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**Golder Associates Ltd.**

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

REPORT ON

# FOUNDATION INVESTIGATION AND DESIGN
# INDIAN RIVER BRIDGE WIDENING
## HIGHWAY 7
## PETERBOROUGH, ONTARIO
## W.P. 355-97-00

Submitted to:

The Greer Galloway Group Inc.
973 Crawford Drive
Peterborough, Ontario
K9J 3X1

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# PART A

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### PART A - FIELD INVESTIGATION

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by The Greer Galloway Group Inc. (Greer Galloway) to carry out a foundation investigation for the widening of the Indian River Bridge located on Highway 7 east of Peterborough, Ontario. The existing bridge is a two span structure and will be widened to accommodate the required traffic lane width and a pedestrian walkway.

The purpose of the investigation is to determine the subsurface conditions at the location of the foundation units of the proposed widening and at the approach embankments by drilling boreholes and performing laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of foundation design and construction are provided.

The plan and profile of the proposed Indian River Bridge widening on Highway 7 were provided to Golder by Greer Galloway. The terms of reference for the scope of work are outlined in Golder's proposal P91-1377, dated October 14, 1999.

## 2.0 SITE DESCRIPTION

The site is located on Highway 7 at the Indian River crossing, and is situated approximately 15 km east of Peterborough, Ontario in the Township of Otonabee (see Figure 1). Indian River flows approximately north-south within a relatively wide shallow floodplain.

The existing Highway 7 embankments approaching the bridge are about 2 m to 3 m in height above the natural ground surface. The embankment side slopes are at about 2 horizontal to 1 vertical and are grass covered. The south side of the east approach embankment appears to have been built up by some extent of infilling of the river channel. The river is relatively shallow at the bridge; generally less than 1 m in water depth.

The existing bridge is a two span rigid frame structure originally constructed in 1933 / 1934 and rehabilitated in 1984. The spans are 15.24 m long and the bridge is about 10 m wide. The existing bridge abutments are located immediately adjacent to the edges of the river.

The top of asphalt on the existing bridge deck is at about Elevation 204.5 m. The river bed level is at about Elevation 201.1 m and the water level in the river was at about Elevation 201.4 m at the time of this investigation. The 100 year storm high water level is at about Elevation 202.7 m.

## 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried between November 18 and November 24, 1999 at which time, seven boreholes were advanced at the site. The table below summarizes the borehole locations and depths. The bedrock surface and condition was proved in Boreholes 99-3, 99-4 and 99-5 by rock coring. The investigated locations are shown in plan on Figure 2.

**Summary of Boreholes**

| *Borehole No.* | *Location* | *Ground Surface Elevation (m)* | *Borehole Depth (m)* |
|---|---|---|---|
| 99-1 | west approach | 204.53 | 5.1 |
| 99-2 | existing west abutment | 204.39 | 4.8 |
| 99-3 | proposed west abutment | 202.08 | 6.5 |
| 99-4 | pier | 201.08 | 5.5 |
| 99-5 | proposed east abutment | 202.17 | 5.8 |
| 99-6 | existing east abutment | 204.39 | 3.6 |
| 99-7 | east approach | 204.35 | 4.0 |

The boreholes were drilled and sampled using portable drilling equipment supplied and operated by Sonic Soils of Concord, Ontario. Sampling of the overburden material was carried out with a Dynamic Ram Sounder drilling apparatus using a 50 mm outside diameter split spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Bedrock coring in BQ size was carried out using a Prospector 89 diamond drill with BQ size casing. The overburden was sampled in each borehole, except Borehole 99-4, until split spoon sampler refusal. In Boreholes 99-3 and 99-5, sampling continued from the refused depth with the coring equipment. In Borehole 99-4, casing was advanced without sampling to a depth of 2.4 m, and rock core was obtained below this depth. Bedrock was cored to depths ranging from about 5.5 m to 6.5 m from the existing ground surface (Elevations 195.4 m to 195.6 m).

The water levels in the open boreholes were observed during drilling and a piezometer was installed in Borehole 99-2 to permit monitoring of the groundwater level adjacent to the river. A water level in the piezometer was obtained on November 23, 1999 to determine stabilized level at that time.

The field work was supervised throughout by a member of our engineering staff, who cleared the site of underground utilities, located the boreholes in the field, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in containers, labelled and transported to our Mississauga laboratory for further examination and testing. Laboratory testing on selected samples included natural water content determinations, grain size analyses, organic content and point load tests. The results of laboratory testing are given on the Record of Borehole and Record of Drillhole sheets and on Figures 1 to 4.

The ground surface elevations at the borehole locations and a borehole plan showing northing / easting coordinates were provided to Golder by Greer Galloway.

## 4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Site Geology

From published geological information, the site is located in the physiographic region known as the Peterborough Drumlin Field. This region is characterized by a rolling till plain in which the overburden consists of variable clayey to sandy till deposits generally underlain by limestone bedrock of the Trenton Formation. The region is also characterized by its numerous eskers – ridges comprised mainly of gravels. Due to the rolling terrain, bedrock is at variable depths but generally between about Elevation 180 m in the Hastings County area rising to about Elevation 245 m in the Simcoe County area.

The Trenton limestone is a relatively soft and more interbedded bedrock than other more massive limestones (such as the Black River Formation) and is also known to be highly fossiliferous in some areas.

### 4.2 Site Stratigraphy

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Record of Borehole and Record of Drillhole sheets and by the laboratory test results. It should be noted that the stratigraphic boundaries indicated on the borehole records are inferred from sampling, observations of drilling progress and results of Standard Penetration Tests (SPTs). These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The ground surface elevation at the borehole locations varies from about Elevation 201.1 m (river bed) to 204.5 m (top of existing embankment). The boreholes were sampled to depths ranging from 3.6 m to 6.5 m.

In general, the existing embankment fill consists of loose to compact brown sandy gravel containing silty clay / clayey silt pockets at some locations. The native soils encountered at the site generally consist of about 0.6 m of peat underlain in the west by deposits of silty sands and gravels, and underlain in the east by a sandy silt / silt till deposit. In Boreholes 99-5 and 99-6 (east abutment), the peat was mixed with silty clay / clayey silt. The sandy silt / silt till deposit was also encountered on the west side of the river, below the silty sands and gravels. Calcareous shale / limestone bedrock was encountered at about 2.4 m below the existing river bed. The following is a brief summary of the fill and native material encountered at the borehole locations.

### 4.2.1 Embankment Fill

The existing embankment fill material was encountered in Boreholes 99-1, 99-2, 99-5, 99-6 and 99-7 and consists mainly of brown sandy gravel containing occasional cobbles. Pockets of silty clay and clayey silt were encountered within the sandy gravel in Boreholes 99-1 and 99-7. Rootlets and organics were noted within the fill in Borehole 99-5 and within the top 0.3 m in Borehole 99-6. The state of packing of the fill is very loose to compact as indicated by measured SPT 'N' values ranging' from 2 blows to 25 blows per 0.3 m of penetration. Measured water contents of samples from the fill range from 1 percent to 10 percent (average 4 percent). Grain size distribution curves of samples from the fill are shown on Figure 2.

### 4.2.2 Peat and Peat / Clay

A deposit of dark brown to black fibrous peat, 0.6 m to 0.8 m thick, was encountered below the embankment fill in Boreholes 99-1, 99-5, 99-6 and 99-7, and at the ground surface in Borehole 99-3. The peat was mixed with grey silty clay / clayey silt in Boreholes 99-5 and 99-6. Measured SPT 'N' values in the peat and peat / clay mixture ranged from 1 blow to 6 blows per 0.3 m of penetration indicating a soft to firm consistency.

Measured water contents of samples from this deposit ranged from 19 percent to 134 percent. Organic contents of two samples were measured to be about 3 percent (peat / clay in Borehole 99-5) and 18 percent (peat in Borehole 99-1). Lower water contents were measured on the peat samples which were retrieved from below the existing embankment versus the near existing surface sample obtained from Borehole 99-3. In addition, lower average water contents were measured from the peat clay mixtures versus the peat samples (26 percent for peat / clay versus 95 percent for peat samples).

### 4.2.3 Sand / Silty Sand with Gravel

Below the peat in Boreholes 99-1 and 99-3, and below the embankment fill in Borehole 99-2, a non-cohesive deposit ranging in composition from silty sand with gravel to sand containing some silt and trace gravel was encountered. Trace organics, wood fragments and shells were contained within most samples from this deposit. Based on the gradations of samples tested, it is considered that this deposit is derived from the underlying native sandy silt / silt till deposit (see below) that has been reworked due to river action. Measured SPT 'N' values ranged from 15 blows to 23 blows per 0.3 m of penetration indicating a compact state of packing.

Measured water contents of samples from this deposit ranged from 8 percent to 17 percent (average 13 percent). Borehole 99-2 was terminated in this deposit (split spoon sampler refusal). Grain size distribution curves of samples from this deposit are shown on Figure 3.

### 4.2.4 Sandy Silt to Silt Till

A till deposit ranging in composition from sandy silt with gravel to silt containing some sand and trace gravel was encountered below the sand / silty sand with gravel deposit in Boreholes 99-1 and 99-3, and below the peat in Boreholes 99-5, 99-6 and 99-7. Trace fragments of shale were encountered within the till in Borehole 99-3. The top of this deposit ranges from about Elevations 199.6 m to 201.3 m, and was encountered at higher elevations on the east side of the river (Boreholes 99-5, 99-6 and 99-7). The state of packing of the till is dense to very dense as indicated by measured SPT 'N' values ranging from 30 blows to greater than 70 blows per 0.3 m of penetration.

Measured water contents of samples from the till ranged from 4 percent to 11 percent (average 7 percent). Grain size distribution curves of two samples from this deposit are shown on Figure 4. Boreholes 99-1, 99-6 and 99-7 were terminated in this deposit (split spoon sampler refusal).

### 4.2.5 Bedrock

In Boreholes 99-3, 99-4 and 99-5, the depth and condition of bedrock was proved by coring. The surface of the bedrock in these boreholes ranged from about Elevations 198.6 m to 199.4 m. In Boreholes 99-2 and 99-6, sampling was terminated due to refusal to advance the split spoon sampler (likely within the very dense till deposit encountered near the bedrock surface). Considering the bedrock surface elevation encountered in Boreholes 99-3, 99-4 and 99-5, and the refusal depths in Boreholes 99-2 and 99-6, the bedrock surface appears to relatively flat, increasing slightly in elevation towards the east.

The bedrock consists mainly of grey calcareous shale with interlayers of limestone within the upper 1.5 m to 2.0 m. Below this depth, the bedrock mainly consists of limestone with interlayers of calcareous shale.

The Rock Quality Designation (RQD) ranged from about 8 percent to 71 percent and was observed to increase with depth. The RQD values measured on the calcareous shale samples generally ranged from 8 percent to 45 percent indicating very poor to poor quality rock mass. The RQD values measured on the limestone samples generally ranged from 50 percent to 70 percent indicating fair quality rock mass. The state of weathering of the bedrock is described as faintly weathered in the upper 0.75 m to 1.5 m of the bedrock (between about Elevation 197.7 m and 197.9 m), and is fresh below this depth.

Diametral point load tests were carried out on samples of the calcareous shale and limestone interlayers. Measured $I_{S_{50}}$ indices for the calcareous shale ranged from about 1.6 to 4.1, with an average of 2.9. These correspond to unconfined compressive strengths ranging from about 35 MPa to 90 MPa, with an average of 65 MPa which classifies the rock as medium to medium strong; however, the upper fractured portion of the rock can be classified as very weak to weak. Measured $I_{S_{50}}$ indices for the limestone ranged from about 3.8 to 4.4, with an average of 4.2. These correspond to unconfined compressive strengths ranging from about 85 MPa to 100 MPa, with an average of about 95 MPa which classifies the limestone as strong.

## 4.3 Groundwater Conditions

At the time of this investigation, the river level was at about Elevation 201.4 m. The ground water levels in open Boreholes 99-1 and 99-7 were measured at Elevation 201.1 m and 202.3 m, respectively, upon completion of drilling. The water level in the piezometer installed in Borehole 99-2 was measured at Elevation 201.8 m on November 23, 1999. Water levels in open Boreholes 99-3 and 99-5 were not measured due to water use during rock coring. Borehole 99-6 caved at Elevation 203.7 m after drilling.

The groundwater table will be governed by the river water level and groundwater levels are expected to fluctuate due to seasonal and river water level fluctuations.

**GOLDER ASSOCIATES LTD.**

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

Anne S. Poschmann, P.Eng.
Principal

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.
Designated MTO Contact

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# PART B

# FOUNDATION DESIGN
# INDIAN RIVER BRIDGE WIDENING
# HIGHWAY 7
# PETERBOROUGH, ONTARIO
# W.P. 355-97-00

## 5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the geotechnical aspects of the design of the proposed Indian River bridge widening based on our interpretation of the factual geotechnical data obtained during the investigation. The recommendations provided are intended for the guidance of the design engineer only. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

The works described in this report are associated with the proposed bridge widening and the approach embankments within 20 m of the structure. The proposed construction consists of widening the existing Indian River Bridge by 1.5 m along the north side to accommodate a pedestrian walkway, and at least 3.0 m to the south to accommodate the required travel width. The proposed construction will require the existing foundation units and embankments to be extended to the south by up to 5.7 m.

### 5.1 Bridge Foundations

The available information (Planning and Design Report) indicates that the existing Indian River Bridge structure is supported on spread footings founded on the bedrock surface. The proposed widening should also be supported on the bedrock surface. The overburden materials overlying the bedrock generally consists of silty sand with gravel grading to sandy silt till. The groundwater level is at about 2 m above the bedrock surface and excavations for foundations extended to the bedrock will have to deal with groundwater inflow.

The foundation alternatives considered feasible for this site are spread footings placed on the bedrock surface (as the existing bridge footings) and caissons extended to the bedrock surface. These two options are discussed below.

The following table summarizes the boreholes advanced as part of this investigation and indicates the bedrock surface as encountered in the boreholes where bedrock coring was completed.

**Summary of Boreholes**

| *Borehole No.* | *Location* | *Ground Surface Elevation (m)* | *Bedrock Surface Elevation (m)* |
|---|---|---|---|
| 1 | west approach | 204.53 | - |
| 2 | existing west abutment | 204.39 | - |
| 3 | proposed west abutment | 202.08 | 198.8 |
| 4 | pier | 201.08 | 198.6 |
| 5 | proposed east abutment | 202.17 | 199.4 |
| 6 | existing east abutment | 204.39 | - |
| 7 | east approach | 204.35 | - |

### 5.1.1 Shallow Spread Footings

Based on the details shown on the contract drawing for the original bridge, the existing footing has a design founding level of Elevation 199.7 m for the abutments and the pier. The results of the boreholes advanced as part of the current investigation indicate that bedrock is at a relatively shallow depth (between about 2.4 m and 3.3 m below the existing ground surface) although the bedrock surface is at a somewhat lower elevation than the design founding level for the existing structure. Based on this information, shallow spread footings would be suitable to support the widened bridge. As indicated above, excavations for spread footing construction will have to include some form of groundwater control.

Shallow spread footings founded on the bedrock surface may be designed for a factored bearing resistance at Ultimate Limit States (ULS) of 2,000 kPa. Serviceability Limit State (SLS) conditions do not apply to footings founded on bedrock at this site. This value of bearing resistance is for vertical concentric applied loading only; effects of possible load inclinations and eccentricity need to be taken into account as per OHBDC using the curve for cohesive soils.

For design, the following foundation elevations may be assumed:

| | |
|---|---|
| West Abutment | 198.8 m |
| Pier | 198.6 m |
| East Abutment | 199.4 m |

The above founding elevations are based on the borehole information and must be confirmed in the field. The contract should accommodate some variability in the bedrock surface and allow for removal of bedrock to the design level if there areas where the surface is higher. In addition, the contract should allow for mass concrete placement to raise the grade as may be required. All footing excavations should be inspected by qualified geotechnical personnel to ensure that the bedrock has been reached and that the base has been cleaned and that the bedrock conditions as exposed are consistent with the design assumptions.

Resistance to lateral forces / sliding resistance between the concrete footings and the bedrock should be calculated between the concrete Section 6.8.4.3 of the OHBDC assuming an unfactored angle of friction of 35 degrees. If necessary, sliding resistance can be supplemented by dowelling into the bedrock.

A value of 300 kPa may be assumed for the grout-to-rock bond stress for ULS design. This value refers to the rock-grout interface and can be used for tension design. The dowels should have a minimum of 1.0 m embedded length in the rock and the structural strength of the dowel and the compressive strength of the grout should not be exceeded.

The actual bond stress along the rock-grout interface may vary from the typical design value given and should therefore be verified in the field. Verification should be carried out on at least one dowel per foundation unit up to at least 10 percent of the total number of dowels at the site. The testing should be carried out on the first dowel installed at a site and the dowel should be tested to 125 percent of the maximum design load on the dowels. The test dowels must have a threaded length of 150 mm exposed in order to complete the testing.

#### 5.1.1.1 Frost Protection

For spread footings founded on the calcareous shale bedrock at this site, soil cover of at least 1.5 m should be provided to the footings for frost protection.

### 5.1.2 Caissons

As an alternative to spread footings, foundations for the abutments and piers may be supported on caissons. The caisson (drilled pier) could be formed using a permanent heavy duty steel liner, capable of being driven, vibrated or oscillated through the overburden, and advanced to and keyed into the bedrock surface. The liner should be fitted with a reinforcing shoe that will allow it to be driven or rotated to seat within the bedrock.

Given the measured groundwater levels at the site, groundwater flow through the bedrock at the founding level into the caisson excavation from below should be anticipated. In addition, groundwater flow will occur at the liner / bedrock interface if a suitable seat into the bedrock is not achieved. Therefore, concrete should be placed using tremie techniques, maintaining the tremie tube below the concrete surface at all times.

It may be feasible, however, to dewater the caissons to allow cleaning and inspection prior to concrete placement. In this case, the caisson excavation should be thoroughly cleaned using bailing methods and all loose material should be removed from the bottom of the caisson before concreting to ensure intimate contact of the concrete with the bedrock surface. If the caisson excavation cannot be fully dewatered to permit inspection, the excavation should be cleaned using a water jet to remove all loose material from the sides and base. Qualified geotechnical personnel should inspect the caisson installation to ensure that the bailing methods remove all debris and that the tremie concrete is properly placed.

Due to the shallow bedrock depth encountered at the site (about 3 m or so below the existing ground surface at the toe of the existing embankment), caissons installed to this depth should be designed for end-bearing only. For design, a factored capacity at ULS for end bearing resistance of concrete caissons founded on the bedrock surface of 3,500 kPa may be assumed. SLS conditions do not apply to end-bearing caissons founded on bedrock at this site.

## 5.2 Foundation Excavations

Excavations for construction of the spread footings will extend through the embankment fill material, and native peat, sands and gravels, and till deposits. The groundwater level observed upon completion of drilling is in the order of 2 m above the bedrock surface. Therefore, difficulties in the form of caving of material and groundwater flow into the excavations extended to the bedrock surface are likely. Some form of dewatering / ground water cutoff will be required to achieve the required excavation depths and permit footing construction.

A cut-off to groundwater flow into the excavation within the overburden can be achieved with driven steel interlocking (closed) sheet piling. Due to the strength of the bedrock, however, there is a potential for the sheet piles to meet practical refusal during driving without forming an effective cut-off to groundwater flow under the sheet piles at the bedrock interface. In addition, fractures within the bedrock may provide a path for groundwater flow up through the base of the excavation. Therefore, some groundwater flow into the excavation may occur from below the sheet piling and possibly up through the bedrock. It is considered that the groundwater inflow through these paths can be controlled by properly located and filtered sump pumps.

Consideration could be given to constructing the footings within open cut excavations, if space permits, and providing groundwater control with pumping from sumps at the base. A temporary support system will be required to support the existing road embankment. A soldier pile and lagging wall with raker support or rock anchors may be feasible at this site. If additional passive restraint is required at the toe of the wall, the soldier piles would either have to be socketted into the rock or rock anchors / bolts provided at the toe of the piles. Socketting of the piles into the bedrock at this site will have to deal with excavation through the limestone interlayers which are present within the upper portion of the bedrock and the limestone that underlies the shale.

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
$\gamma$ = soil unit weight = 21 $kN/m^3$

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
$\gamma$ = soil unit weight = 21 $kN/m^3$

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.

Temporary open cut slopes should not be formed steeper than 2.5 horizontal to 1 vertical. Groundwater flow through the overburden will occur and measures must be taken to deal with sloughing of the side slopes.

Available information indicates that the spread footings for the existing bridge foundation units are founded on the bedrock surface; however, the design drawings indicate a higher founding level than the design founding levels given in this report. As long as the existing footings are actually placed on the bedrock, undermining of the existing foundations should not be a concern during excavation for the new footings. However, careful inspection of the existing foundations should be carried out to confirm the actual founding condition as the excavation for the new footings advance. If the footings are in fact founded above the bedrock surface on the silty sands and sandy silts, measures should be taken to prevent undermining of the existing spread footings since the sands and silts are subject disturbance. In addition, if it is found that the existing footings are on bedrock but at a higher elevation than the current design, careful excavation of the bedrock (fractured, calcareous shale) will have to be ensured.

### 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 and the retaining walls in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B, Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the abutments and walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent 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 1.6 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 existing and new embankment fill materials and the following parameters (unfactored) may be assumed (based on the use of fill materials meeting the specifications for Select Subgrade Material):

| | |
|---|---|
| Soil unit weight<br>(assuming clean earth fill) | 21 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 Embankments

The results of the boreholes put down near the shoulder of the road indicate that at least a portion of the existing embankment was constructed directly on the native peat deposit. The boreholes are located just outside the paved width, so it is not possible to confirm whether the peat was removed from under the travelled portion of the road. Considering that the existing embankment was constructed about 30 years ago, and given the thickness and nature of the peat deposit, it is probable that whatever peat was left in place has undergone a significant amount of the expected settlement (i.e., greater than 90 percent).

Placement of additional fill material will be required on the south side of the existing embankment for the road widening. The increase in loading will induce additional settlement of the peat and peat / clay mixture encountered in the boreholes and there will be differential settlement occurring between the current travelled portion and the proposed widening.

It is recommended that these organic and soft cohesive materials be removed from under the proposed widening to as great an extent as possible prior to placement of additional fill. This can be accomplished by excavating a wedge of material formed by line drawn at 1 horizontal to 1 vertical (1H:1V) extending from the edge of the existing road shoulder to the base of the peat deposit (at about Elevation 201 m). This excavation should also remove the existing vegetative cover along the existing embankment, and should extend along the entire length of the proposed embankment widening.

The excavation for peat removal should be carried out in narrow strips with maximum width of 6 m and backfilling of each strip should be completed before commencing the next strip.

The newly placed embankment fill should be keyed into the existing embankment by benching in accordance with OPSD 208.01. Construction of embankments above the prepared subgrade may be carried out using Select Subgrade Material (in accordance with OPSS 1010). All embankment fill should be placed in regular loose lifts with loose thickness not exceeding 300 mm, and be compacted to 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to the placement of the granular subbase or base course should be compacted to 100 percent of the material's Standard Proctor maximum dry density. Vegetation cover should be established on all slopes to protect the embankment fill against surficial erosion as per OPSS 572.

Provided that the embankment subgrade is properly prepared, placement of the new embankment fill at a slope of 2H:1V would be stable. The embankment loading will result in settlement of the underlying sands and silts, however, the majority of this settlement will occur during construction and is expected to be small. The magnitude of additional long-term settlement of the embankment (following construction) is therefore expected to be less than 25 mm. It should be noted, however, that this settlement of the widened portion of the embankment will be differential with respect to the existing embankment.

## 5.5 Review

Prior to finalizing the design and prior to tendering, the foundation aspects of the drawings and specifications should be reviewed to confirm that the intent of this report has been met.

**GOLDER ASSOCIATES LTD.**

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

Anne S. Poschmann, P.Eng.
Principal

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

For Fintan J. Heffernan, P.Eng.
Designated MTO Contact

ASP/FJH/clg
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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) |
| $\gamma$ | 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

$\pi$ = 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

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. in stress: $\Delta\sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| $\nu$ | Poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma - u$) |
| $\sigma'_{vo}$ | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stresses (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$ | 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 |
| $\gamma'$ | 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 $\rho$. Unit weight symbol is $\gamma$ 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 |
| $C_\alpha$ | 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 |
| $\sigma'_p$ | pre-consolidation pressure |
| OCR | Overconsolidation ratio $= \sigma'_p/\sigma'_{vo}$ |

### (e) Shear Strength

| | |
|---|---|
| $\tau_p, \tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | 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

# 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
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
II - Parallel To
⊥ - Perpendicular To

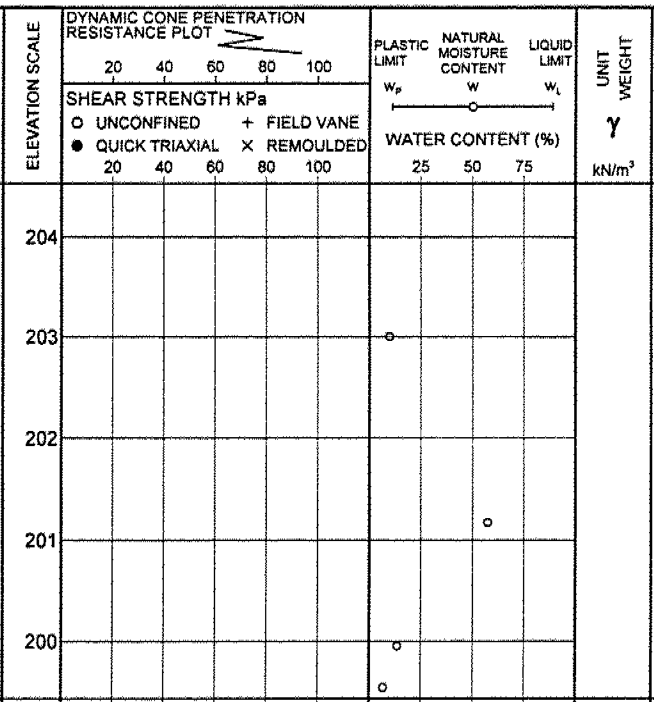
P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

# RECORD OF BOREHOLE No 99-1

1 OF 1 **METRIC**

PROJECT 991-1178
W.P. 355-97-00
DIST
HWY #7
DATUM Geodetic
LOCATION N 4910314.7; E 411868.4
BOREHOLE TYPE Dynamic Ram Sounder
DATE 19.11.99
ORIGINATED BY SB
COMPILED BY BVB
CHECKED BY ASP

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | GR | SA | SI | CL |
| 204.53 | | | | | | | | | |
| 0.00 | Sandy Gravel, with pockets of silty clay between 0.6m and 1.8m depth<br>Loose<br>Brown<br>(Fill) | 1 | 50 DO | 9 | | | | | |
| | | 2 | 50 DO | 8 | | | | | |
| | | 3 | 50 DO | 9 | | 42 | 39 | 17 | 2 |
| | | 4 | 50 DO | 6 | ∇ | | | | |
| 201.48 | | 5 | 50 DO | 4 | | | | | |
| 3.05 | Fibrous Peat, trace shells<br>Firm<br>Black | 6 | 50 DO | 6 | | | | | |
| 200.72<br>3.96 | Sandy Silt<br>Compact<br>Grey | 7 | 50 DO | 21 | | | | | |
| 199.96<br>4.57 | Sand, some silt, trace gravel<br>Compact<br>Grey | 8 | 50 DO | 22 | | 3 | 79 | 17 | 1 |
| 199.43<br>5.10 | Sandy Silt with gravel, trace clay<br>Compact to very dense<br>Moist<br>Grey<br>(Till) | 9 | 50 DO | 50/.08 | | 37 | 24 | 34 | 5 |
| | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in open borehole at Elevation 202.1m upon completion of drilling. | | | | | | | | |



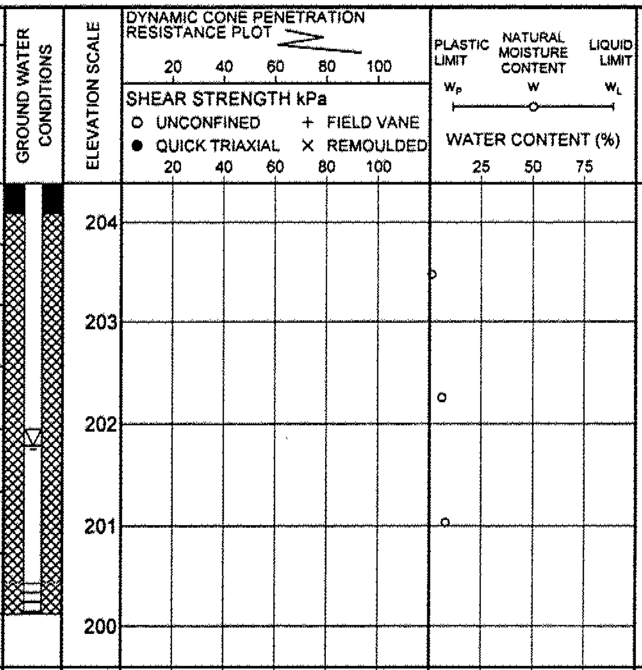
$+^3, \times^3$: Numbers refer to Sensitivity    $\circ^{3\%}$ STRAIN AT FAILURE

# RECORD OF BOREHOLE No 99-2

1 OF 1 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 991-1178 | | | |
| W.P. 355-97-00 | LOCATION N 4910326.0; E 411882.7 | | ORIGINATED BY SB |
| DIST | HWY #7 | BOREHOLE TYPE Dynamic Ram Sounder | COMPILED BY BVB |
| DATUM Geodetic | DATE 19.11.99 | | CHECKED BY ASP |

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| 204.39 | | | | | | | | |
| 0.00 | Sandy Gravel, trace clay, trace rootlets, cobbles<br>Compact<br>Brown<br>(Fill) | 1 | 50 DO | 19 | | | | |
| | | 2 | 50 DO | 10 | | | | |
| | | 3 | 50 DO | 19 | | | | |
| 201.95 | | 4 | 50 DO | 12 | | | | |
| 2.44 | Silty Sand with gravel, trace clay, wood fragments at 4.6m depth.<br>Compact to very dense<br>Grey | 5 | 50 DO | 19 | | | | |
| | | 6 | 50 DO | 21 | 42 | 32 | 23 | 3 |
| | | 7 | 50 DO | 22 | | | | |
| 199.59 | | 8 | 50 DO | 23 | | | | |
| 4.80 | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in piezometer at Elevation 201.79 on Nov.23/99. | | | | | | | |



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

# RECORD OF BOREHOLE No 99-3

1 OF 1

**METRIC**

PROJECT 991-1178

W.P. 355-97-00

LOCATION N 4910321.5; E 411808.4

ORIGINATED BY SB

DIST ______ HWY #7

BOREHOLE TYPE Dynamic Ram Sounder

COMPILED BY BVB

DATUM Geodetic

DATE 19.11.99

CHECKED BY ASP

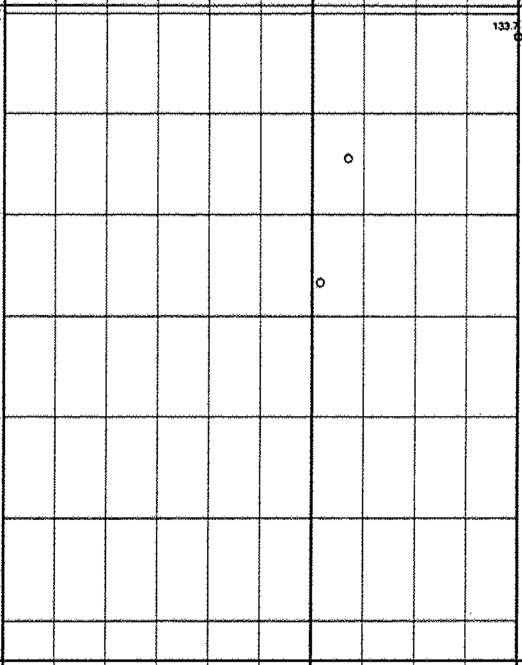
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|
| 202.08 | | | | | | | | |
| 0.00 | Fibrous Peat<br>Soft<br>Black | 1 | 50 DO | 1 | | 202 | | |
| 201.32<br>0.76 | Silty Sand with gravel, trace organics, trace shells<br>Compact<br>Grey | 2 | 50 DO | 25 | | 201 | | |
| | | 3 | 50 DO | 15 | | | | |
| | | 4 | 50 DO | 19 | | 200 | | |
| 199.64<br>2.44 | Sandy Silt, trace gravel, trace clay, shale fragments<br>Very dense<br>Grey<br>(Till) | 5 | 50 DO | 68 | | | | |
| | | 6 | 50 DO | 25/0.1 | | 199 | | |
| 198.82<br>3.26 | Calcareous Shale with limestone interlayers grading to Limestone with calcareous shale interlayers at 5.2m depth<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br><br>Bedrock cored between 3.26m and 6.47m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-3. | | | | | 198<br>197<br>196 | | |
| 195.61<br>6.47 | END OF BOREHOLE | | | | | | | |

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 (%)
25 50 75



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

ON_MOT 991-1178.GPJ ON_MOT.GDT 5/1/00

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-3

SHEET 1 OF 1

LOCATION: N 4910321.5; E 411808.4

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 24, 1999

DRILL RIG: Prospector 89

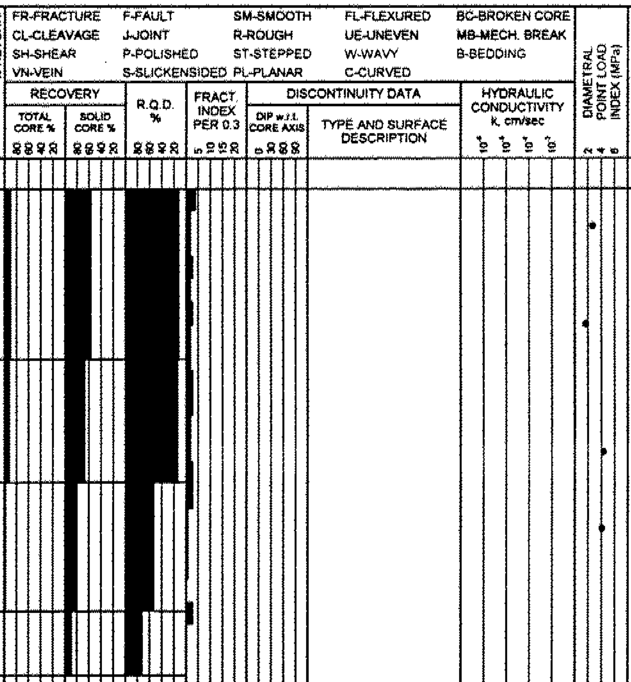
DRILLING CONTRACTOR: Sonic Soils

DATUM: Geodetic

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. |
|---|---|---|---|---|---|
| | | *CONTINUED FROM PREVIOUS PAGE* | | 198.82 | |
| 4<br>5<br>6 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered to fresh<br>Grey<br>Very poor quality<br>Weak to strong<br>(Bedrock) | | 3.26 | 1<br>2 |
| | | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | | 196.88<br>5.20 | 3<br>4 |
| 7<br>8<br>9<br>10<br>11<br>12<br>13 | | END OF HOLE | | 195.61<br>6.47 | |

PENETRATION RATE (m/min) | COLOUR / % RETURN | FLUSH



NOTES
WATER LEVELS
INSTRUMENTATION

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: BVB
CHECKED: ASP

DRILLHOLE 1178ROCK.GPJ GLDR_CAN.GDT 5/1/00 MMZ

# RECORD OF BOREHOLE No 99-4

1 OF 1 **METRIC**

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910334.7; E 411898.9
ORIGINATED BY SB
DIST
HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 24.11.99
CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 <br> SHEAR STRENGTH kPa <br> ○ UNCONFINED + FIELD VANE <br> ● QUICK TRIAXIAL × REMOULDED <br> 20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$ <br> WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | | | | |
| 201.08 | | | | | | | | | | | |
| 0.00 | Unsampled<br>Probable Silty Sand and Gravel | | | | | | 201<br>200<br>199 | | | | |
| 198.64<br>2.44 | Calcareous Shale with limestone interlayers grading to Limestone with calcareous shale interlayers at 4.7m depth<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br>Bedrock cored between 2.44m and 5.49m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-4. | | | | | | 198<br>197<br>196 | | | | |
| 195.59<br>5.49 | END OF BOREHOLE | | | | | | | | | | |

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

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-4

SHEET 1 OF 1

LOCATION: N 4910334.7; E 411898.9

DRILLING DATE: November 23, 1999

DATUM: Geodetic

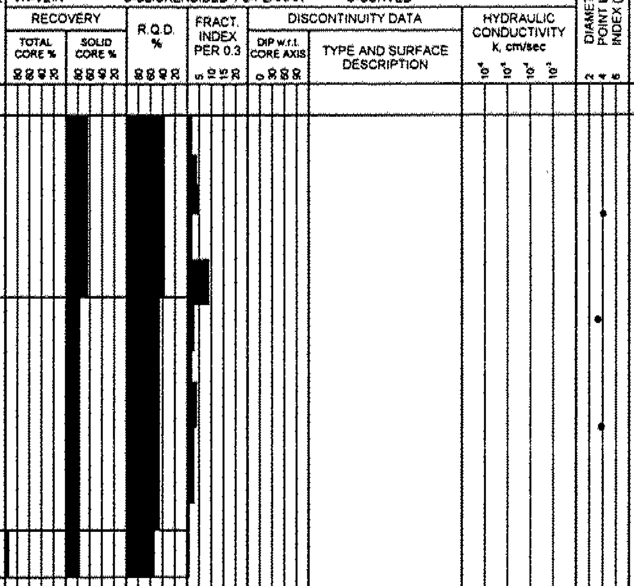
DRILL RIG: Prospector 89

INCLINATION: -90°    AZIMUTH: —

DRILLING CONTRACTOR: Sonic Soils

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | COLOUR / % RETURN / FLUSH | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|
| | | *CONTINUED FROM PREVIOUS PAGE* | | 198.64 | | | | |
| 3 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered<br>Grey<br>Poor quality<br>Weak to strong<br>(Bedrock) | | 2.44<br>197.88 | 1 | | | |
| 4 | | Calcareous Shale, limestone interlayers<br>Fresh<br>Grey<br>Poor quality<br>Weak to strong<br>(Bedrock) | | 3.20<br>196.35 | 2 | | | |
| 5 | | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | | 4.73<br>195.59 | 3 | | | |
| | | END OF HOLE | | 5.49 | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |
| 8 | | | | | | | | |
| 9 | | | | | | | | |
| 10 | | | | | | | | |
| 11 | | | | | | | | |
| 12 | | | | | | | | |



DRILLHOLE 1178ROCK.GPJ GLDR_CAN.GDT 5/1/00 MMZ

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: BVB
CHECKED: ASP

Ministry of Transportation
Ontario

Foundation Design

# RECORD OF BOREHOLE No 99-5

1 OF 1 METRIC

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910342.8; E 411913.5
ORIGINATED BY SB
DIST
HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 18.11.99
CHECKED BY ASP

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$<br>WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | | | | |
| 202.17 | | | | | | | | | | |
| 0.00 | Sand, trace gravel, trace rootlets, trace organics<br>Loose<br>Brown<br>(Fill) | 1 | 50 DO | 2 | | 202 | | | | |
| 201.56<br>0.61 | Silty Clay with sand and gravel, mixed with black fibrous peat and wood fragments<br>Very loose<br>Grey | 2 | 50 DO | 2 | | | | | | |
| 200.95<br>1.22 | Sandy Silt, trace gravel<br>Compact<br>Grey<br>(Till) | 3 | 50 DO | 27 | | 201<br>200 | | | | |
| 199.43<br>2.74 | Calcareous Shale with limestone interlayers becoming Limestone with calcareous shale interlayers<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br>Bedrock cored between 2.74m and 5.79m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-5. | | | | | 199<br>198<br>197 | | | | |
| 196.38<br>5.79 | END OF BOREHOLE<br>Note:<br>Borehole continued below 1.8m depth, after sampler refusal encountered, using casing and wash bore techniques. | | | | | | | | | |

ON_MOT 991-1178.GPJ ON_MOT.GDT 5/1/00

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-5

SHEET 1 OF 1

LOCATION: N 4910342.8; E 411913.5

INCLINATION: -90°    AZIMUTH: —

DRILLING DATE: November 22, 1999

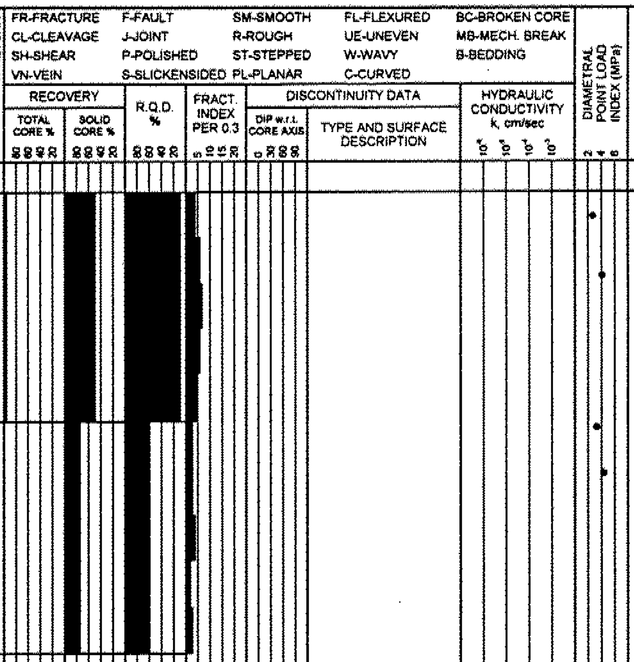
DRILL RIG: Prospector 89

DRILLING CONTRACTOR: Sonic Soils

DATUM: Geodetic

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | COLOUR % RETURN | FLUSH |
|---|---|---|---|---|---|---|---|---|
| | | *CONTINUED FROM PREVIOUS PAGE* | | 199.43 | | | | |
| 3<br>4 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered<br>Grey<br>Very poor quality<br>Weak to strong<br>(Bedrock) | | 2.74<br>197.91 | 1 | | | |
| 5 | | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | | 4.26<br>196.38 | 2 | | | |
| 6<br>7<br>8<br>9<br>10<br>11<br>12 | | END OF HOLE | | 5.79 | | | | |



NOTES
WATER LEVELS
INSTRUMENTATION

DEPTH SCALE
1 : 50

Golder Associates

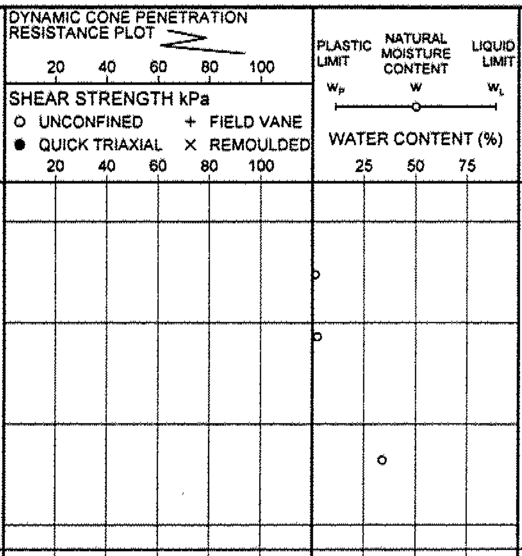
LOGGED: BVB
CHECKED: ASP

# RECORD OF BOREHOLE No 99-6

1 OF 1 **METRIC**

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910345.7; E 411911.9
ORIGINATED BY SB
DIST
HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 18.11.99
CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | GR | SA | SI | CL |
| 204.39 / 0.00 | Sandy gravel, trace organics and rootlets in upper 0.3m, cobbles<br>Compact<br>Brown<br>(Fill) | | 1 | 50 DO | 15 | | 204 | | | | | |
| | | | 2 | 50 DO | 19 | | | | | | | |
| | | | 3 | 50 DO | 15 | | 203 | | 72 | 23 | 4 | 1 |
| | | | 4 | 50 DO | 18 | | 202 | | | | | |
| 201.60 / 2.79 | Fibrous peat, with clayey silt<br>Firm<br>Black/Grey | | 5 | 50 DO | 25 | | | | | | | |
| 201.04 | Sandy Silt, trace gravel<br>Very dense<br>(Till) | | 6 | 50 DO | 27 | | 201 | | | | | |
| 200.76 / 3.63 | END OF BOREHOLE<br>Sampler Refusal<br><br>Borehole caved at 0.7m depth upon completion of drilling. | | | | | | | | | | | |



$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

# RECORD OF BOREHOLE No 99-7

1 OF 1 **METRIC**

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910354.8; E 411928.2
ORIGINATED BY SB
DIST ____ HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 18.11.99
CHECKED BY ASP

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 (%): 25 50 75
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)

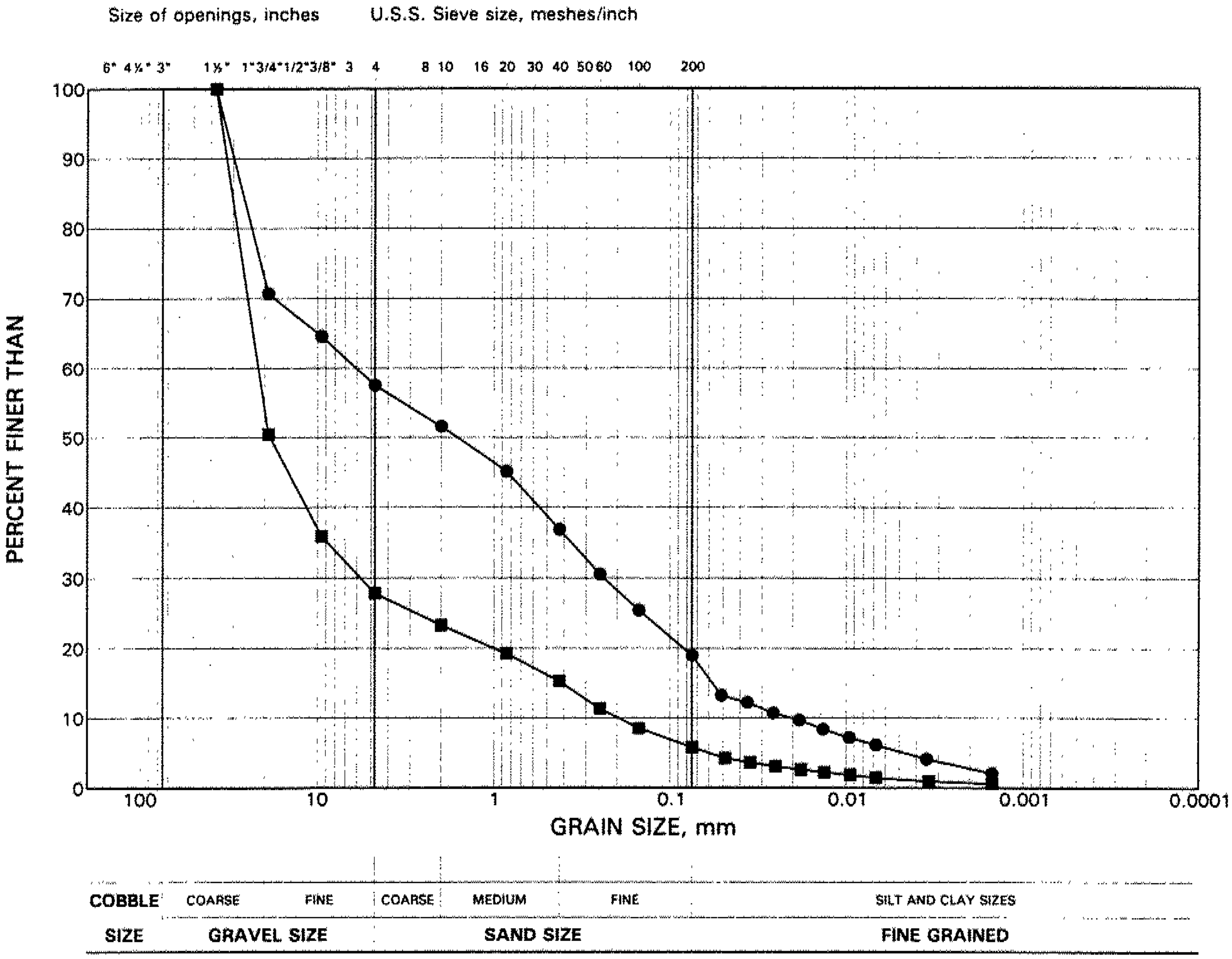
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 204.35 | | | | | | | | | | | |
| 0.00 | Sandy Gravel, with clayey silt pockets<br>Compact to very loose<br>Brown<br>(Fill) | | 1 | 50 DO | 9 | | 204 | | | | |
| | | | 2 | 50 DO | 17 | | | | | | |
| | | | 3 | 50 DO | 12 | | 203 | | | | |
| | | | 4 | 50 DO | 3 | ▽ | | | | | |
| 201.91<br>2.44 | Fibrous Peat with silty clay<br>Soft<br>Dark grey | | 5 | 50 DO | 3 | | 202 | | | | |
| 201.30<br>3.05 | Silt, some sand, trace gravel, trace clay<br>Dense to very dense<br>Grey<br>(Till) | | 6 | 50 DO | 30 | | 201 | | | | |
| 200.39 | | | 7 | 50 DO | 70/.08 | | | 3 | 16 | 76 | 5 |
| 3.96 | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in open borehole at Elevation 202.3m upon completion of drilling. | | | | | | | | | | |

ON_MOT 991-1178.GPJ ON_MOT.GDT 5/1/00

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

# GRAIN SIZE DISTRIBUTION
## Sandy gravel trace to some silt (Fill)

FIGURE 2



LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 99-1 | 3 | 1.8 |
| ■ | 99-6 | 3 | 1.8 |

Project 991-1178

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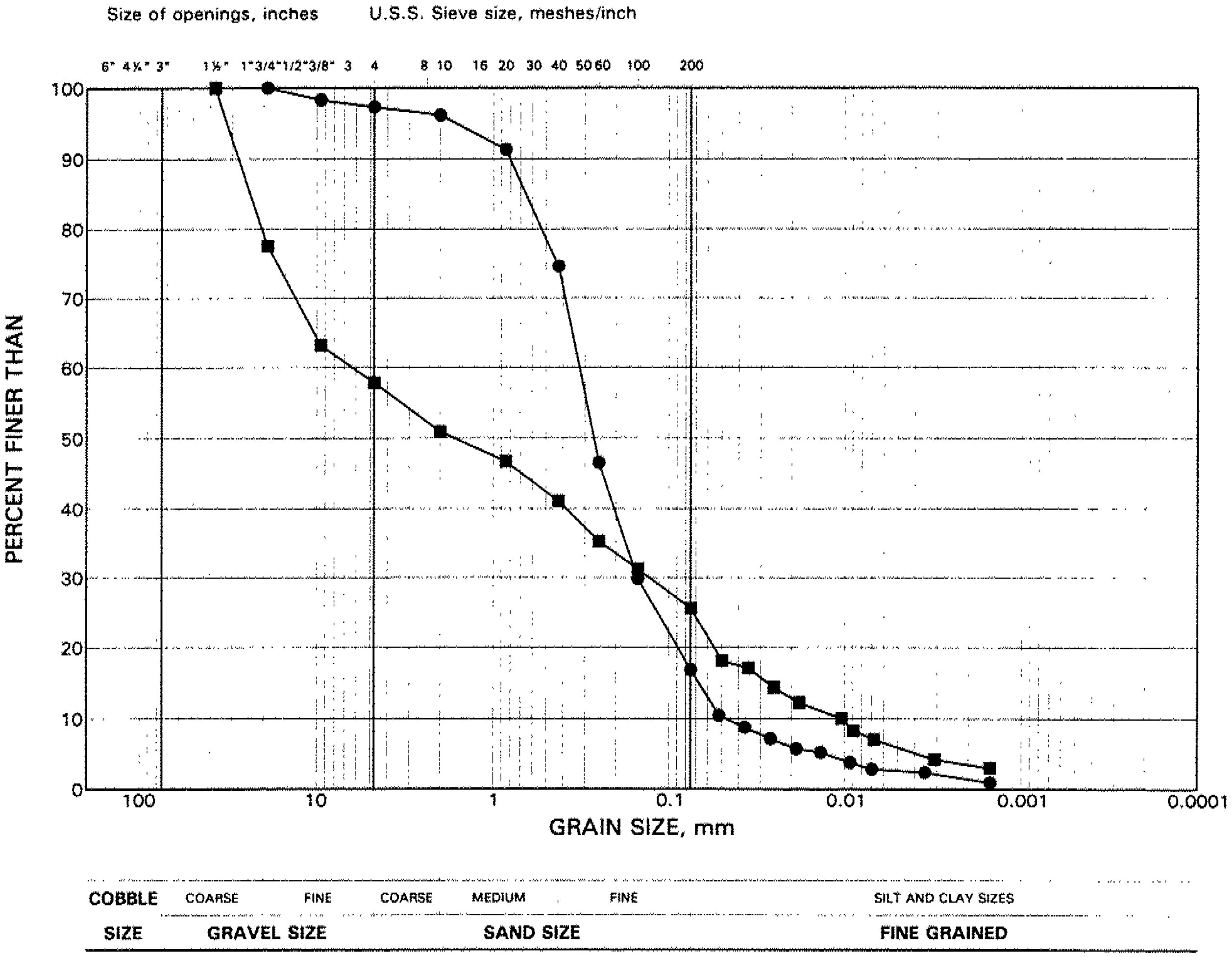
# GRAIN SIZE DISTRIBUTION

Sand, some silt, trace gravel; Silty Sand with gravel

FIGURE 3



LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 1 | 8 | 4.9 |
| ■ | 2 | 6 | 3.7 |

Project 991-1178

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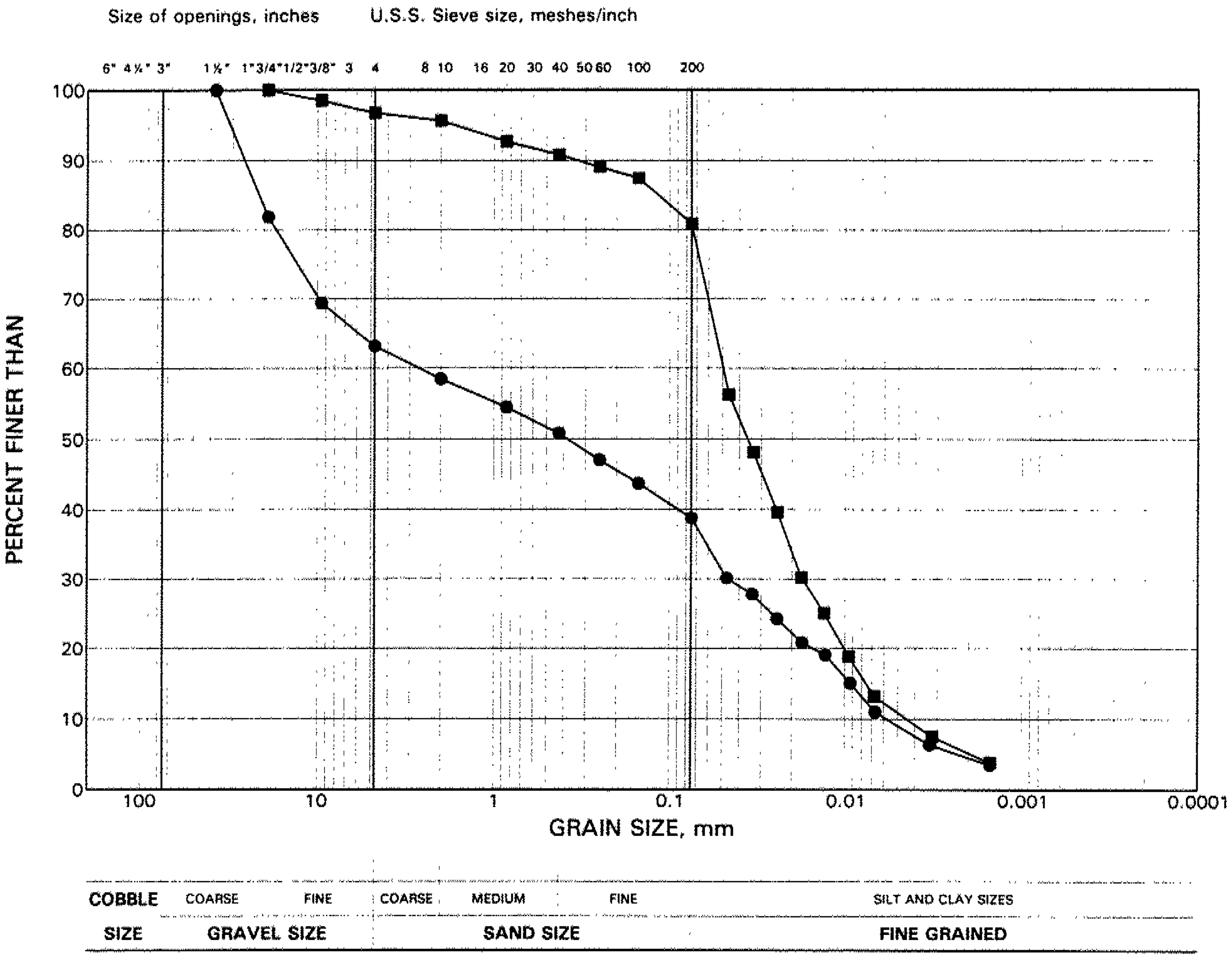
# GRAIN SIZE DISTRIBUTION
## Sandy Silt to Silt (Till)

FIGURE 4



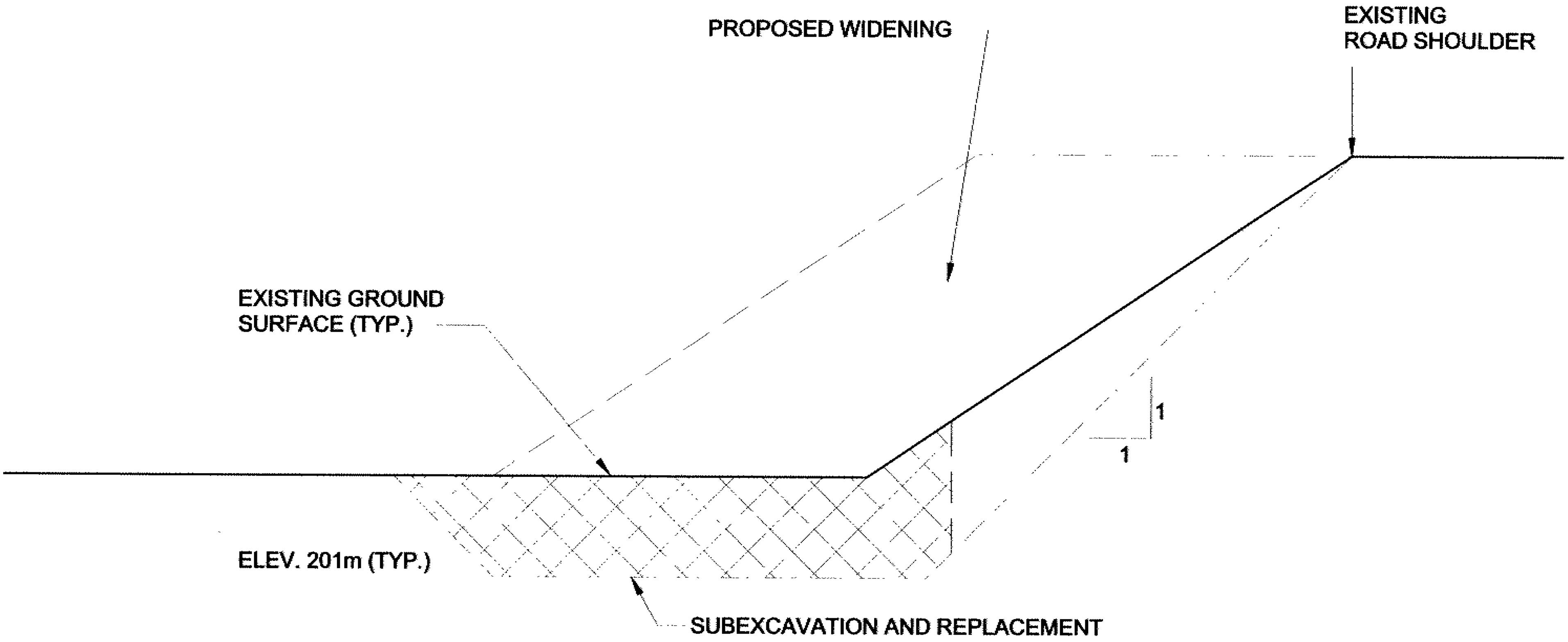
LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 1 | 9 | 5.1 |
| ■ | 7 | 7 | 4.0 |

[illegible]

# SCHEMATIC SHOWING ORGANIC/PEAT SUBEXCAVATION CONFIGURATION

FIGURE 5



NOTES:

1. WHERE EXISTING EMBANKMENT SIDE SLOPE IS STEEPER THAN OR EQUAL TO 1H:1V, THE SUBEXCAVATION LINE SHOULD BE A CONTINUATION OF THE EXISTING EMBANKMENT SIDE SLOPE.
2. BENCHING OF EXISTING EMBANKMENT SIDE SLOPE TO BE CARRIED OUT FOR PLACEMENT OF NEW FILL FOR PROPOSED WIDENING.

NOT TO SCALE

Date MAY, 2000
Project 991-1178

**Golder Associates**

Drawn JFC
Chkd

G.I.-30 SEPT. 1976

GEOCRES No. 31D-383

DIST. 43 REGION

W.P. No. 355-97-00

CONT. No.

W. O. No.

STR. SITE No. 29-85

HWY. No. 7

LOCATION Indian River Bridge

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**Golder Associates Ltd.**

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

REPORT ON

# FOUNDATION INVESTIGATION AND DESIGN
# INDIAN RIVER BRIDGE WIDENING
# HIGHWAY 7
# PETERBOROUGH, ONTARIO
# W.P. 355-97-00

Submitted to:

The Greer Galloway Group Inc.
973 Crawford Drive
Peterborough, Ontario
K9J 3X1

GEOCRES #31D-383

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Peterborough, Ontario

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Mississauga, Ontario

May 2000

991-1178

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### PART B - FOUNDATION DESIGN

# PART A

# FIELD INVESTIGATION
# INDIAN RIVER BRIDGE WIDENING
HIGHWAY 7
PETERBOROUGH, ONTARIO
W.P. 355-97-00

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### PART A - FIELD INVESTIGATION

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by The Greer Galloway Group Inc. (Greer Galloway) to carry out a foundation investigation for the widening of the Indian River Bridge located on Highway 7 east of Peterborough, Ontario. The existing bridge is a two span structure and will be widened to accommodate the required traffic lane width and a pedestrian walkway.

The purpose of the investigation is to determine the subsurface conditions at the location of the foundation units of the proposed widening and at the approach embankments by drilling boreholes and performing laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of foundation design and construction are provided.

The plan and profile of the proposed Indian River Bridge widening on Highway 7 were provided to Golder by Greer Galloway. The terms of reference for the scope of work are outlined in Golder's proposal P91-1377, dated October 14, 1999.

## 2.0 SITE DESCRIPTION

The site is located on Highway 7 at the Indian River crossing, and is situated approximately 15 km east of Peterborough, Ontario in the Township of Otonabee (see Figure 1). Indian River flows approximately north-south within a relatively wide shallow floodplain.

The existing Highway 7 embankments approaching the bridge are about 2 m to 3 m in height above the natural ground surface. The embankment side slopes are at about 2 horizontal to 1 vertical and are grass covered. The south side of the east approach embankment appears to have been built up by some extent of infilling of the river channel. The river is relatively shallow at the bridge; generally less than 1 m in water depth.

The existing bridge is a two span rigid frame structure originally constructed in 1933 / 1934 and rehabilitated in 1984. The spans are 15.24 m long and the bridge is about 10 m wide. The existing bridge abutments are located immediately adjacent to the edges of the river.

The top of asphalt on the existing bridge deck is at about Elevation 204.5 m. The river bed level is at about Elevation 201.1 m and the water level in the river was at about Elevation 201.4 m at the time of this investigation. The 100 year storm high water level is at about Elevation 202.7 m.

## 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried between November 18 and November 24, 1999 at which time, seven boreholes were advanced at the site. The table below summarizes the borehole locations and depths. The bedrock surface and condition was proved in Boreholes 99-3, 99-4 and 99-5 by rock coring. The investigated locations are shown in plan on Figure 2.

**Summary of Boreholes**

| *Borehole No.* | *Location* | *Ground Surface Elevation (m)* | *Borehole Depth (m)* |
|---|---|---|---|
| 99-1 | west approach | 204.53 | 5.1 |
| 99-2 | existing west abutment | 204.39 | 4.8 |
| 99-3 | proposed west abutment | 202.08 | 6.5 |
| 99-4 | pier | 201.08 | 5.5 |
| 99-5 | proposed east abutment | 202.17 | 5.8 |
| 99-6 | existing east abutment | 204.39 | 3.6 |
| 99-7 | east approach | 204.35 | 4.0 |

The boreholes were drilled and sampled using portable drilling equipment supplied and operated by Sonic Soils of Concord, Ontario. Sampling of the overburden material was carried out with a Dynamic Ram Sounder drilling apparatus using a 50 mm outside diameter split spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Bedrock coring in BQ size was carried out using a Prospector 89 diamond drill with BQ size casing. The overburden was sampled in each borehole, except Borehole 99-4, until split spoon sampler refusal. In Boreholes 99-3 and 99-5, sampling continued from the refused depth with the coring equipment. In Borehole 99-4, casing was advanced without sampling to a depth of 2.4 m, and rock core was obtained below this depth. Bedrock was cored to depths ranging from about 5.5 m to 6.5 m from the existing ground surface (Elevations 195.4 m to 195.6 m).

The water levels in the open boreholes were observed during drilling and a piezometer was installed in Borehole 99-2 to permit monitoring of the groundwater level adjacent to the river. A water level in the piezometer was obtained on November 23, 1999 to determine stabilized level at that time.

The field work was supervised throughout by a member of our engineering staff, who cleared the site of underground utilities, located the boreholes in the field, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in containers, labelled and transported to our Mississauga laboratory for further examination and testing. Laboratory testing on selected samples included natural water content determinations, grain size analyses, organic content and point load tests. The results of laboratory testing are given on the Record of Borehole and Record of Drillhole sheets and on Figures 1 to 4.

The ground surface elevations at the borehole locations and a borehole plan showing northing / easting coordinates were provided to Golder by Greer Galloway.

## 4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Site Geology

From published geological information, the site is located in the physiographic region known as the Peterborough Drumlin Field. This region is characterized by a rolling till plain in which the overburden consists of variable clayey to sandy till deposits generally underlain by limestone bedrock of the Trenton Formation. The region is also characterized by its numerous eskers – ridges comprised mainly of gravels. Due to the rolling terrain, bedrock is at variable depths but generally between about Elevation 180 m in the Hastings County area rising to about Elevation 245 m in the Simcoe County area.

The Trenton limestone is a relatively soft and more interbedded bedrock than other more massive limestones (such as the Black River Formation) and is also known to be highly fossiliferous in some areas.

### 4.2 Site Stratigraphy

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Record of Borehole and Record of Drillhole sheets and by the laboratory test results. It should be noted that the stratigraphic boundaries indicated on the borehole records are inferred from sampling, observations of drilling progress and results of Standard Penetration Tests (SPTs). These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The ground surface elevation at the borehole locations varies from about Elevation 201.1 m (river bed) to 204.5 m (top of existing embankment). The boreholes were sampled to depths ranging from 3.6 m to 6.5 m.

In general, the existing embankment fill consists of loose to compact brown sandy gravel containing silty clay / clayey silt pockets at some locations. The native soils encountered at the site generally consist of about 0.6 m of peat underlain in the west by deposits of silty sands and gravels, and underlain in the east by a sandy silt / silt till deposit. In Boreholes 99-5 and 99-6 (east abutment), the peat was mixed with silty clay / clayey silt. The sandy silt / silt till deposit was also encountered on the west side of the river, below the silty sands and gravels. Calcareous shale / limestone bedrock was encountered at about 2.4 m below the existing river bed. The following is a brief summary of the fill and native material encountered at the borehole locations.

### 4.2.1 Embankment Fill

The existing embankment fill material was encountered in Boreholes 99-1, 99-2, 99-5, 99-6 and 99-7 and consists mainly of brown sandy gravel containing occasional cobbles. Pockets of silty clay and clayey silt were encountered within the sandy gravel in Boreholes 99-1 and 99-7. Rootlets and organics were noted within the fill in Borehole 99-5 and within the top 0.3 m in Borehole 99-6. The state of packing of the fill is very loose to compact as indicated by measured SPT 'N' values ranging' from 2 blows to 25 blows per 0.3 m of penetration. Measured water contents of samples from the fill range from 1 percent to 10 percent (average 4 percent). Grain size distribution curves of samples from the fill are shown on Figure 2.

### 4.2.2 Peat and Peat / Clay

A deposit of dark brown to black fibrous peat, 0.6 m to 0.8 m thick, was encountered below the embankment fill in Boreholes 99-1, 99-5, 99-6 and 99-7, and at the ground surface in Borehole 99-3. The peat was mixed with grey silty clay / clayey silt in Boreholes 99-5 and 99-6. Measured SPT 'N' values in the peat and peat / clay mixture ranged from 1 blow to 6 blows per 0.3 m of penetration indicating a soft to firm consistency.

Measured water contents of samples from this deposit ranged from 19 percent to 134 percent. Organic contents of two samples were measured to be about 3 percent (peat / clay in Borehole 99-5) and 18 percent (peat in Borehole 99-1). Lower water contents were measured on the peat samples which were retrieved from below the existing embankment versus the near existing surface sample obtained from Borehole 99-3. In addition, lower average water contents were measured from the peat clay mixtures versus the peat samples (26 percent for peat / clay versus 95 percent for peat samples).

### 4.2.3 Sand / Silty Sand with Gravel

Below the peat in Boreholes 99-1 and 99-3, and below the embankment fill in Borehole 99-2, a non-cohesive deposit ranging in composition from silty sand with gravel to sand containing some silt and trace gravel was encountered. Trace organics, wood fragments and shells were contained within most samples from this deposit. Based on the gradations of samples tested, it is considered that this deposit is derived from the underlying native sandy silt / silt till deposit (see below) that has been reworked due to river action. Measured SPT 'N' values ranged from 15 blows to 23 blows per 0.3 m of penetration indicating a compact state of packing.

Measured water contents of samples from this deposit ranged from 8 percent to 17 percent (average 13 percent). Borehole 99-2 was terminated in this deposit (split spoon sampler refusal). Grain size distribution curves of samples from this deposit are shown on Figure 3.

### 4.2.4 Sandy Silt to Silt Till

A till deposit ranging in composition from sandy silt with gravel to silt containing some sand and trace gravel was encountered below the sand / silty sand with gravel deposit in Boreholes 99-1 and 99-3, and below the peat in Boreholes 99-5, 99-6 and 99-7. Trace fragments of shale were encountered within the till in Borehole 99-3. The top of this deposit ranges from about Elevations 199.6 m to 201.3 m, and was encountered at higher elevations on the east side of the river (Boreholes 99-5, 99-6 and 99-7). The state of packing of the till is dense to very dense as indicated by measured SPT 'N' values ranging from 30 blows to greater than 70 blows per 0.3 m of penetration.

Measured water contents of samples from the till ranged from 4 percent to 11 percent (average 7 percent). Grain size distribution curves of two samples from this deposit are shown on Figure 4. Boreholes 99-1, 99-6 and 99-7 were terminated in this deposit (split spoon sampler refusal).

### 4.2.5 Bedrock

In Boreholes 99-3, 99-4 and 99-5, the depth and condition of bedrock was proved by coring. The surface of the bedrock in these boreholes ranged from about Elevations 198.6 m to 199.4 m. In Boreholes 99-2 and 99-6, sampling was terminated due to refusal to advance the split spoon sampler (likely within the very dense till deposit encountered near the bedrock surface). Considering the bedrock surface elevation encountered in Boreholes 99-3, 99-4 and 99-5, and the refusal depths in Boreholes 99-2 and 99-6, the bedrock surface appears to relatively flat, increasing slightly in elevation towards the east.

The bedrock consists mainly of grey calcareous shale with interlayers of limestone within the upper 1.5 m to 2.0 m. Below this depth, the bedrock mainly consists of limestone with interlayers of calcareous shale.

The Rock Quality Designation (RQD) ranged from about 8 percent to 71 percent and was observed to increase with depth. The RQD values measured on the calcareous shale samples generally ranged from 8 percent to 45 percent indicating very poor to poor quality rock mass. The RQD values measured on the limestone samples generally ranged from 50 percent to 70 percent indicating fair quality rock mass. The state of weathering of the bedrock is described as faintly weathered in the upper 0.75 m to 1.5 m of the bedrock (between about Elevation 197.7 m and 197.9 m), and is fresh below this depth.

Diametral point load tests were carried out on samples of the calcareous shale and limestone interlayers. Measured $I_{S_{50}}$ indices for the calcareous shale ranged from about 1.6 to 4.1, with an average of 2.9. These correspond to unconfined compressive strengths ranging from about 35 MPa to 90 MPa, with an average of 65 MPa which classifies the rock as medium to medium strong; however, the upper fractured portion of the rock can be classified as very weak to weak. Measured $I_{S_{50}}$ indices for the limestone ranged from about 3.8 to 4.4, with an average of 4.2. These correspond to unconfined compressive strengths ranging from about 85 MPa to 100 MPa, with an average of about 95 MPa which classifies the limestone as strong.

### 4.3 Groundwater Conditions

At the time of this investigation, the river level was at about Elevation 201.4 m. The ground water levels in open Boreholes 99-1 and 99-7 were measured at Elevation 201.1 m and 202.3 m, respectively, upon completion of drilling. The water level in the piezometer installed in Borehole 99-2 was measured at Elevation 201.8 m on November 23, 1999. Water levels in open Boreholes 99-3 and 99-5 were not measured due to water use during rock coring. Borehole 99-6 caved at Elevation 203.7 m after drilling.

The groundwater table will be governed by the river water level and groundwater levels are expected to fluctuate due to seasonal and river water level fluctuations.

**GOLDER ASSOCIATES LTD.**

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

Anne S. Poschmann, P.Eng.
Principal

F. J. HEFFERNAN

Fintan J. Heffernan, P.Eng.
Designated MTO Contact

ASP/FJH/clg
FINALDAT\1000\991-1178\2000\991178ER1.DOC

# PART B

# FOUNDATION DESIGN
# INDIAN RIVER BRIDGE WIDENING
# HIGHWAY 7
# PETERBOROUGH, ONTARIO
# W.P. 355-97-00

## 5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the geotechnical aspects of the design of the proposed Indian River bridge widening based on our interpretation of the factual geotechnical data obtained during the investigation. The recommendations provided are intended for the guidance of the design engineer only. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

The works described in this report are associated with the proposed bridge widening and the approach embankments within 20 m of the structure. The proposed construction consists of widening the existing Indian River Bridge by 1.5 m along the north side to accommodate a pedestrian walkway, and at least 3.0 m to the south to accommodate the required travel width. The proposed construction will require the existing foundation units and embankments to be extended to the south by up to 5.7 m.

### 5.1 Bridge Foundations

The available information (Planning and Design Report) indicates that the existing Indian River Bridge structure is supported on spread footings founded on the bedrock surface. The proposed widening should also be supported on the bedrock surface. The overburden materials overlying the bedrock generally consists of silty sand with gravel grading to sandy silt till. The groundwater level is at about 2 m above the bedrock surface and excavations for foundations extended to the bedrock will have to deal with groundwater inflow.

The foundation alternatives considered feasible for this site are spread footings placed on the bedrock surface (as the existing bridge footings) and caissons extended to the bedrock surface. These two options are discussed below.

The following table summarizes the boreholes advanced as part of this investigation and indicates the bedrock surface as encountered in the boreholes where bedrock coring was completed.

**Summary of Boreholes**

| *Borehole No.* | *Location* | *Ground Surface Elevation (m)* | *Bedrock Surface Elevation (m)* |
|---|---|---|---|
| 1 | west approach | 204.53 | - |
| 2 | existing west abutment | 204.39 | - |
| 3 | proposed west abutment | 202.08 | 198.8 |
| 4 | pier | 201.08 | 198.6 |
| 5 | proposed east abutment | 202.17 | 199.4 |
| 6 | existing east abutment | 204.39 | - |
| 7 | east approach | 204.35 | - |

### 5.1.1 Shallow Spread Footings

Based on the details shown on the contract drawing for the original bridge, the existing footing has a design founding level of Elevation 199.7 m for the abutments and the pier. The results of the boreholes advanced as part of the current investigation indicate that bedrock is at a relatively shallow depth (between about 2.4 m and 3.3 m below the existing ground surface) although the bedrock surface is at a somewhat lower elevation than the design founding level for the existing structure. Based on this information, shallow spread footings would be suitable to support the widened bridge. As indicated above, excavations for spread footing construction will have to include some form of groundwater control.

Shallow spread footings founded on the bedrock surface may be designed for a factored bearing resistance at Ultimate Limit States (ULS) of 2,000 kPa. Serviceability Limit State (SLS) conditions do not apply to footings founded on bedrock at this site. This value of bearing resistance is for vertical concentric applied loading only; effects of possible load inclinations and eccentricity need to be taken into account as per OHBDC using the curve for cohesive soils.

For design, the following foundation elevations may be assumed:

| | |
|---|---|
| West Abutment | 198.8 m |
| Pier | 198.6 m |
| East Abutment | 199.4 m |

The above founding elevations are based on the borehole information and must be confirmed in the field. The contract should accommodate some variability in the bedrock surface and allow for removal of bedrock to the design level if there areas where the surface is higher. In addition, the contract should allow for mass concrete placement to raise the grade as may be required. All footing excavations should be inspected by qualified geotechnical personnel to ensure that the bedrock has been reached and that the base has been cleaned and that the bedrock conditions as exposed are consistent with the design assumptions.

Resistance to lateral forces / sliding resistance between the concrete footings and the bedrock should be calculated between the concrete Section 6.8.4.3 of the OHBDC assuming an unfactored angle of friction of 35 degrees. If necessary, sliding resistance can be supplemented by dowelling into the bedrock.

A value of 300 kPa may be assumed for the grout-to-rock bond stress for ULS design. This value refers to the rock-grout interface and can be used for tension design. The dowels should have a minimum of 1.0 m embedded length in the rock and the structural strength of the dowel and the compressive strength of the grout should not be exceeded.

The actual bond stress along the rock-grout interface may vary from the typical design value given and should therefore be verified in the field. Verification should be carried out on at least one dowel per foundation unit up to at least 10 percent of the total number of dowels at the site. The testing should be carried out on the first dowel installed at a site and the dowel should be tested to 125 percent of the maximum design load on the dowels. The test dowels must have a threaded length of 150 mm exposed in order to complete the testing.

#### 5.1.1.1 Frost Protection

For spread footings founded on the calcareous shale bedrock at this site, soil cover of at least 1.5 m should be provided to the footings for frost protection.

### 5.1.2 Caissons

As an alternative to spread footings, foundations for the abutments and piers may be supported on caissons. The caisson (drilled pier) could be formed using a permanent heavy duty steel liner, capable of being driven, vibrated or oscillated through the overburden, and advanced to and keyed into the bedrock surface. The liner should be fitted with a reinforcing shoe that will allow it to be driven or rotated to seat within the bedrock.

Given the measured groundwater levels at the site, groundwater flow through the bedrock at the founding level into the caisson excavation from below should be anticipated. In addition, groundwater flow will occur at the liner / bedrock interface if a suitable seat into the bedrock is not achieved. Therefore, concrete should be placed using tremie techniques, maintaining the tremie tube below the concrete surface at all times.

It may be feasible, however, to dewater the caissons to allow cleaning and inspection prior to concrete placement. In this case, the caisson excavation should be thoroughly cleaned using bailing methods and all loose material should be removed from the bottom of the caisson before concreting to ensure intimate contact of the concrete with the bedrock surface. If the caisson excavation cannot be fully dewatered to permit inspection, the excavation should be cleaned using a water jet to remove all loose material from the sides and base. Qualified geotechnical personnel should inspect the caisson installation to ensure that the bailing methods remove all debris and that the tremie concrete is properly placed.

Due to the shallow bedrock depth encountered at the site (about 3 m or so below the existing ground surface at the toe of the existing embankment), caissons installed to this depth should be designed for end-bearing only. For design, a factored capacity at ULS for end bearing resistance of concrete caissons founded on the bedrock surface of 3,500 kPa may be assumed. SLS conditions do not apply to end-bearing caissons founded on bedrock at this site.

## 5.2 Foundation Excavations

Excavations for construction of the spread footings will extend through the embankment fill material, and native peat, sands and gravels, and till deposits. The groundwater level observed upon completion of drilling is in the order of 2 m above the bedrock surface. Therefore, difficulties in the form of caving of material and groundwater flow into the excavations extended to the bedrock surface are likely. Some form of dewatering / ground water cutoff will be required to achieve the required excavation depths and permit footing construction.

A cut-off to groundwater flow into the excavation within the overburden can be achieved with driven steel interlocking (closed) sheet piling. Due to the strength of the bedrock, however, there is a potential for the sheet piles to meet practical refusal during driving without forming an effective cut-off to groundwater flow under the sheet piles at the bedrock interface. In addition, fractures within the bedrock may provide a path for groundwater flow up through the base of the excavation. Therefore, some groundwater flow into the excavation may occur from below the sheet piling and possibly up through the bedrock. It is considered that the groundwater inflow through these paths can be controlled by properly located and filtered sump pumps.

Consideration could be given to constructing the footings within open cut excavations, if space permits, and providing groundwater control with pumping from sumps at the base. A temporary support system will be required to support the existing road embankment. A soldier pile and lagging wall with raker support or rock anchors may be feasible at this site. If additional passive restraint is required at the toe of the wall, the soldier piles would either have to be socketted into the rock or rock anchors / bolts provided at the toe of the piles. Socketting of the piles into the bedrock at this site will have to deal with excavation through the limestone interlayers which are present within the upper portion of the bedrock and the limestone that underlies the shale.

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
$\gamma$ = soil unit weight = 21 $kN/m^3$

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
$\gamma$ = soil unit weight = 21 $kN/m^3$

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.

Temporary open cut slopes should not be formed steeper than 2.5 horizontal to 1 vertical. Groundwater flow through the overburden will occur and measures must be taken to deal with sloughing of the side slopes.

Available information indicates that the spread footings for the existing bridge foundation units are founded on the bedrock surface; however, the design drawings indicate a higher founding level than the design founding levels given in this report. As long as the existing footings are actually placed on the bedrock, undermining of the existing foundations should not be a concern during excavation for the new footings. However, careful inspection of the existing foundations should be carried out to confirm the actual founding condition as the excavation for the new footings advance. If the footings are in fact founded above the bedrock surface on the silty sands and sandy silts, measures should be taken to prevent undermining of the existing spread footings since the sands and silts are subject disturbance. In addition, if it is found that the existing footings are on bedrock but at a higher elevation than the current design, careful excavation of the bedrock (fractured, calcareous shale) will have to be ensured.

### 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 and the retaining walls in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B, Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the abutments and walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent 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 1.6 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 existing and new embankment fill materials and the following parameters (unfactored) may be assumed (based on the use of fill materials meeting the specifications for Select Subgrade Material):

| | |
|---|---|
| Soil unit weight<br>(assuming clean earth fill) | 21 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 Embankments

The results of the boreholes put down near the shoulder of the road indicate that at least a portion of the existing embankment was constructed directly on the native peat deposit. The boreholes are located just outside the paved width, so it is not possible to confirm whether the peat was removed from under the travelled portion of the road. Considering that the existing embankment was constructed about 30 years ago, and given the thickness and nature of the peat deposit, it is probable that whatever peat was left in place has undergone a significant amount of the expected settlement (i.e., greater than 90 percent).

Placement of additional fill material will be required on the south side of the existing embankment for the road widening. The increase in loading will induce additional settlement of the peat and peat / clay mixture encountered in the boreholes and there will be differential settlement occurring between the current travelled portion and the proposed widening.

It is recommended that these organic and soft cohesive materials be removed from under the proposed widening to as great an extent as possible prior to placement of additional fill. This can be accomplished by excavating a wedge of material formed by line drawn at 1 horizontal to 1 vertical (1H:1V) extending from the edge of the existing road shoulder to the base of the peat deposit (at about Elevation 201 m). This excavation should also remove the existing vegetative cover along the existing embankment, and should extend along the entire length of the proposed embankment widening.

The excavation for peat removal should be carried out in narrow strips with maximum width of 6 m and backfilling of each strip should be completed before commencing the next strip.

The newly placed embankment fill should be keyed into the existing embankment by benching in accordance with OPSD 208.01. Construction of embankments above the prepared subgrade may be carried out using Select Subgrade Material (in accordance with OPSS 1010). All embankment fill should be placed in regular loose lifts with loose thickness not exceeding 300 mm, and be compacted to 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to the placement of the granular subbase or base course should be compacted to 100 percent of the material's Standard Proctor maximum dry density. Vegetation cover should be established on all slopes to protect the embankment fill against surficial erosion as per OPSS 572.

Provided that the embankment subgrade is properly prepared, placement of the new embankment fill at a slope of 2H:1V would be stable. The embankment loading will result in settlement of the underlying sands and silts, however, the majority of this settlement will occur during construction and is expected to be small. The magnitude of additional long-term settlement of the embankment (following construction) is therefore expected to be less than 25 mm. It should be noted, however, that this settlement of the widened portion of the embankment will be differential with respect to the existing embankment.

### 5.5 Review

Prior to finalizing the design and prior to tendering, the foundation aspects of the drawings and specifications should be reviewed to confirm that the intent of this report has been met.

**GOLDER ASSOCIATES LTD.**

A. S. POSCHMANN

Anne S. Poschmann, P.Eng.
Principal

F. J. HEFFERNAN

For Fintan J. Heffernan, P.Eng.
Designated MTO Contact

ASP/FJH/clg
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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) |
| $\gamma$ | 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

$\pi$ = 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

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. in stress: $\Delta\sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| $\nu$ | Poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma - u$) |
| $\sigma'_{vo}$ | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stresses (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$ | 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 |
| $\gamma'$ | 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 $\rho$. Unit weight symbol is $\gamma$ 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 |
| $C_\alpha$ | 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 |
| $\sigma'_p$ | pre-consolidation pressure |
| OCR | Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$ |

### (e) Shear Strength

| | |
|---|---|
| $\tau_p, \tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | 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

# 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
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
II - Parallel To
⊥ - Perpendicular To

P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

Ministry of Transportation
Ontario

Foundation Design

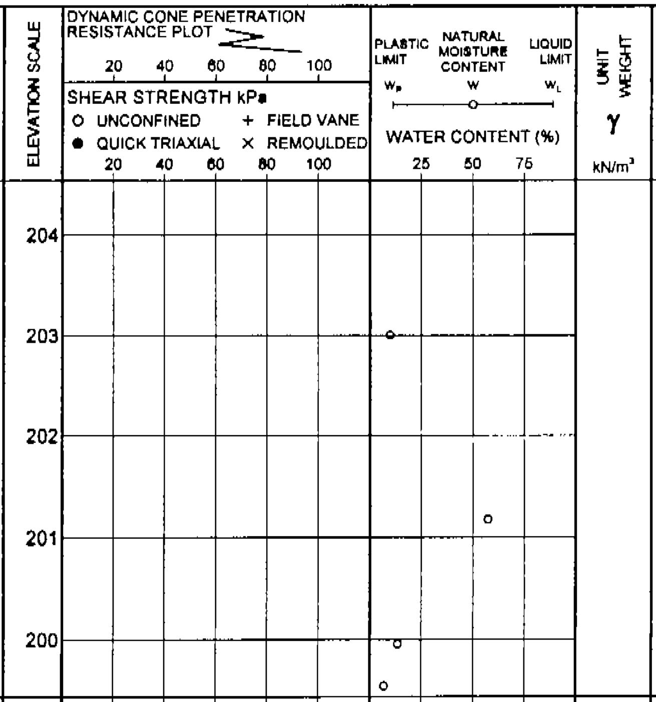
# RECORD OF BOREHOLE No 99-1

1 OF 1

**METRIC**

PROJECT 991-1178
W.P. 355-97-00
DIST
HWY #7
DATUM Geodetic
LOCATION N 4910314.7; E 411868.4
BOREHOLE TYPE Dynamic Ram Sounder
DATE 19.11.99
ORIGINATED BY SB
COMPILED BY BVB
CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | GR | SA | SI | CL |
| 204.53 | | | | | | | | | | |
| 0.00 | Sandy Gravel, with pockets of silty clay between 0.6m and 1.8m depth<br>Loose<br>Brown<br>(Fill) | | 1 | 50 DO | 9 | | | | | |
| | | | 2 | 50 DO | 8 | | | | | |
| | | | 3 | 50 DO | 9 | | 42 | 39 | 17 | 2 |
| | | | 4 | 50 DO | 6 | ⊽ | | | | |
| | | | 5 | 50 DO | 4 | | | | | |
| 201.48<br>3.05 | Fibrous Peat, trace shells<br>Firm<br>Black | | 6 | 50 DO | 6 | | | | | |
| 200.72<br>3.96 | Sandy Silt<br>Compact<br>Grey | | 7 | 50 DO | 21 | | | | | |
| 199.96<br>4.57 | Sand, some silt, trace gravel<br>Compact<br>Grey | | 8 | 50 DO | 22 | | 3 | 79 | 17 | 1 |
| 199.43<br>5.10 | Sandy Silt with gravel, trace clay<br>Compact to very dense<br>Moist<br>Grey<br>(Till) | | 9 | 50 DO | 50/.08 | | 37 | 24 | 34 | 5 |
| | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in open borehole at Elevation 202.1m upon completion of drilling. | | | | | | | | | |



$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation
Ontario

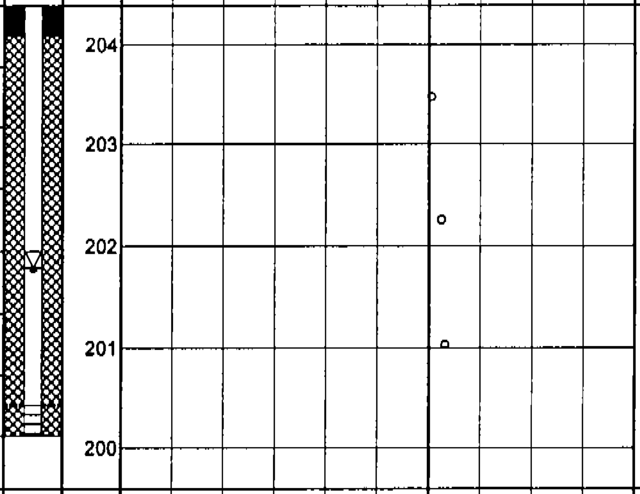
# RECORD OF BOREHOLE No 99-2

1 OF 1 **METRIC**

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910326.0; E 411882.7
ORIGINATED BY SB
DIST ____ HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 19.11.99
CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
| 204.39 | | | | | | | | | |
| 0.00 | Sandy Gravel, trace clay, trace rootlets, cobbles<br>Compact<br>Brown<br>(Fill) | ▩ | 1 | 50 DO | 19 | | | | |
| | | | 2 | 50 DO | 10 | | | | |
| | | | 3 | 50 DO | 19 | | | | |
| 201.95 | | | 4 | 50 DO | 12 | | | | |
| 2.44 | Silty Sand with gravel, trace clay, wood fragments at 4.6m depth.<br>Compact to very dense<br>Grey | ⁝ | 5 | 50 DO | 19 | | | | |
| | | | 6 | 50 DO | 21 | 42 | 32 | 23 | 3 |
| | | | 7 | 50 DO | 22 | | | | |
| 199.59 | | | 8 | 50 DO | 23 | | | | |
| 4.80 | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in piezometer at Elevation 201.79 on Nov.23/99. | | | | | | | | |

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 (%) 25 50 75 | UNIT WEIGHT γ kN/m³



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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No 99-3

1 OF 1

**METRIC**

PROJECT 991-1178

W.P. 365-97-00

LOCATION N 4910321.5; E 411608.4

ORIGINATED BY SB

DIST

HWY #7

BOREHOLE TYPE Dynamic Ram Sounder

COMPILED BY BVB

DATUM Geodetic

DATE 19.11.99

CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS |
|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | |
| 202.08 | | | | | | |
| 0.00 | Fibrous Peat<br>Soft<br>Black | | 1 | 50 DO | 1 | |
| 201.32<br>0.76 | Silty Sand with gravel, trace organics, trace shells<br>Compact<br>Grey | | 2 | 50 DO | 25 | |
| | | | 3 | 50 DO | 15 | |
| | | | 4 | 50 DO | 19 | |
| 199.64<br>2.44 | Sandy Silt, trace gravel, trace clay, shale fragments<br>Very dense<br>Grey<br>(Till) | | 5 | 50 DO | 68 | |
| | | | 6 | 50 DO | 25/0.1 | |
| 198.82<br>3.26 | Calcareous Shale with limestone interlayers grading to Limestone with calcareous shale interlayers at 5.2m depth<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br>Bedrock cored between 3.26m and 6.47m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-3. | | | | | |
| 195.61<br>6.47 | END OF BOREHOLE | | | | | |

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 $W_L$

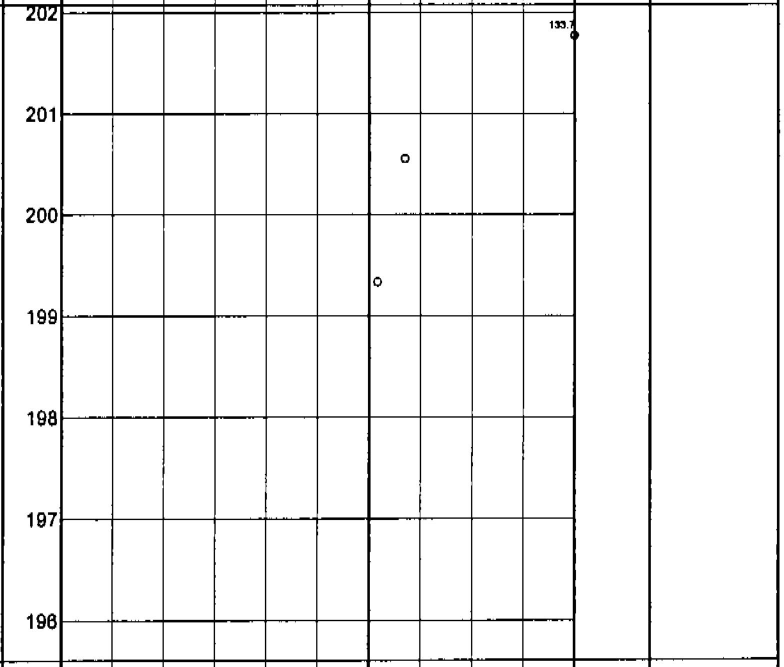
WATER CONTENT (%)

25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL



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

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-3

SHEET 1 OF 1

LOCATION: N 4910321.5; E 411808.4

INCLINATION: -90°    AZIMUTH: —

DRILLING DATE: November 24, 1999

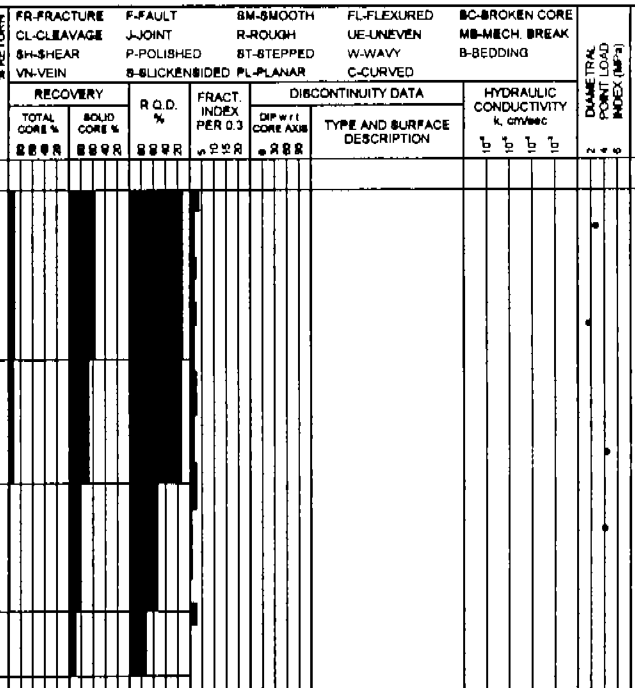
DRILL RIG: Prospector 89

DRILLING CONTRACTOR: Sonic Soils

DATUM: Geodetic

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. |
|---|---|---|---|---|
| | | *CONTINUED FROM PREVIOUS PAGE* | 198.82 | |
| 4<br>5 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered to fresh<br>Grey<br>Very poor quality<br>Weak to strong<br>(Bedrock) | 3.26<br>196.88 | 1<br>2 |
| 6 | | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | 5.20<br>195.61 | 3<br>4 |
| 7<br>8<br>9<br>10<br>11<br>12<br>13 | | END OF HOLE | 6.47 | |



NOTES
WATER LEVELS
INSTRUMENTATION

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: BVB
CHECKED: ASP

PROJECT 991-1178

# RECORD OF BOREHOLE No 99-4

1 OF 1

**METRIC**

W.P. 355-97-00
LOCATION N 4910334.7; E 411898.9
ORIGINATED BY SB
DIST
HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 24.11.99
CHECKED BY ASP

| SOIL PROFILE | | | SAMPLES | | | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT $\gamma$ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH<br>201.08 | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | | | | |
| 0.00 | Unsampled<br>Probable Silty Sand and Gravel | | | | | | 201<br>200<br>199 | | | | |
| 198.64<br>2.44 | Calcareous Shale with limestone interlayers grading to Limestone with calcareous shale interlayers at 4.7m depth<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br>Bedrock cored between 2.44m and 5.49m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-4. | | | | | | 198<br>197<br>196 | | | | |
| 195.59<br>5.49 | END OF BOREHOLE | | | | | | | | | | |

ON_MOT 991-1178.GPJ ON_MOT.GDT 5/1/00

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-4

SHEET 1 OF 1

LOCATION: N 4910334.7; E 411898.9

INCLINATION: -90°     AZIMUTH: —

DRILLING DATE: November 23, 1999

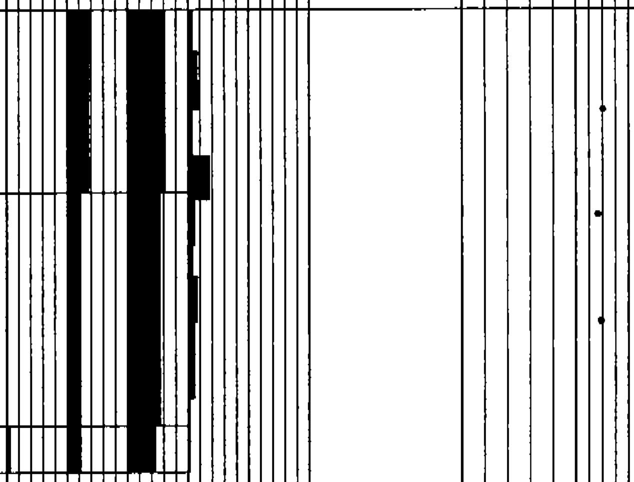
DRILL RIG: Prospector 89

DRILLING CONTRACTOR: Sonic Soils

DATUM: Geodetic

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | COLOUR % RETURN / FLUSH | RECOVERY: TOTAL CORE % (20 40 60 80) | RECOVERY: SOLID CORE % (20 40 60 80) | R.Q.D. % (20 40 60 80) | FRACT INDEX PER 0.3 (5 10 15 20) | DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90) | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | HYDRAULIC CONDUCTIVITY k, cm/sec ($10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$) | DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6) | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *CONTINUED FROM PREVIOUS PAGE* | | 198.64 | | | | | | | | | | | | |
| 3 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered<br>Grey<br>Poor quality<br>Weak to strong<br>(Bedrock) | | 2.44<br>197.88 | 1 | | | | | | | | | | | |
| 4 | | Calcareous Shale, limestone interlayers<br>Fresh<br>Grey<br>Poor quality<br>Weak to strong<br>(Bedrock) | | 3.20<br>196.35 | 2 | | | | | | | | | | | |
| 5 | | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | | 4.73<br>194.59 | 3 | | | | | | | | | | | |
| 6<br>7<br>8<br>9<br>10<br>11<br>12 | | END OF HOLE | | 6.49 | | | | | | | | | | | | |



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: BVB
CHECKED: ASP

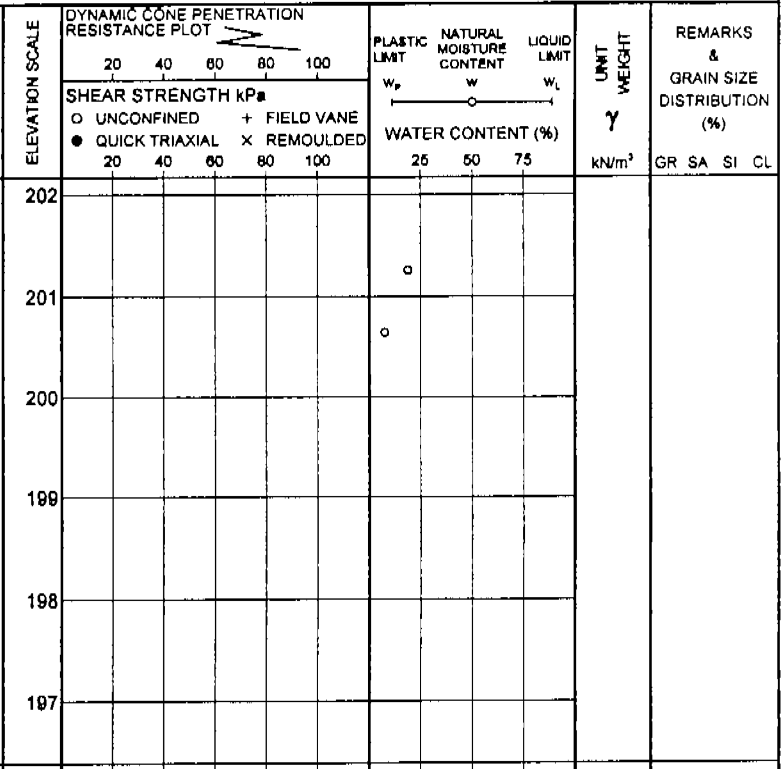
# RECORD OF BOREHOLE No 99-5

1 OF 1

**METRIC**

PROJECT 991-1178
W.P. 355-97-00
LOCATION N 4910342.8; E 411913.5
ORIGINATED BY SB
DIST ______ HWY #7
BOREHOLE TYPE Dynamic Ram Sounder
COMPILED BY BVB
DATUM Geodetic
DATE 18.11.99
CHECKED BY ASP

| ELEV / DEPTH | SOIL PROFILE – DESCRIPTION | STRAT PLOT | SAMPLES – NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS |
|---|---|---|---|---|---|---|
| 202.17 | | | | | | |
| 0.00 | Sand, trace gravel, trace rootlets, trace organics<br>Loose<br>Brown<br>(Fill) | | 1 | 50 DO | 2 | |
| 201.56<br>0.61 | Silty Clay with sand and gravel, mixed with black fibrous peat and wood fragments<br>Very loose<br>Grey | | 2 | 50 DO | 2 | |
| 200.95<br>1.22 | Sandy Silt, trace gravel<br>Compact<br>Grey<br>(Till) | | 3 | 50 DO | 27 | |
| 199.43<br>2.74 | Calcareous Shale with limestone interlayers becoming Limestone with calcareous shale interlayers<br>Faintly weathered to fresh<br>Grey<br>(Bedrock)<br>Bedrock cored between 2.74m and 5.79m depth.<br>For bedrock coring details, refer to Record of Drillhole 99-5. | | | | | |
| 196.38<br>5.79 | END OF BOREHOLE<br>Note:<br>Borehole continued below 1.8m depth, after sampler refusal encountered, using casing and wash bore techniques. | | | | | |



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

PROJECT: 991-1178

# RECORD OF DRILLHOLE: 99-5

SHEET 1 OF 1

LOCATION: N 4910342.8; E 411913.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 22, 1999

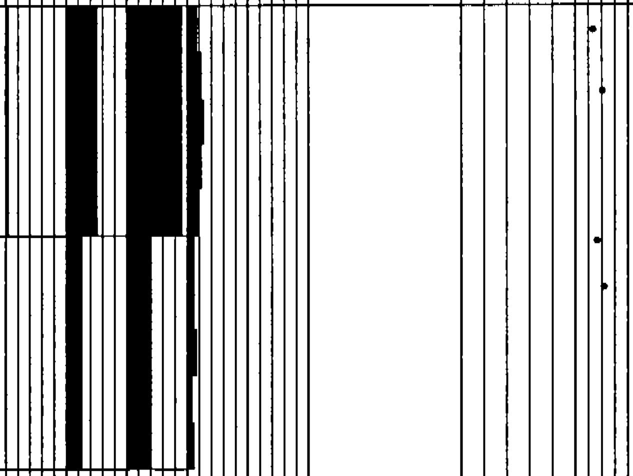
DRILL RIG: Prospector 89

DRILLING CONTRACTOR: Sonic Soils

DATUM: Geodetic

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

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | COLOUR % RETURN | FLUSH | RECOVERY TOTAL CORE % | RECOVERY SOLID CORE % | R.Q.D. % | FRACT INDEX PER 0.3 | 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* | | 199.43 | | | | | | | | | | | | | |
| 3 | BQ CORE | Calcareous Shale, limestone interlayers<br>Faintly weathered<br>Grey<br>Very poor quality<br>Weak to strong<br>(Bedrock) | | 2.74<br>197.91 | 1 | | | | | | | | | | | | |
| 4–5 | BQ CORE | Limestone, calcareous shale interlayers<br>Fresh<br>Grey<br>Fair quality<br>Strong<br>(Bedrock) | | 4.26<br>196.38 | 2 | | | | | | | | | | | | |
| 6 | | END OF HOLE | | 5.79 | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | |



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BVB

CHECKED: ASP

# RECORD OF BOREHOLE No 99-6

1 OF 1 **METRIC**

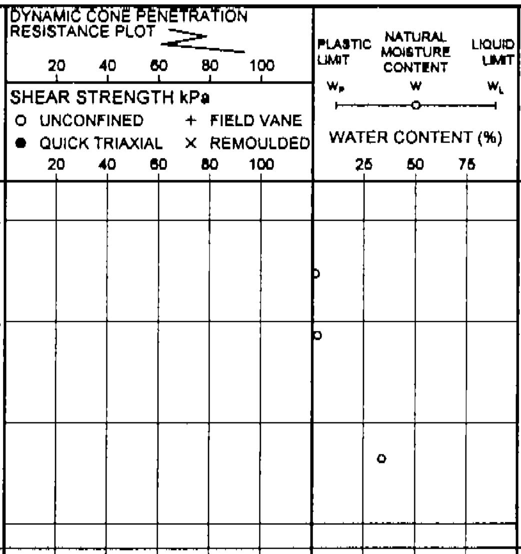
PROJECT 991-1178

W.P. 355-97-00 LOCATION N 4910345.7; E 411911.9 ORIGINATED BY SB

DIST ______ HWY #7 BOREHOLE TYPE Dynamic Ram Sounder COMPILED BY BVB

DATUM Geodetic DATE 18.11.99 CHECKED BY ASP

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | | GR | SA | SI | CL |
| 204.39<br>0.00 | Sandy gravel, trace organics and rootlets in upper 0.3m, cobbles<br>Compact<br>Brown<br>(Fill) | 1 | 50 DO | 15 | | 204 | | | | | |
| | | 2 | 50 DO | 19 | | | | | | | |
| | | 3 | 50 DO | 16 | | 203 | | 72 | 23 | 4 | 1 |
| | | 4 | 50 DO | 18 | | 202 | | | | | |
| | | 5 | 50 DO | 26 | | | | | | | |
| 201.60<br>2.79 | Fibrous peat, with clayey silt<br>Firm<br>Black/Grey | | | | | | | | | | |
| 201.04 | Sandy Silt, trace gravel<br>Very dense<br>(Till) | 6 | 50 DO | 27 | | 201 | | | | | |
| 200.76<br>3.63 | END OF BOREHOLE<br>Sampler Refusal<br><br>Borehole caved at 0.7m depth upon completion of drilling. | | | | | | | | | | |



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

Ministry of Transportation
Ontario

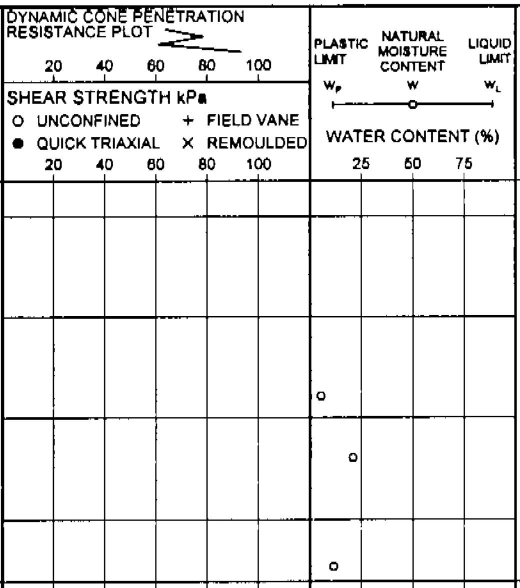
# RECORD OF BOREHOLE No 99-7

1 OF 1

**METRIC**

PROJECT 991-1178
W.P. 355-97-00
DIST
HWY #7
DATUM Geodetic
LOCATION N 4910354.6; E 411928.2
BOREHOLE TYPE Dynamic Ram Sounder
DATE 18.11.99
ORIGINATED BY SB
COMPILED BY BVB
CHECKED BY ASP

| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 204.35 | | | | | | | | | | | | |
| 0.00 | Sandy Gravel, with clayey silt pockets<br>Compact to very loose<br>Brown<br>(Fill) | ▩ | 1 | 50 DO | 9 | | 204 | | | | | |
| | | ▩ | 2 | 50 DO | 17 | | 203 | | | | | |
| | | ▩ | 3 | 50 DO | 12 | | | | | | | |
| 201.91 | | ▩ | 4 | 50 DO | 3 | ∇ | 202 | | | | | |
| 2.44<br>201.30 | Fibrous Peat with silty clay<br>Soft<br>Dark grey | ▤ | 5 | 50 DO | 3 | | | | | | | |
| 3.05 | Silt, some sand, trace gravel, trace clay<br>Dense to very dense<br>Grey<br>(Till) | ▧ | 6 | 50 DO | 30 | | 201 | | | | | |
| 200.39 | | ▧ | 7 | 50 DO | 70/.08 | | | | 3 | 16 | 76 | 5 |
| 3.96 | END OF BOREHOLE<br>Sampler Refusal<br><br>Water level in open borehole at Elevation 202.3m upon completion of drilling. | | | | | | | | | | | |



REMARKS & GRAIN SIZE DISTRIBUTION (%)

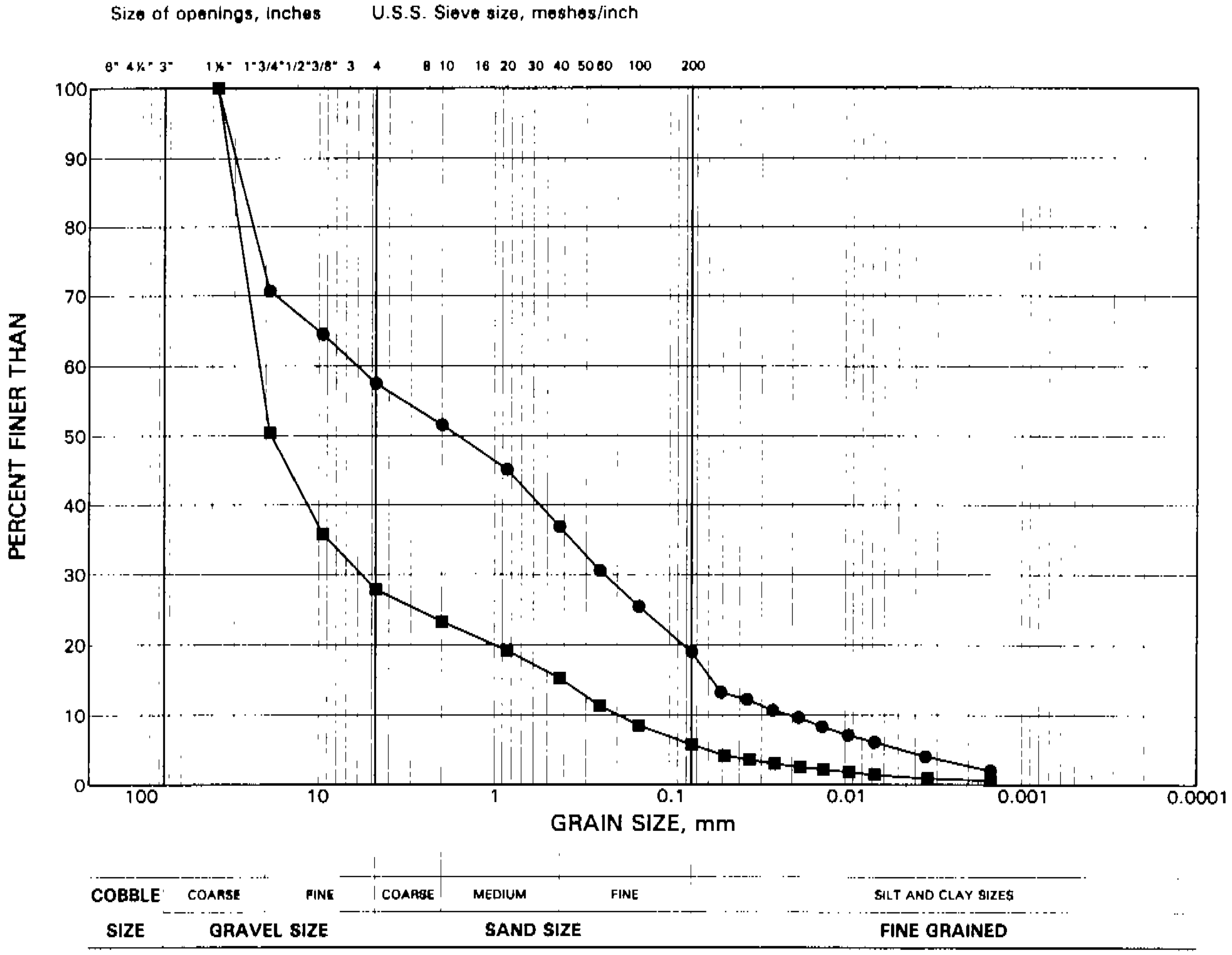
$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

# GRAIN SIZE DISTRIBUTION

Sandy gravel trace to some silt (Fill)

FIGURE 2



LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 99-1 | 3 | 1.8 |
| ■ | 99-6 | 3 | 1.8 |

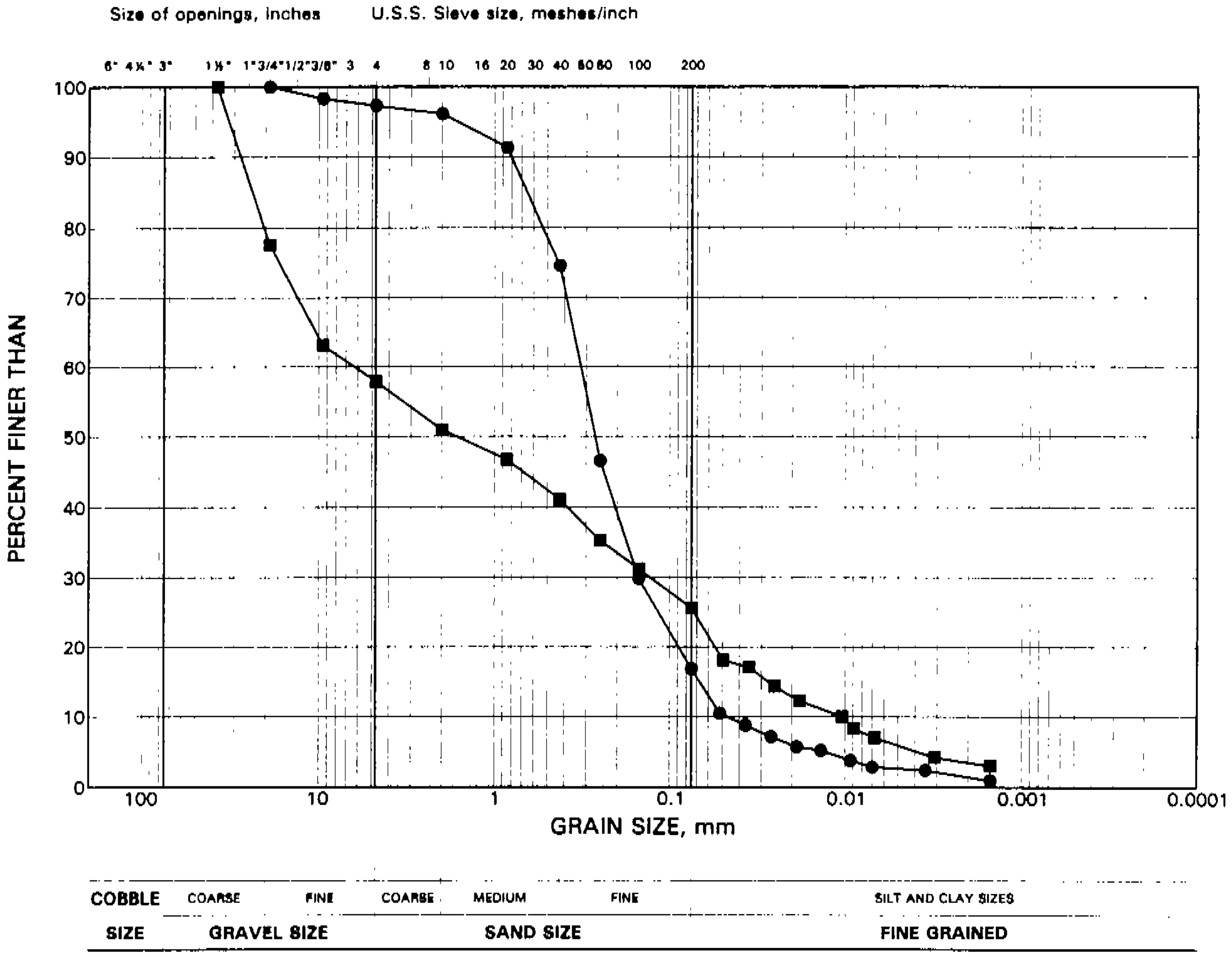
Project 991-1178

**Golder Associates**

# GRAIN SIZE DISTRIBUTION

Sand, some silt, trace gravel; Silty Sand with gravel

FIGURE 3

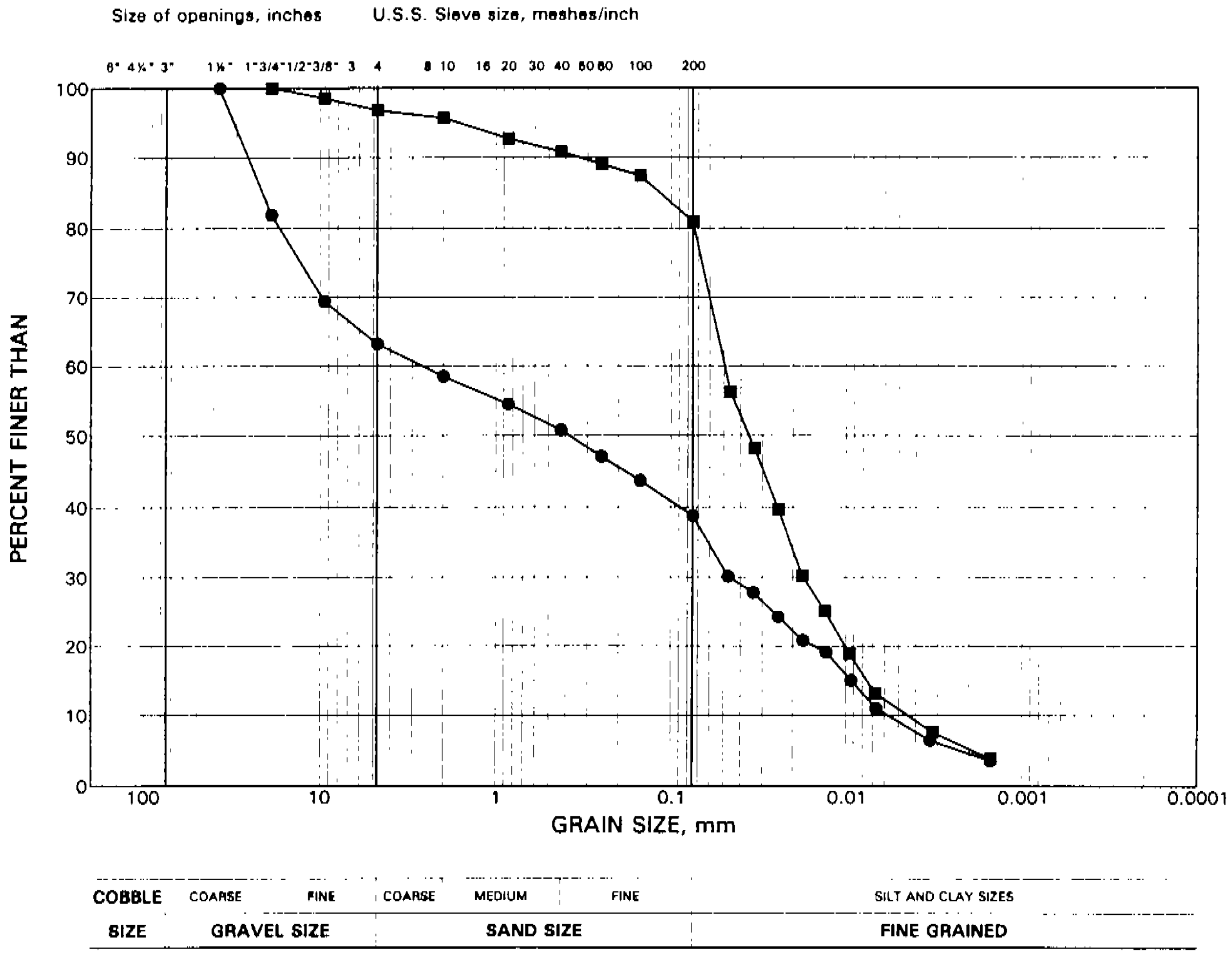


## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 1 | 8 | 4.9 |
| ■ | 2 | 6 | 3.7 |

# GRAIN SIZE DISTRIBUTION
## Sandy Silt to Silt (Till)

FIGURE 4



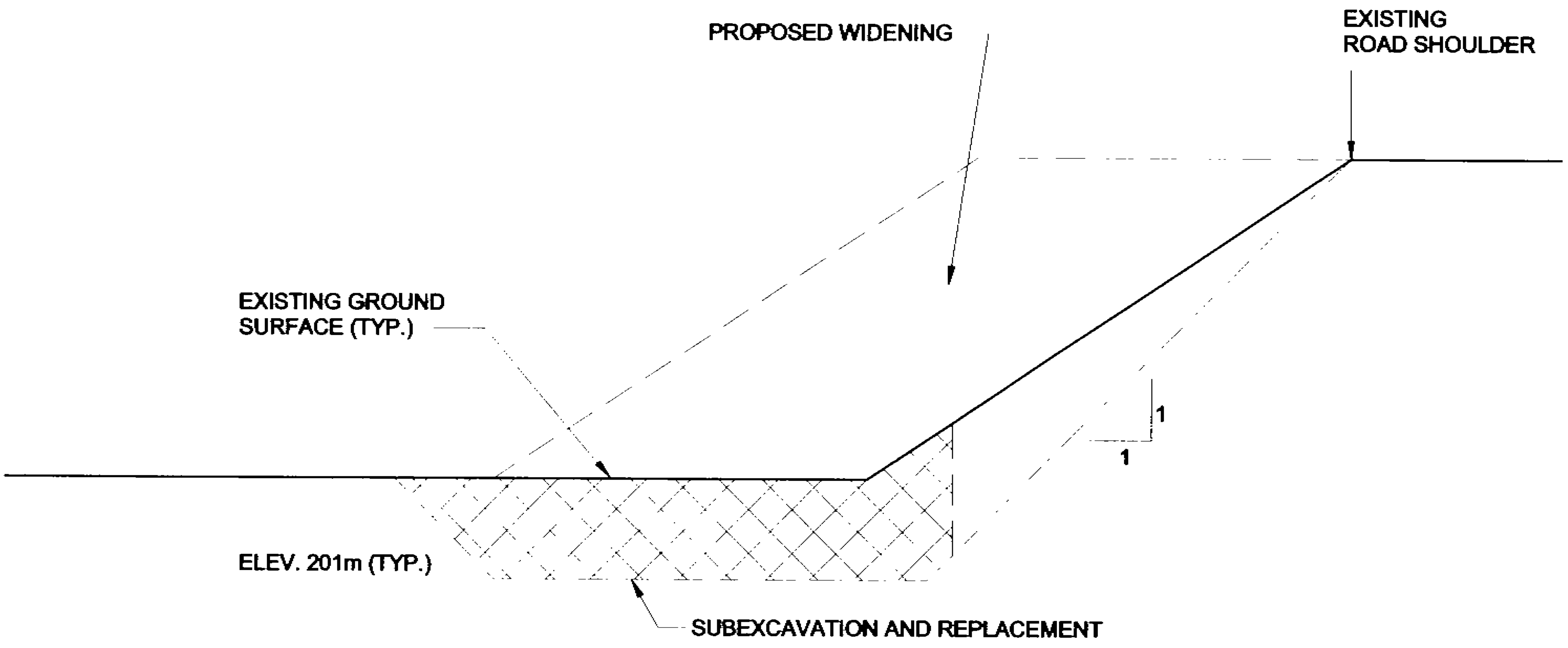
LEGEND

| SYMBOL | BOREHOLE | SAMPLE | DEPTH(m) |
|---|---|---|---|
| ● | 1 | 9 | 5.1 |
| ■ | 7 | 7 | 4.0 |

[illegible]178005.DWG

# SCHEMATIC SHOWING ORGANIC/PEAT SUBEXCAVATION CONFIGURATION

FIGURE 5



NOTES:

1. WHERE EXISTING EMBANKMENT SIDE SLOPE IS STEEPER THAN OR EQUAL TO 1H:1V, THE SUBEXCAVATION LINE SHOULD BE A CONTINUATION OF THE EXISTING EMBANKMENT SIDE SLOPE.
2. BENCHING OF EXISTING EMBANKMENT SIDE SLOPE TO BE CARRIED OUT FOR PLACEMENT OF NEW FILL FOR PROPOSED WIDENING.

NOT TO SCALE

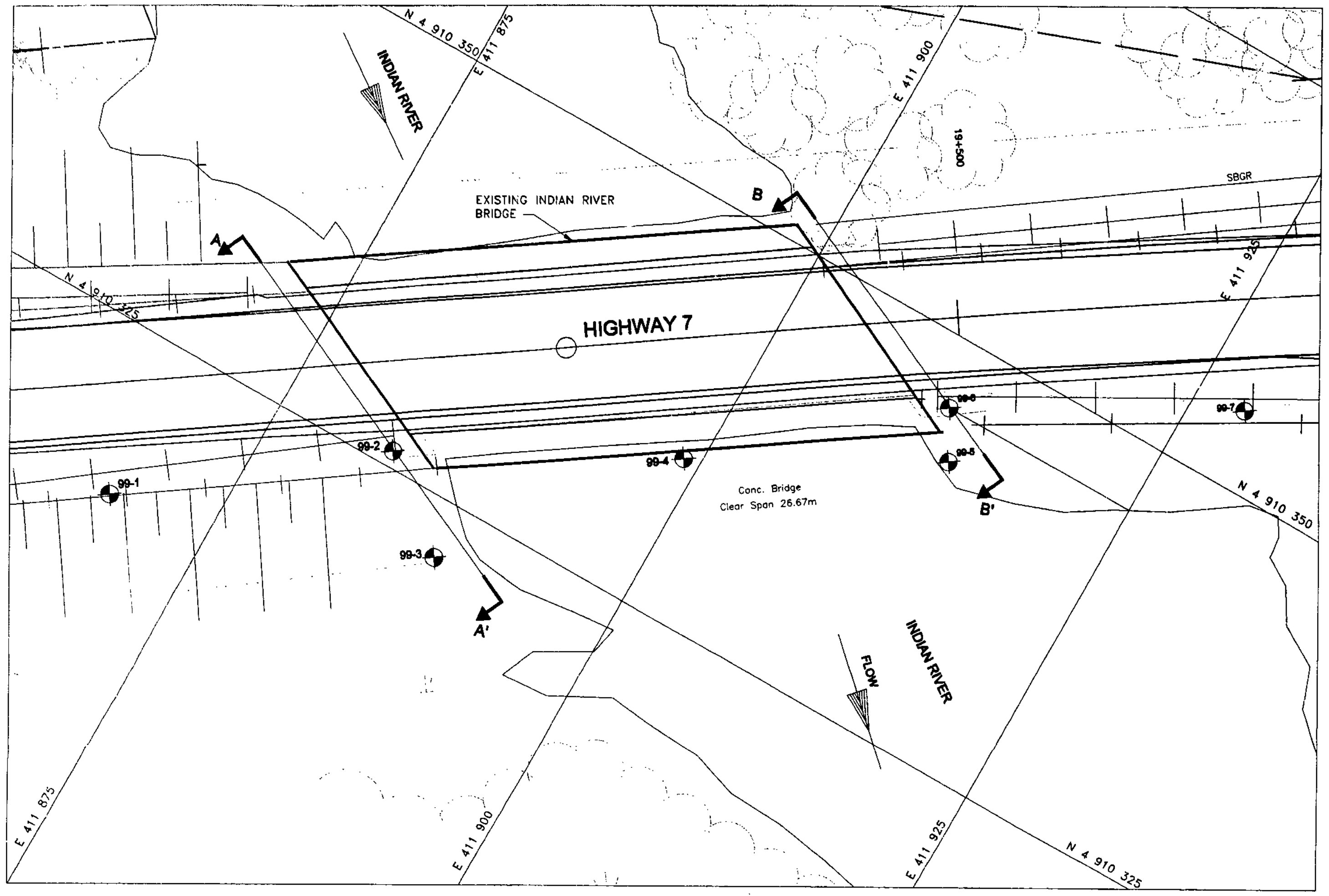
Date MAY 2000

Project 991 1178

**Golder Associates**

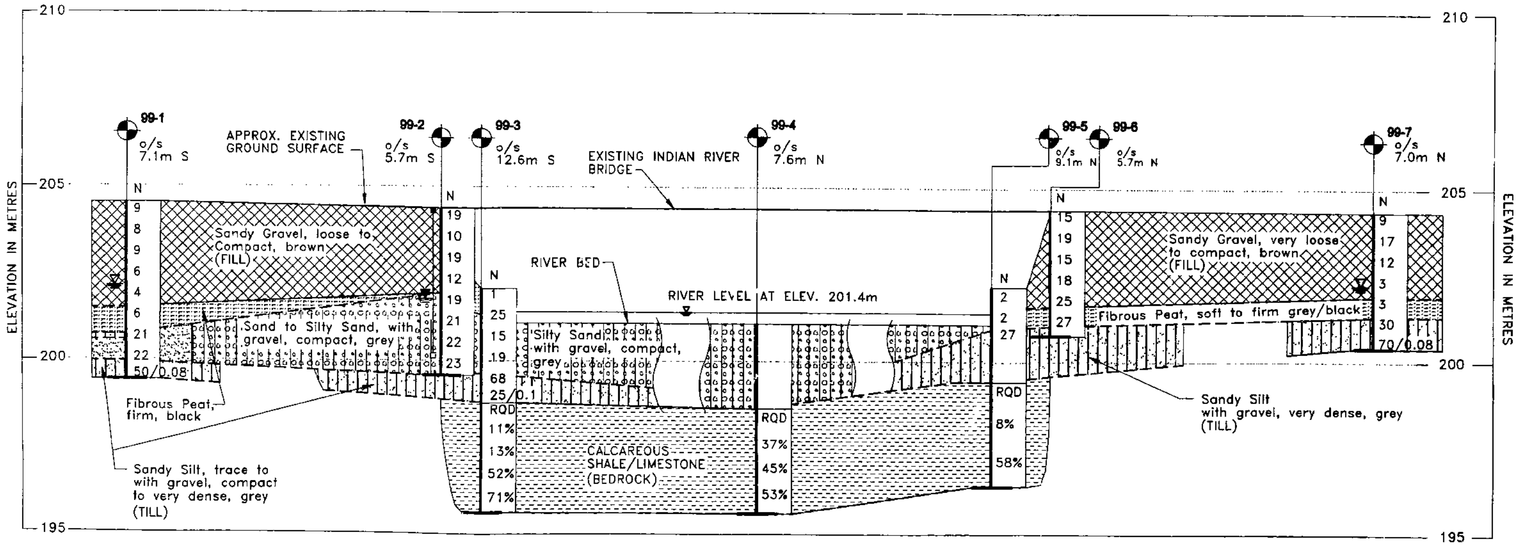
Drawn JFC

Chkd



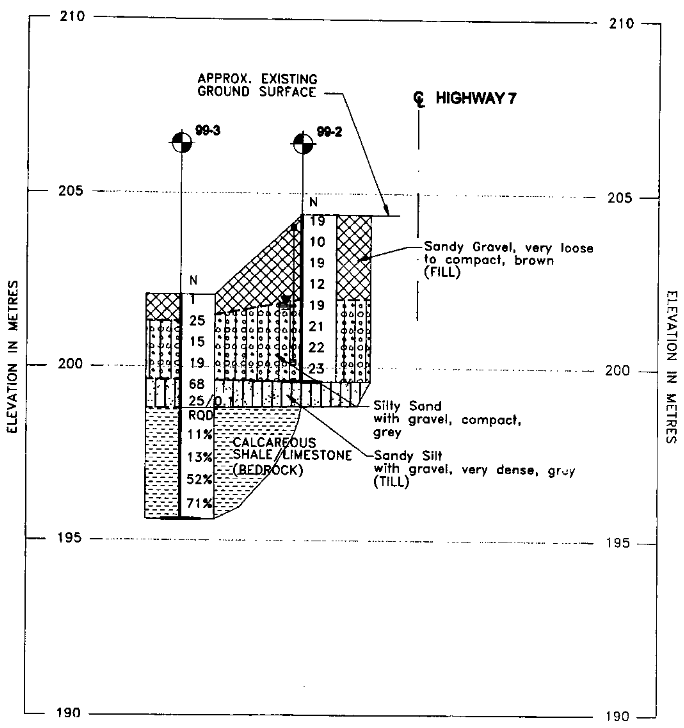
## PLAN

0 5 10 15 20 30 METRES
SCALE



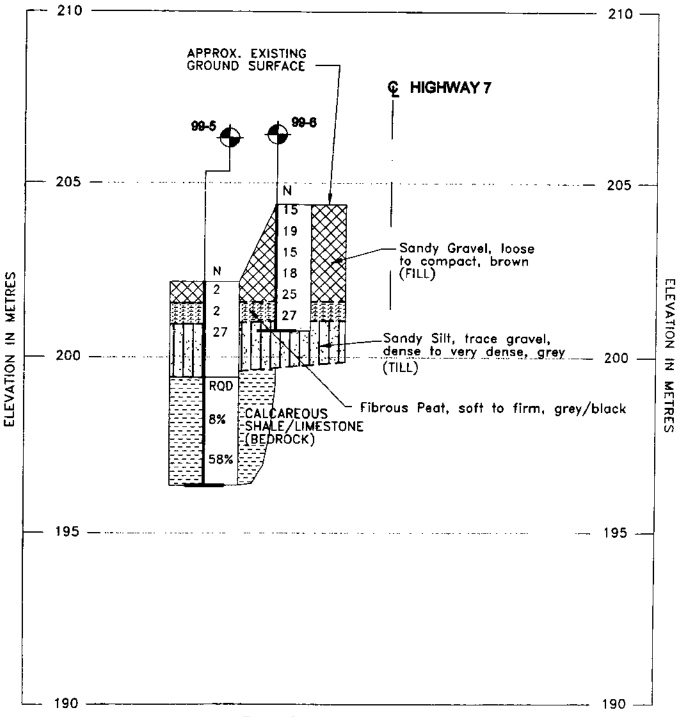
## PROFILE ALONG HWY 7 CENTRELINE

HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES



## SECTION A-A'

HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES



## SECTION B-B'

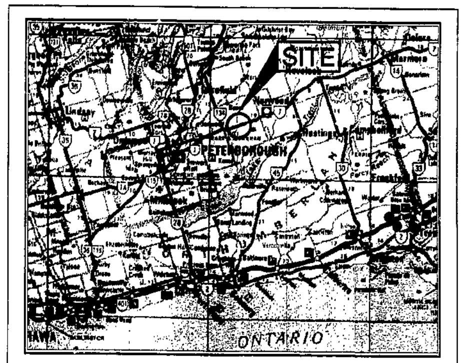
HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES

| | |
|---|---|
| CONT. No. | HWY 7 |
| WP No. | 355-97-00 |

**INDIAN RIVER BRIDGE**
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

**Golder Associates**
**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

| | |
|---|---|
| ⊕ | Borehole |
| ▮ | Seal |
| ▯ | Piezometer |
| N | Blows/0.3m (Std. Pen. Test, 475 j/blow) |
| 16 | Blows/0.3m unless otherwise stated |
| 100% | Rock Quality Designation (RQD) |
| ▼ | WL in piezometer on Nov. 23, 1999 |
| ∇ | WL upon completion of drilling |

| No. | ELEVATION | LOCATION | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 1 | 204.53 | 4910314.7 | 411868.4 |
| 2 | 204.39 | 4910326.0 | 411882.7 |
| 3 | 202.08 | 4910321.5 | 411808.4 |
| 4 | 201.08 | 4910334.7 | 411898.9 |
| 5 | 202.17 | 4910342.8 | 411913.5 |
| 6 | 204.39 | 4910345.7 | 411911.9 |
| 7 | 204.35 | 4910354.8 | 411928.2 |

NOTES

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

For detailed stratigraphy at the borehole locations, refer to Record of Borehole and Record of Drillhole sheets.

REFERENCE

This drawing was created from digital file "HWY7-BHoles.dwg" provided by Greer Galloway Group Inc.

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

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

Geocres No.

| | | | |
|---|---|---|---|
| HWY. No. 7 | | PROJECT NO.: 991-1178 | DIST. |
| SUBM'D. BVB | CHKD: ASP | DATE: 1999 12 22 | |
| DRAWN: JFC | CHKD. BVB | APPD. | DWG. 1 |

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31D-383

DIST. 43 REGION

W.P. No. 355-97-00

CONT. No.

W. O. No.

STR. SITE No. 29-85

HWY. No. 7

LOCATION Indian River Bridge

No OF PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

CONT. No. HWY 7
WP No. 355-97-00

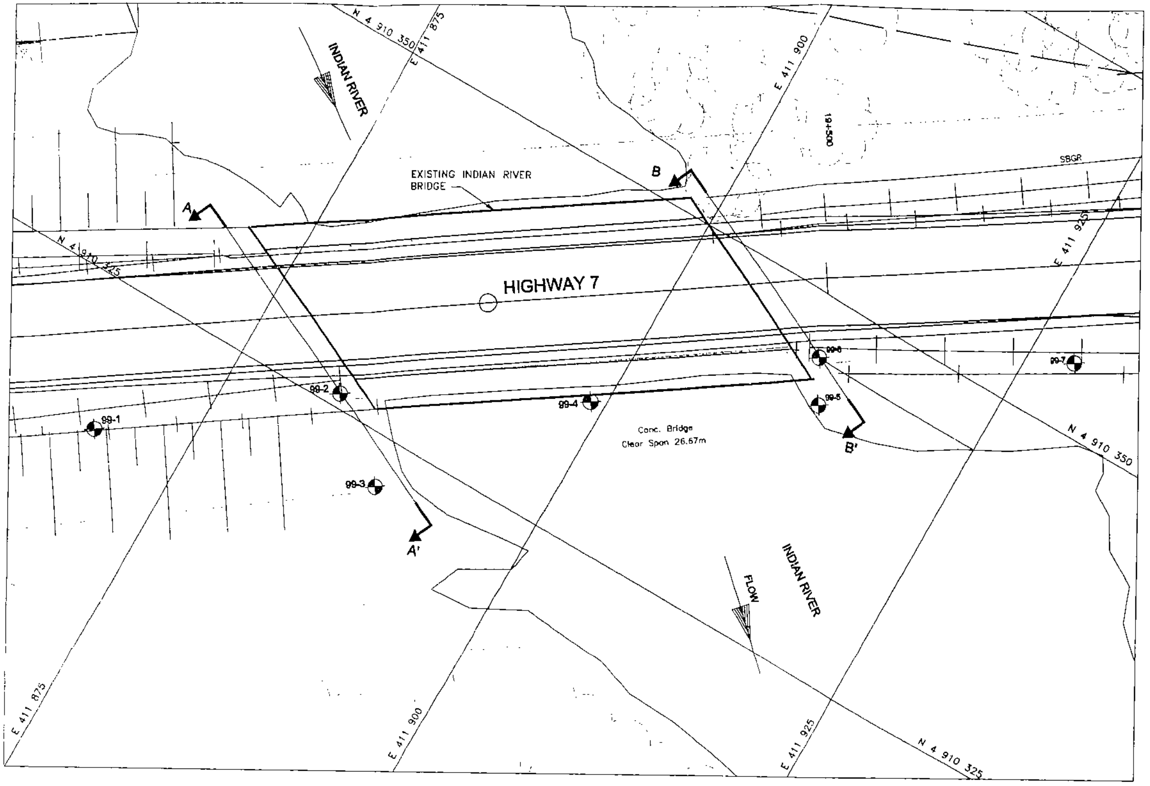
SHEET

# INDIAN RIVER BRIDGE

## BOREHOLE LOCATIONS & SOIL STRATA

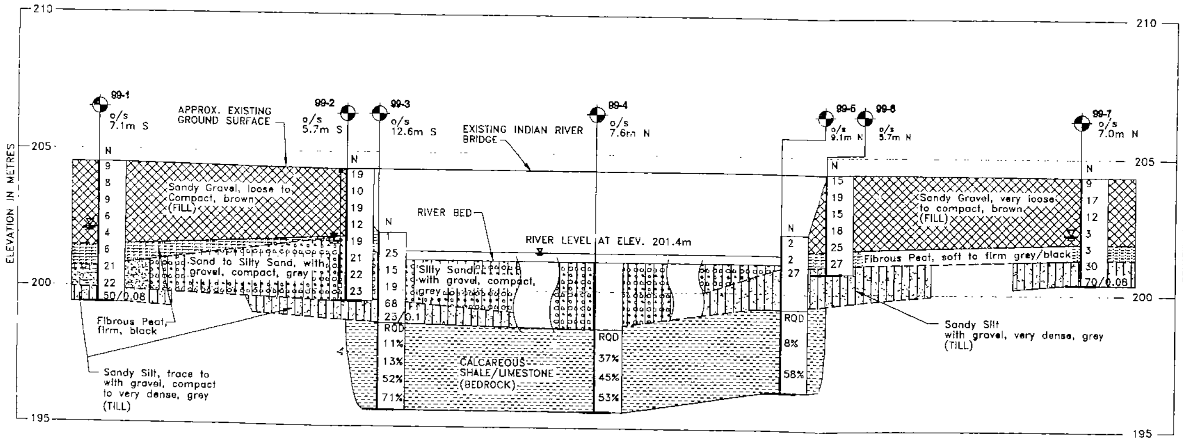
**Golder Associates**

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA



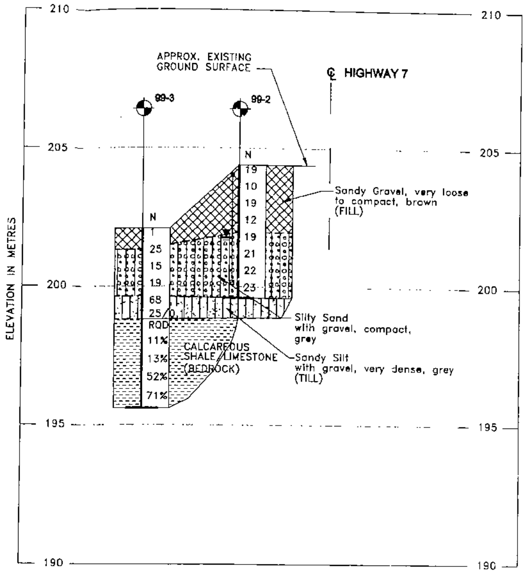
## PLAN

0 5 10 15 20 30 METRES
SCALE



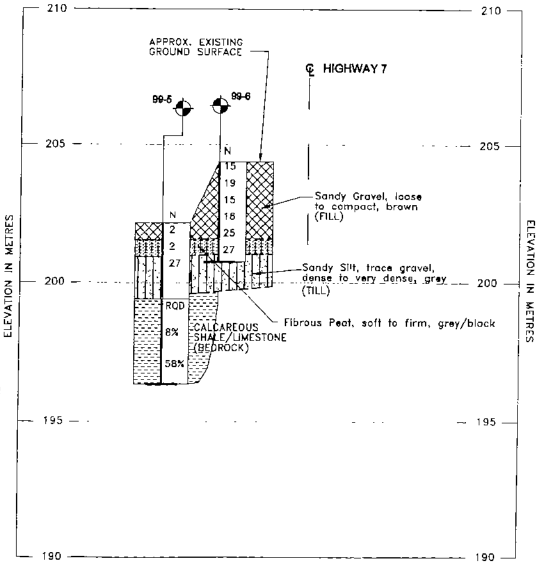
## PROFILE ALONG HWY 7 CENTRELINE

HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES



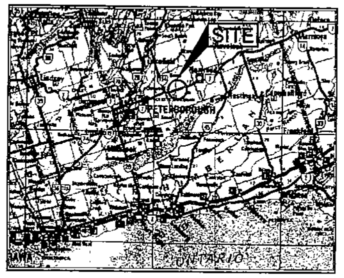
## SECTION A-A'

HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES



## SECTION B-B'

HORIZONTAL SCALE 0 4 8 12 16 20 METRES
VERTICAL SCALE 0 2 4 6 8 10 METRES



KEY PLAN

LEGEND

| | |
|---|---|
| ⊕ | Borehole |
| | Seal |
| | Piezometer |
| N | Blows/0.3m (Std. Pen. Test, 475 j/blow) |
| 16 | Blows/0.3m unless otherwise stated |
| 100% | Rock Quality Designation (RQD) |
| ▼ | WL in piezometer on Nov. 23, 1999 |
| ▽ | WL upon completion of drilling |

| No. | ELEVATION | LOCATION | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 1 | 204.53 | 4910314.7 | 411868.4 |
| 2 | 204.39 | 4910326.0 | 411882.7 |
| 3 | 202.08 | 4910321.5 | 411808.4 |
| 4 | 201.08 | 4910334.7 | 411898.9 |
| 5 | 202.17 | 4910342.8 | 411913.5 |
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| | | | |

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Geocres No.

| | | | |
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