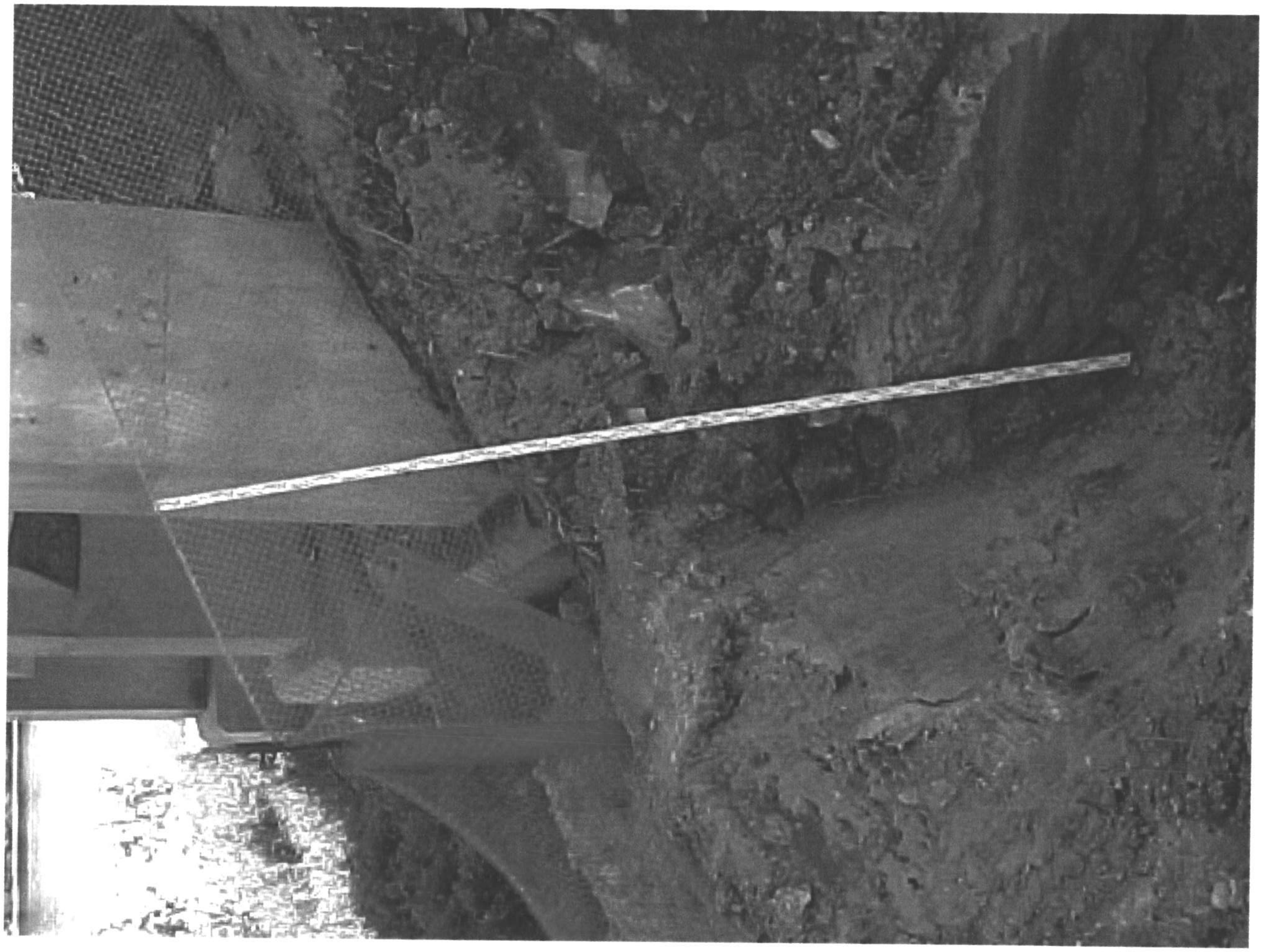
Tae spoken to paul Lecomer on August 8, 2000. 10:30 a.m

(1) Required a detailed design
(2) Not acceptable for Long Term performance

(X) agreed - Not accept.

northland engineering (1987) limited Consulting Engineers and Planners		
CONT. No 2000-0223 WP 315-85-00 PORCUPINE RIVER BRIDGE		
ACROW BRIDGE DETOUR		
PROPOSED SHEET PILE ABUTMENT		
DRAWN BY: TSL	CHECKED BY: SI	DATE: JULY 2000
SCALE: NTS	PROJECT NO. S0-179	DRAWING NO. 1 OF 1





METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

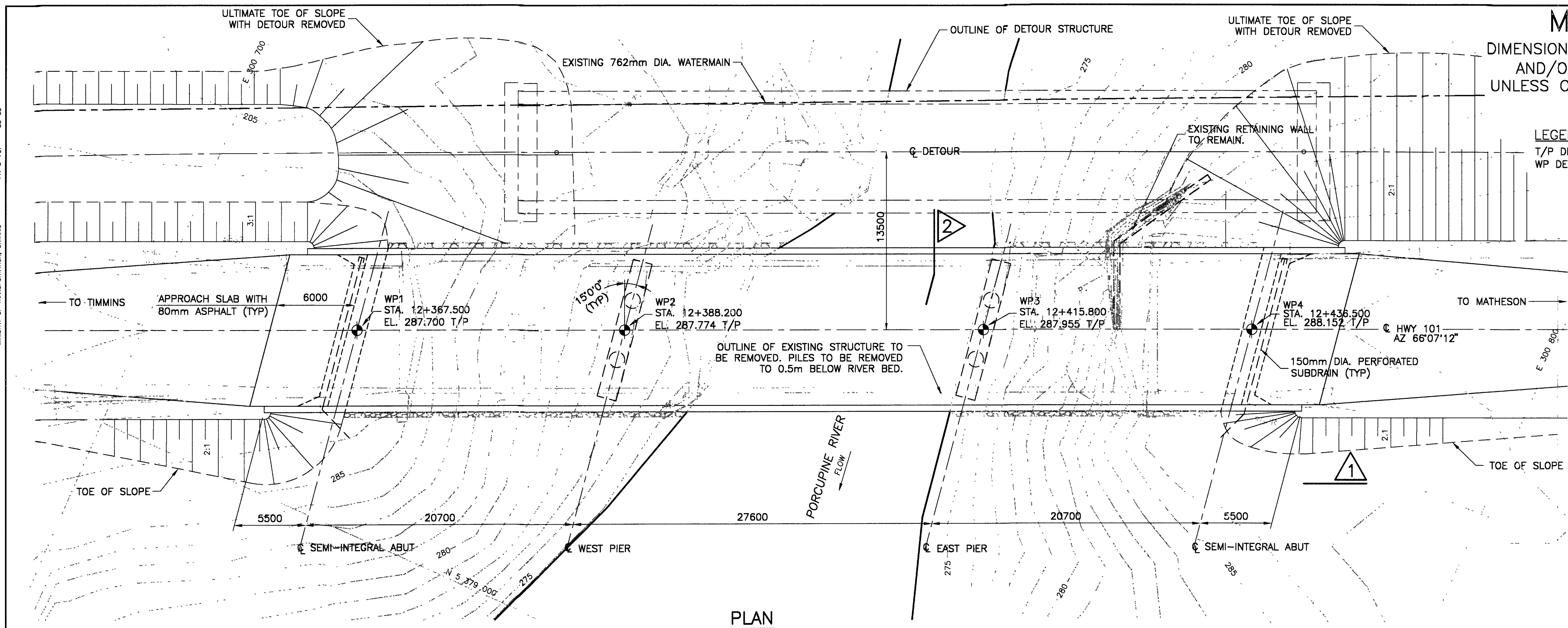
DISTRICT No 53
CONT No
WP No 313-85-00

PORCUPINE RIVER BRIDGE
GENERAL ARRANGEMENT

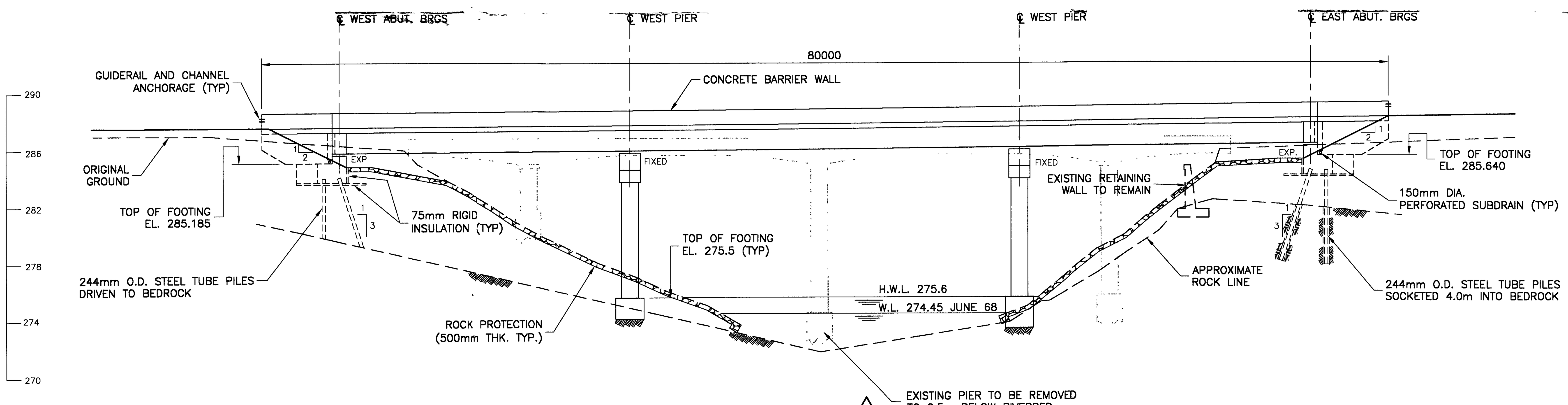
SHEET

McCORMICK RANKIN CORPORATION

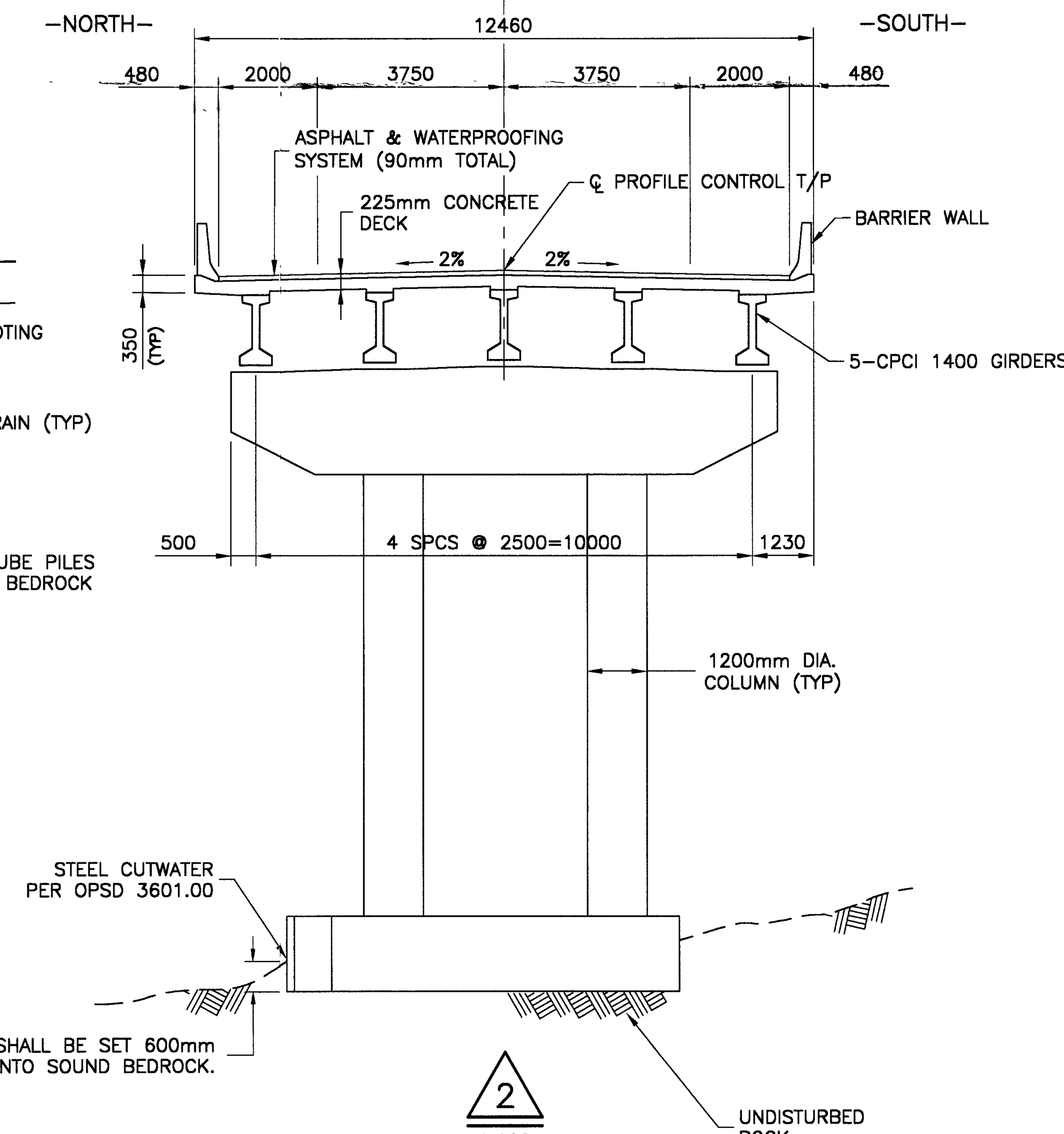
LEGEND
T/P DENOTES TOP OF PAVEMENT
WP DENOTES WORKING POINT



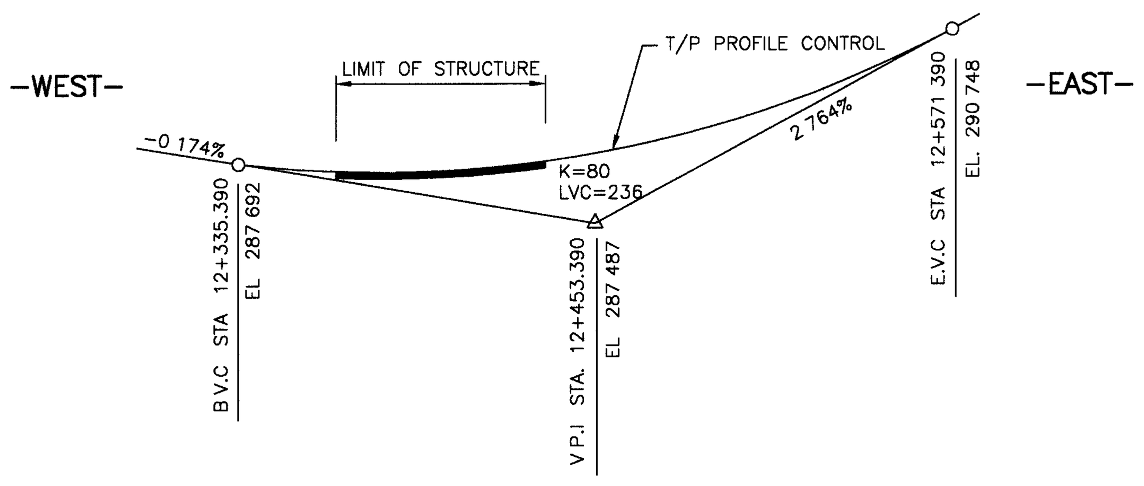
PLAN
1:200



ELEVATION 1
1:200



2
1:100



PROFILE OF HIGHWAY 101
N.T.S

GENERAL NOTES

1. CLASS OF CONCRETE

CLASS OF CONCRETE SHALL BE 30 MPa.

2. CLEAR COVER TO REINFORCING STEEL

FOOTINGS . . . . 100 ±25

DECK TOP . . . . 70 ±20
BOTTOM . . . . 40 ±10

REMAINDER – UNLESS OTHERWISE NOTED. . . . . . . .70 ±20

3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400, UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH PREFIX S WILL BE STAINLESS STEEL. STAINLESS STEEL REBAR SHALL BE TYPE 316LN OR DUPLEX 2205 WITH A MINIMUM YIELD STRESS OF 420MPa.

4. CONSTRUCTION NOTES:

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

GBM
DHO BM 229-71
292.016
TABLET SET VERTICALLY IN TOP OF ROCK
12+586.164 21.687 RT

WP No.	NORTHING	EASTING
1	5379008.635	300715.114
2	5379017.014	300734.041
3	5379028.187	300759.278
4	5379036.567	300778.207

REGISTERED PROFESSIONAL ENGINEER A. S. WING 99/9/21 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER M. VACHON 99-09-20 PROVINCE OF ONTARIO

APPLICABLE STANDARD DRAWINGS

OPSD-4010.00 GUIDERAIL AND CHANNEL ANCHORAGE
OPSD-4601.000 LOCATION OF SITE NUMBER AND DATE FIGURES
OPSD-4670.000 TYPICAL JOINT DETAILS
SS16-21 ROCK SLOPE PROTECTION WITHOUT BERM
SS5-1 GRANULAR BACKFILL REQUIREMENTS (MOD.)

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN ASW	CHK MV	CODE OHBDC 91	LOAD OHBD	DATE SEPT. 99			
DRAWN SC/GG	CHK MV	SITE 39E-083	STRUCT	SCHEME	DWG P1		

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05

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MODIFIED: 99/09/20 11:40:53 Revised