

GEOCRES No. _____

DIST. 52 REGION _____W.P. No. 467-93-01

CONT. No. _____

W. O. No. _____

STR. SITE No. 44-391HWY. No. 11LOCATION Fern Glen Rd. UnderpassNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
FERN GLEN ROAD UNDERPASS
STRUCTURE SITE NO. 44-391
DISTRICT 52, HUNTSVILLE
W.P. 467-93-01**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada**

Submitted By:

**AGRA
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**September 1999
TT98820D**

August 31, 1999.
Ref. No.: TT98820D

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.

Dear Sir:

**Re: FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
FERN GLEN ROAD UNDERPASS
STRUCTURE SITE NO. 44-391
DISTRICT 52, HUNTSVILLE
W.P. 467-93-01**

We take pleasure in enclosing seven (7) copies of our Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

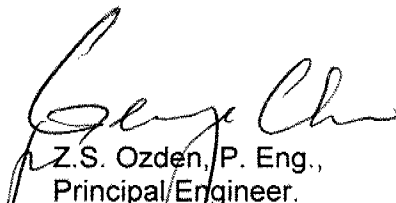

Z.S. Ozden, P. Eng.,
Principal Engineer.
ZSO/dee

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1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Delcan Corporation to conduct a foundation investigation at the site of a proposed bridge that will carry the proposed realigned Fern Glen Road over the proposed northbound and southbound lanes of Highway 11 and associated interchange ramps. The site is located near the Village of Emsdale, about 50 m south of the existing Fern Glen Road intersection with Highway 11, in the Township of Perry, Lot 16, Concession 8 in MTO District 52-Huntsville (see Key Plan, Drawing No. 1). The proposed bridge will be an approximately 70 m long, 2-lane, two span structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the foundation design of the proposed structure and associated approach fills/cuts.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located about 50 m south of the intersection of Highway 11 and existing Fern Glen Road, near the Village of Emsdale. The ground elevation in the general area of the proposed bridge site is fairly level, ranging in Elevation from about 342.5 to 340.5 m. The ground rises mildly west of the proposed bridge site, and drops to the east. The surrounding area is sparsely wooded with sand exposed along highway cuts at the existing Highway 11 intersection.

Based on available geologic information, the site is in an area of glaciolacustrine sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin depositing sands, silts and clays in the low lying areas.

Published information show that the bedrock can be expected to be composed of strongly foliated, gneissic to migmatic rocks which form part of the Central Gneiss Belt of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out during the period of December 16, 1998, February 7 to 17, April 14, and May 10 to 11, 1999, and consisted of drilling and sampling eight boreholes (Borehole Nos. FG1 through FG8) to depths of 2.3 to 15.7 m below the ground surface, forty auger probes and two dynamic cone penetration tests.

The delays in the field investigation were primarily due to permission to enter difficulties.

The plan locations of the boreholes and cone tests, along with stratigraphic sections are shown on Drawing No. 2. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, are presented on the Borehole Log Sheets. The results of the auger probes are tabulated in Appendix B.

The boreholes were advanced using solid and hollow stem continuous flight augers with a track-mounted power auger drilling rig (CME 75) owned and operated by Canadian Soil Drilling Inc., under the full-time supervision of a soils engineer from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

In addition, dynamic cone penetration tests were performed adjacent to two of the boreholes. This test consists of driving a 60° point, 50 mm diameter cone attached to the drill rod continuously into the undisturbed ground with a driving energy of 475 J (63.5 kg hammer falling freely a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Coring of the bedrock was effected by diamond drilling methods, using an NXL size core barrel.

The borehole locations were established in the field by our engineering staff, in relation to the already staked out proposed centre-line of Fern Glen Road (by Dearden and Stanton Limited). The borehole geodetic elevations and co-ordinates were subsequently taken by surveyors from Dearden and Stanton Limited and supplied to us.

The soil samples and rock cores were shipped to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also on Figure Nos. 1, 2 and 3.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. The boreholes were grouted on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at eight borehole locations (Borehole Nos. FG1 to FG8), and were inferred at the locations of two dynamic cone penetration tests and forty auger probes. The locations of the boreholes and cone penetration tests are shown on the Plan and Profile Drawing No. 2 and are also indicated on the individual Borehole Log Sheets. Cross sections of inferred subsurface stratigraphy are given on Drawing No. 2. The results of the auger probes are tabulated in Appendix B.

The ground surface at the proposed site is relatively level and the ground elevation at the proposed bridge location generally ranges from about 342.5 to 340.5 m.

Below a surficial topsoil layer, the overburden generally consists of a sand deposit to depths ranging from 2.3 to 15.3 m below existing grade. At Borehole FG3 overlying the sand deposit is a sand to gravelly sand layer which extends to a depth of 3.8 m below the ground surface while in Borehole FG7 the sand is underlain by a 1.2 m thick sandy silt layer immediately above the bedrock. Below the cohesionless overburden is the Precambrian bedrock. At the time of the investigation no groundwater was encountered within the overburden.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets. Descriptions of various strata are given in this section of the report.

4.1 TOPSOIL/FILL

A surficial topsoil layer, ranging in thickness from 0.1 to 0.25 m, was encountered at Boreholes FG4, 5, 6, 7 and 8. Measured natural moisture contents range from 12 to 18%.

At Borehole FG3, a 0.15 m thick layer of gravelly sand fill was encountered.

In our experience the thickness of topsoil and fill deposits frequently varies in between and beyond the borehole locations. In addition, at the time of our investigation the ground near the surface was frozen; therefore the soil conditions within the upper several decimeters could not be accurately determined and the descriptions given for this upper zone should be considered approximate only.

4.2 SAND TO GRAVELLY SAND

Underlying the surficial topsoil in Borehole FG3, a brown sand to gravelly sand layer was encountered to a depth of 3.8 m (Elevation 337.2 m) below existing grade. A grain size distribution analysis was conducted on a sample from this cohesionless (granular) deposit, and the resulting curve is presented in Figure No. 1. The analysis indicates:

Gravel:	29%
Sand:	68%
Silt and Clay:	3%

Measured 'N' values within this deposit range from 9 to 27 blows/0.3 m indicating a loose to compact condition. Higher values of 32 and 53 were also measured, but we believe these high values are due to the depth of frost and oversize gravel particles and are unreliable.

Measured natural moisture contents range from 3 to 5%.

4.3 SAND

Below the surficial topsoil, (gravelly sand in Borehole FG3), the predominant overburden in all of the boreholes consists of a brown sand deposit. At the borehole locations this granular (i.e. cohesionless) deposit extends to depths ranging between 2.3 m or Elevation 340.2 m (Borehole FG8) and 12.5 m or Elevation 328.5 m (Borehole FG3), generally to the surface of the bedrock. The grain size distribution of the material is generally in the fine to medium sand range. Grain size distribution analyses were conducted on twelve samples from the material and the range of particle sizes are presented in an envelope form in Figure No. 2. The analyses indicate:

Gravel:	0 - 9%
Sand:	86 - 98%
Silt and Clay:	2 - 9%

Measured 'N'-values within the surficial 2 ± m, range from 2 to 32 blows/0.3 m, indicating surficially a very loose to dense condition (generally loose to compact). Below this depth, measured 'N'-values in this deposit generally range from 20 to 59 blows/0.3 m indicating a compact to very dense condition, but generally compact to dense. Measured natural moisture contents range from 1 to 10%.

The presence of occasional cobbles and boulders was noted immediately below the ground surface in Borehole FG5 where auger refusal was encountered. Auger refusal was also encountered due to cobbles (or boulders) immediately above the bedrock surface in Borehole FG6 and rock coring was used to advance through the cobbles (from 4.1 to 4.3 m below ground surface).

4.4 SANDY SILT

Underlying the sand deposit and overlying the bedrock in Borehole FG7, a grey sandy silt layer was encountered from a depth of about 5.0 m to 6.2 m below ground surface. This is a cohesionless material and contains some gravel size particles. One grain size distribution analysis was conducted within this deposit and the results are presented in Figure No. 3. The analysis indicated:

Gravel:	20%
Sand:	40%
Silt:	40%
Clay:	0%

Because of the presence of oversized particles, the recorded 'N'-values within this layer are generally considered to be unreliable but from the overall results and resistance to augering while drilling the material appears to be compact to dense. Measured natural moisture contents are 12 and 17%.

Within this deposit auger refusal was obtained immediately above the bedrock surface. In order to advance the borehole through cobbles and boulders, triconing had to be resorted to from 5.8 to 6.2 m below ground surface (i.e. to the surface of the bedrock).

4.5 BEDROCK

Below the overburden soils, bedrock was encountered and cored to obtain NXL size cores in Boreholes FG1, 2, 3, 4, 6 and 7 at depths of 4.3 to 12.5 m below existing ground surface (Elevations 338.0 to 328.5 m).

Auger probes were advanced at the site to attempt to delineate the undulating bedrock surface. Auger refusal on probable bedrock was encountered in Borehole FG8 and in the forty auger probes advanced at the site. The probe results are given in Appendix B. The results show that the recorded refusal depths range from 2.3 m or Elevation 340.2 m in Borehole FG8 (drilled for the west approach) to 15.3 m (Elevation 327.3± m) at a probehole put down 3.5 m left of proposed centre-line at Station 10+041 near (i.e. to the east) the proposed east abutment location. It is believed that while in most cases the refusals represent the depths to the surface of the bedrock, in some cases they may be due to cobbles and boulders in the overburden (probably immediately above the bedrock elevation).

From the results it can be surmized that in the general area of the proposed west abutment location the presumed bedrock surface elevation generally ranges from 337.5 to 336.0± m, dipping down from west to east. In the general area of the proposed central pier location it ranges generally from about Elevation 334 to 331 m, while in the area of the east abutment location a more complex picture emerges with rock elevations ranging from about 334 to 327.5± m. Here the rock surface appears to dip from about Elevation 334 m in the south-west area of the proposed abutment location to about 327.5± m towards the north and north-east of the abutment location.

The depth to the surface of the bedrock below the footprint of each foundation element will also depend on the width of the footing.

The rock was cored for a vertical distance of 3.0 to 4.2 m.

From the recovered rock cores the bedrock is a massive, moderately closely jointed, Precambrian gneiss. It is generally sound, while in some areas (Boreholes FG1 and 2) the upper 0.2 m is moderately weathered.

In Boreholes FG1 and 2, the upper 0.2 m has a rock quality designation of 0 to 52%, indicating a very poor to fair quality, and a core recovery of 31 to 97%. Below this upper weathered zone in Boreholes FG1 and 2, and from the surface of the bedrock in the remaining boreholes, a rock quality designation ranging from 42 to 100% was measured indicating the rock to be poor to excellent quality, but generally good to excellent. The core recovery was 97 to 100%.

From the results of the boreholes and auger probes, the bedrock surface in the general area appears to be dipping down from west to east. It should be noted that due to undulations in the surface of the bedrock, which is not uncommon in Northern Ontario, the bedrock elevation in between and beyond the boreholes may vary considerably.

4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. No groundwater was observed in the overburden of any of the boreholes drilled, which was confirmed by the measured natural moisture contents of the recovered samples. It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and in response to major weather events.

5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided highway with an approximately 30 m wide median (the proposed northbound lanes will follow the alignment of existing Highway 11). The grade for Highway 11 will be lowered by about 8 m from the existing. The proposed bridge will carry the realigned Fern Glen Road over the proposed realigned northbound and southbound lanes of Highway 11 and the associated interchange ramps. It will be an approximately 70 m long, 2-lane (11 m wide), two span structure. The ground surface at the bridge site is generally level. In general the existing ground elevation is 342.5 to 340.5 m, while the grade beyond the bridge site rises to the west and drops to the east. The proposed grade of Highway 11 at the bridge site is approximately Elevation 334 m. The existing grades under the proposed bridge will therefore be lowered to build the highway, while the existing grades at the west and east abutments will be adjusted marginally (i.e. by up to about 1 to 2 m). The grade at the pier location will be cut by about 8.5 m below existing grade (i.e. to about Elevation 333.5 m). The proposed bridge elevation over the highway is approximately 342 m.

In general, below a surficial topsoil layer, and a gravelly sand deposit at the east abutment location, a generally compact to dense sand deposit was encountered to depths ranging from 2.3 to 15.3 m below existing grade. Underlying this cohesionless overburden is a Precambrian granodiorite bedrock. At the time of the investigation no groundwater table was encountered within the overburden.

5.1 FOUNDATIONS

5.1.1 Spread Footing Foundations

All footings can be founded on sound bedrock. For this purpose all loose or weathered rock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary.

Based on the findings of the boreholes and auger probes advanced in the area of the proposed foundation elements, for design purposes sound bedrock can be assumed (along the north-south axis of the centre-line of each foundation element) at the elevations noted below :

TABLE I

FOUNDATION LOCATION	REFERENCE BOREHOLE	APPROXIMATE DEPTH TO PRESUMED BEDROCK SURFACE (m)	PRESUMED BEDROCK SURFACE ELEVATION (m)
West Abutment North side	Auger Probe Nos. W4, 7, 10 and Borehole FG6	5 - 7 ±	337 - 335 ±
Centre	Auger Probe Nos. W3, 6 and 8	5 - 6 ±	337 - 336 ±
South side	Auger Probe Nos. W5, 9 and Borehole FG7	5.5 - 6.5 ±	336.5 - 335.5 ±
Central Pier North side	Auger Probe Nos. P4, 6, 9 and Borehole FG2	8 - 11 ±	334 - 331 ±
Centre	Auger Probe Nos. P1, 5 and 8	8 - 11 ±	334 - 331 ±
South side	Auger Probe Nos. P2, 10 and Borehole FG1	10 - 11 ±	332 - 331 ±
East Abutment North side	Auger Probe Nos. E2, 7 and 11	13 - 15 ±	328 - 327 ±
Centre	Auger Probe Nos. E4, 6, 10 and Borehole FG3	11 - 13 ±	330 - 328 ± *
South side	Auger Probe Nos. E5, 8 and Borehole FG4	10 ±	333 - 332 ±

* Note that Auger Probe No. E1, located at Station 10+030 (i.e. somewhat west) showed refusal at about Elevation 332.9 m.

It should also be noted that in between and beyond the borehole locations and auger probes, the bedrock surface may vary considerably, as surmized from variable auger probe refusal depths at the east abutment. Additionally, the auger probe refusal depths may be due to refusal on cobbles or boulders within the overburden and the actual bedrock surface may be lower than anticipated.

For design purposes the following O.H.B.D.C. bearing resistances may be used:

Factored Bearing Resistance at U.L.S. = 10,000 kPa
Bearing Resistance at S.L.S. will not govern

Bedrock would be prone to deterioration due to the opening of existing joints or fractures in the bedrock as a result of frost action. Provided that surface water is diverted away from the footings, frost protection need not be provided for footings placed on massive, sound bedrock, although for added protection an earth cover of at least 0.3 m is recommended. The surface of the earth protection should be clayey to minimize the infiltration of surface water or the protection could be provided by concrete. If however the bedrock is not massive and water can accumulate in the joints or fractures of the rock (thus causing deterioration of the founding medium by expansion due to freezing) then there may be a requirement to provide up to full frost protection (i.e. 1.8 m). For this purpose, the proposed bearing surface should be inspected by qualified engineering personnel. If the rock is not massive, then the excavation can be extended deeper until acceptable rock is found or to the full frost protection depth of 1.8 m, whichever comes first. Based on our experience in the general area, however, the depth of excavation into the bedrock for frost protection purposes seldom exceeds 0.6 m. Nevertheless, as mentioned before, the final decision regarding this should rest by the engineer in the field inspecting the exposed rock.

Sliding resistance can be provided by penetrating into the bedrock (i.e. keying-in and utilizing passive rock resistance), utilizing the sliding resistance between concrete and bedrock, shear in grouted dowels and/or rock anchors. For the evaluation of the sliding resistance of the foundation (O.H.B.D.C. 6-8.4.3) the ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as 30 degrees. If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Alternatively, the surface of the bedrock can be chiseled (i.e. roughened), increasing the ultimate angle of friction to 35 degrees. This, in our opinion, is likely to be the most cost effective method.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing sliding resistance, the factored rock/grout bond capacity at U.L.S. can be taken as 500 kPa and S.L.S. will not govern. The upper 0.2 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.5 m into sound rock. The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C., 3rd Edition.

Details of foundation conditions at each support location are discussed below.

5.1.1.1 West Abutment

The west abutment can be founded directly on suitable bedrock. From the probe results the rock surface appears to be sloping unfavourably towards the proposed Highway 11 cut face (i.e. in an easterly direction). In order to minimize this condition it is recommended that a relatively low foundation level of 336.5 m (or lower) be selected. This may involve some rock excavation but will reduce the effect of placing the foundation on unfavourably sloping rock surface. This recommended founding elevation may however have to be revised (i.e. somewhat lowered) after excavation, depending on actual conditions (e.g. if the rock surface is lower than the inferred elevations quoted in the report).

Where the rock surface is lower than the proposed founding level, all the overburden and shattered rock should be removed to the surface of acceptable rock. The grade can then be raised using mass concrete.

In order to ensure the stability at the west abutment, the footing should be located far enough from the face of any proposed rock cuts along Highway 11 to outside a plane defined by 1/2H:1V from the toe of the cut slope. In addition the outside edge of the footing should be no closer than 1.5 m to the face of the rock cut. The rock portion of the cut slope can be maintained at nearly vertical slopes (i.e. 1/4 H in 1V) but depending on the orientation of joints and fractures, rock bolting of the rock face may be required.

5.1.1.2 Central Pier

From the borehole and probe results the rock surface appears to be V-shaped with lower rock surface along the centre portion. In this case too the footing should be supported on the suitable bedrock. The suggested footing elevation is 332.0 m, which will involve removing any rock knobs that extend above this elevation and clearing of overburden and unsuitable surface rock where the surface of the rock extends below this elevation. After clearing of the overburden and unsuitably weathered rock in the low areas the grade can be raised to the footing elevation using mass concrete.

5.1.1.3 East Abutment

At the proposed east abutment location the recorded and inferred depths to the surface of the bedrock show a large variation, as indicated on the appropriate borehole log and probehole records. In general, the recorded values range from about Elevation 333 m on the south side dipping to about 328 m towards the north. As mentioned before, the abutment can be supported on the bedrock. For this purpose all the overburden and shattered or otherwise unsuitable rock should be removed, exposing the acceptable, clean bedrock surface. The grade can then be raised to the required footing elevation using mass concrete. The operation should ensure that the surface of the rock and/or mass concrete will be sufficiently clean and roughened to facilitate

proper bonding between rock & concrete or concrete & concrete.

If the possibility of greater than normally accepted differential settlements would be acceptable then consideration may be given to the option of supporting the east abutment on the overburden and on engineered fill consisting of Granular 'A' type material.

For this option, the suggested foundation elevation (i.e. underside footing elevation for the east abutment foundation) is $337.5 \pm$ m, based on the findings of Boreholes FG3 and 4. The soil beneath the proposed footing elevation should be removed to a depth of not less than 1.5 m (i.e. about Elevation $336.0 \pm$ m) within an area at least 1.5 m beyond the perimeter of the proposed footing. For example, for a footing measuring 12.0×5.0 m in plan, the size of the excavation at the bottom would be $5 + 1.5 + 1.5 = 8$ m by $12 + 1.5 + 1.5 = 15$ m plan. The sides of the excavation would be sloped not steeper than 1H:1V (flatter if necessary) and therefore at the proposed founding level, the size of the excavation would be 11 m by 18 m or greater. The sides of excavations (above the water table) should not be sloped steeper than 1H:1V but may require flatter side slopes, especially since vibrations induced during compaction may create instability. If excavations extend below the water table (or perched water table) or come close to it, flatter side slopes and dewatering will be required. This however is considered to be very unlikely.

When the excavation reaches the required depth, the subgrade should be evaluated and approved by a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. If necessary, the excavation may need to be deepened to the surface of a sufficiently competent soil. After its approval, the exposed subgrade at the base of the excavation may need to be compacted, if requested by the Geotechnical Engineer, to achieve a density of not less than about 98% of the Material's Standard Proctor Maximum Dry Density (SPMDD). The fill used to raise the grade inside the excavation should consist of Granular 'A' material placed when its moisture content is within $\pm 2\%$ of its optimum moisture content. It should be placed in layers not exceeding 200 mm in thickness and should be uniformly compacted to not less than 100% of its SPMDD.

A factored bearing resistance at U.L.S. of 900 kPa and a bearing resistance at S.L.S. equal to 350 kPa can be assigned to soil prepared in this manner. The serviceability condition is based on the premise that total settlements will not exceed 25 mm. However, since the pier will be supported directly on bedrock where settlements can be expected to be negligible the total settlement will translate into differential settlements between the pier and the east abutment. While the settlement of the east abutment can be expected to be completed rather rapidly due to the granular nature of the soil, nevertheless this aspect should be evaluated by the structural engineer. In addition, it should be noted that the rock surface at Borehole FG3 was recorded at Elevation 328.5 m while at Borehole FG4 it was recorded at Elevation 333.5 m (i.e. a difference of 5 m). In the probeholes drilled in the general east abutment area, the recorded refusal (i.e. probable rock surface) elevations ranged from a high Elevation of 334.5 m on the south side to a low Elevation of 327.4 m on the north side, with an elevation difference of about 7 m. This indicates a potential for greater settlements on the north side of the east abutment in comparison with the south side. This should also be considered in the design.

For frost protection, the footing should have a permanent earth cover of not less than 1.8 m.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C., 3rd Edition.

The unfactored horizontal resistance against sliding between concrete and Granular 'A' type material can be calculated using a friction angle of 35 degrees.

While this option is likely to be more cost effective, from geotechnical engineering point of view it is less acceptable, as it is undesirable to have one of the supports of the bridge founded on the overburden while the others are supported on bedrock (i.e. potential for increased differential settlements). Therefore, in our opinion, footings on bedrock are the preferred option.

5.1.2 Deep Foundations

Due to the shallow bedrock depths at the proposed foundation locations, deep foundations are not considered to be a viable alternative, except possibly for the east abutment. Here, too, footings on bedrock are the preferred option, but the deep foundations option is discussed below for the sake of completeness.

Because of the significant grade raise required using mass concrete at the east abutment location, consideration could be given to supporting the abutment on drilled and cast-in-place concrete piles (i.e. caisson foundations) extending into the bedrock. Caissons extended into the sufficiently sound bedrock immediately below the surficial weathered zone can be designed for a Factored Bearing Resistance at U.L.S. of up to 10,000 kPa and Bearing Resistance at S.L.S. will not govern. Because of the sloping rock surface, the installation of the caisson units may present problems as it will probably require churn drilling to sufficiently socket the caissons into the bedrock. The minimum socket depth should be 300 mm. As the rock surface can be expected to be sloping and as the socket depth given above is the minimum, the average depth of socket into the sufficiently sound rock in any one caisson unit will likely be greater than 300 mm. This may be difficult to achieve and will likely be an expensive procedure. Because of this it is our opinion that the use of caissons will unlikely be economical. If required, however, we can further discuss this alternative.

The use of driven steel piles is not feasible at the west abutment and central pier locations as the bedrock surface is high. Their use at the east abutment location is not recommended, owing to the uneven bedrock surface.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B'

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

Rock Fill

Unit Weight = 18 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition.

Foundations on bedrock will be unyielding and in that case the at-rest condition will govern the earth pressure.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice and as specified in OPSS 501.

Some rock will likely be excavated for the highway cut near the west abutment. If rock fill is used for backfill, special care is required to prevent damage to the retaining structures. The placement of the rock fill behind the abutments should be carried out in accordance with OPSD 3505.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and moderate to high appearance.

5.3 APPROACH EMBANKMENTS

The proposed grades indicate that up to about 8 m deep cuts will be required at the immediate vicinity of the abutments (e.g. forward slopes). Beyond the immediate approaches minor cuts and fill embankments are to be constructed.

Permanent cut slopes in the overburden above the water table will be stable at 2H:1V. For slope heights of more than 6 m, however, a 2 m wide mid-height bench (berm) is recommended. Cut slopes should be inspected after construction and where deemed necessary, measures such as granular blanket (sheeting) should be provided.

For both earth fill embankments and cut slopes in the overburden proper erosion measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572).

Rock cut slopes should be stable at nearly vertical faces (e.g. 1/4H:1V) providing that blasting is carefully controlled and unfavourable orientation of joints and fractures is not encountered. This aspect is further elaborated in the next section of this report.

As the proposed Fern Glen Road Elevation is about 343 to 340 m and the existing grade elevations at the approaches are approximately 342 to 341 m, up to about 2 m deep cuts and 1 m high fill embankments will have to be built. Based on the borehole results, the strength of the foundation materials is such that deep-seated failures are not anticipated. We recommend however that all organic soils, weak or otherwise unsuitable materials be removed as per MTO Standards before placing the fill.

The average thickness of the unsuitable soils under fill embankments to be built can be assumed to be about 0.1 m to 0.15 m. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of qualified personnel. Permanent earth fill embankments should be stable at 2H:1V slopes.

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, negligible settlement of the foundation materials is anticipated.

The materials used for the construction of the embankment fills should consist of acceptable earth fill (e.g. Select Subgrade Materials - OPSS 1010). The majority of the clean, inorganic in-situ materials (i.e. sands) are considered to be suitable for this purpose. The fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density as per OPSS 501. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. The settlement of the embankment fills prepared as described above should be negligible.

5.4 CONSTRUCTION COMMENTS

No groundwater was encountered in the boreholes and, based on this, no problems with groundwater seepage are anticipated at the site. Any surface water seepage, if necessary, can easily be handled by gravity drainage and pumping from open sumps.

All excavations should be carried out in accordance with the Occupational Health & Safety Acts of Ontario, including regulations for Construction Projects and Regulations for Mines and Mining Plants (in bedrock).

Excavation of the bedrock by mechanical methods such as hoe-ramming or ripping will probably only be feasible within the uppermost weathered and fractured rock. In our opinion, in most cases blasting will be necessary. Blasting should however be carried out in a manner to minimize damage to the founding bedrock and permanent rock face. Where blasting is required, controlled perimeter (line) blasting (or a similarly acceptable method) is recommended in order to provide a neat excavation line, minimize over break, minimize face instabilities and long term maintenance problems.

Temporary rock slopes should be stable at near vertical faces, but adequate precaution should be taken to protect workers from spalling rock. The stability of rock faces will also depend on the inclination of planes of weakness in the rock mass.

To reduce the excavation depths and to facilitate the construction of spread footing foundations it would be advisable to carry out the cut for Highway 11 first. We recommend that all blasting for rock cuts be completed prior to the construction of the footings.

All rock excavations, foundation bases and bearing surfaces should be inspected and approved by a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. The rock surface which will receive the foundation and/or mass concrete should be properly cleared of all overburden, debris and shattered, unsuitable rock in order to provide a suitable bond between the concrete and the rock surface. Similarly the surface of the mass concrete should be properly cleaned to achieve a good bond with the foundation concrete.

6.0 CLOSURE

We recommend that once the details of the structure are finalized, our recommendations be reviewed for their specific applicability.

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

Sincerely,


Andrew Drevininkas, P. Eng.

AD/dee




Z.S. Ozden, P. Eng.



APPENDIX A

AGRA

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX B

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	WEST ABUTMENT				
W1	9+960	Centerline	342.0 ±	4.5	337.5 ±
W2	9+960	6.0 RT of Centerline	342.0 ±	4.9	337.1 ±
W3	9+962	Centerline	342.0 ±	4.9	337.1 ±
W4	9+962	6.0 LT of Centerline	342.0 ±	4.5	337.5 ±
W5	9+962	6.0 RT of Centerline	342.0 ±	5.2	336.8 ±
W6	9+965	Centerline	342.0 ±	5.3	336.7 ±
W7	9+965	6.0 LT of Centerline	342.0 ±	5.5	336.5 ±
W8	9+968	Centerline	342.0 ±	5.8	336.2 ±
W9	9+968	6.0 RT of Centerline	342.0 ±	5.5	336.5 ±
W10	9+968	6.0 LT of Centerline	342.0 ±	6.7	335.3 ±

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	PIER				
P1	9+997	Centerline	342.0 ±	7.6	334.4 ±
P2	9+997	8.0 RT of Centerline	341.7	10.3	331.4 ±
P3	9+997	12.1 RT of Centerline	342.0 ±	9.9	332.1 ±
P4	9+997	6.0 LT of Centerline	342.0 ±	7.7	334.3 ±
P5	10+000	Centerline	342.0 ±	10.8	331.2 ±
P6	10+000	6.0 LT of Centerline	342.0 ±	10.9	331.1 ±
P7	10+000	12.2 RT of Centerline	342.0 ±	8.4	333.6 ±
P8	10+003	Centerline	342.0 ±	10.0	332.0 ±
P9	10+003	6.0 LT of Centerline	342.1	9.5	332.6 ±
P10	10+003	6.0 RT of Centerline	342.0 ±	9.9	332.1 ±
P11	10+003	9.0 RT of Centerline	342.0 ±	10.6	331.4 ±
P12	10+003	12.2 RT of Centerline	342.0 ±	10.0	332.0 ±

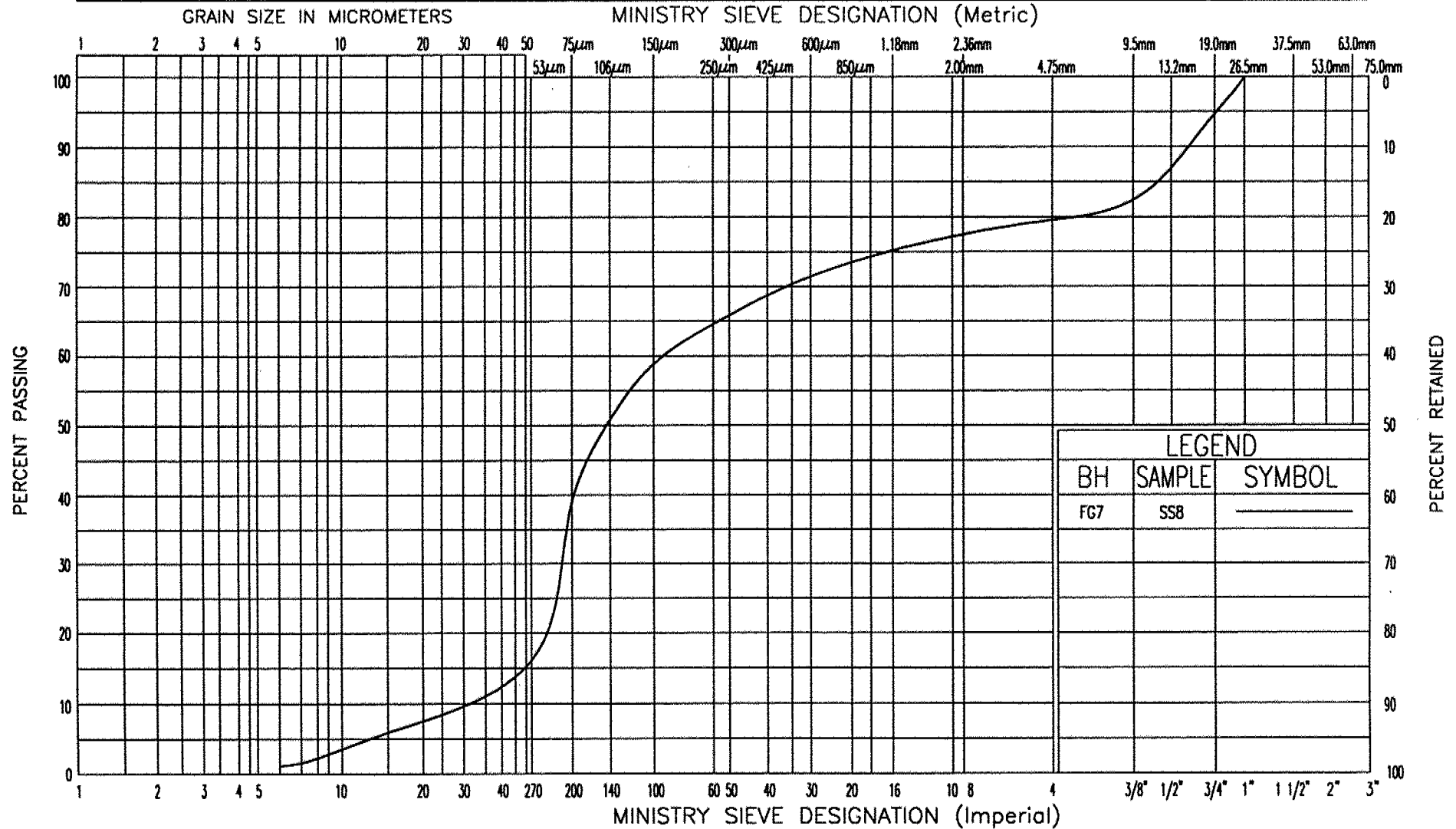
PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	EAST ABUTMENT				
E1	10+030	Centerline	342.0 ±	9.1	332.9 ±
E2	10+030	7.0 LT of Centerline	342.0 ±	14.6	327.4 ±
E3	10+031	7.0 RT of Centerline	342.0 ±	7.5	334.5 ±
E4	10+033	2.0 LT of Centerline	341.3	11.6	329.7 ±
E5	10+034	4.0 RT of Centerline	341.0	8.8	332.2 ±
E6	10+034	1.0 LT of Centerline	341.0	11.4	329.6 ±
E7	10+035	7.0 LT of Centerline	341.3	13.4	327.9 ±
E8	10+037	4.0 RT of Centerline	341.3	9.7	331.6 ±
E9	10+037	14.0 RT of Centerline	340.9	8.7	332.2 ±
E10	10+037	Centerline	341.7	11.7	330.0 ±
E11	10+037	7.0 LT of Centerline	341.9	13.9	328.0 ±
E12	10+039	14.0 RT of Centerline	342.0 ±	6.9	335.1 ±
E13	10+040	7.0 LT of Centerline	342.6	13.9	328.7 ±
E14	10+040	7.0 RT of Centerline	341.5	8.8	332.7 ±
E15	10+041	3.5 LT of Centerline	342.6	15.3	327.3 ±
E16	10+042	14.0 RT of Centerline	341.5	8.6	332.9 ±

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
E17	10+042	1.0 LT of Centerline	342.4	12.4	330.0 ±
E18	10+043	6.0 RT of Centerline	342.1	10.6	331.5 ±

FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

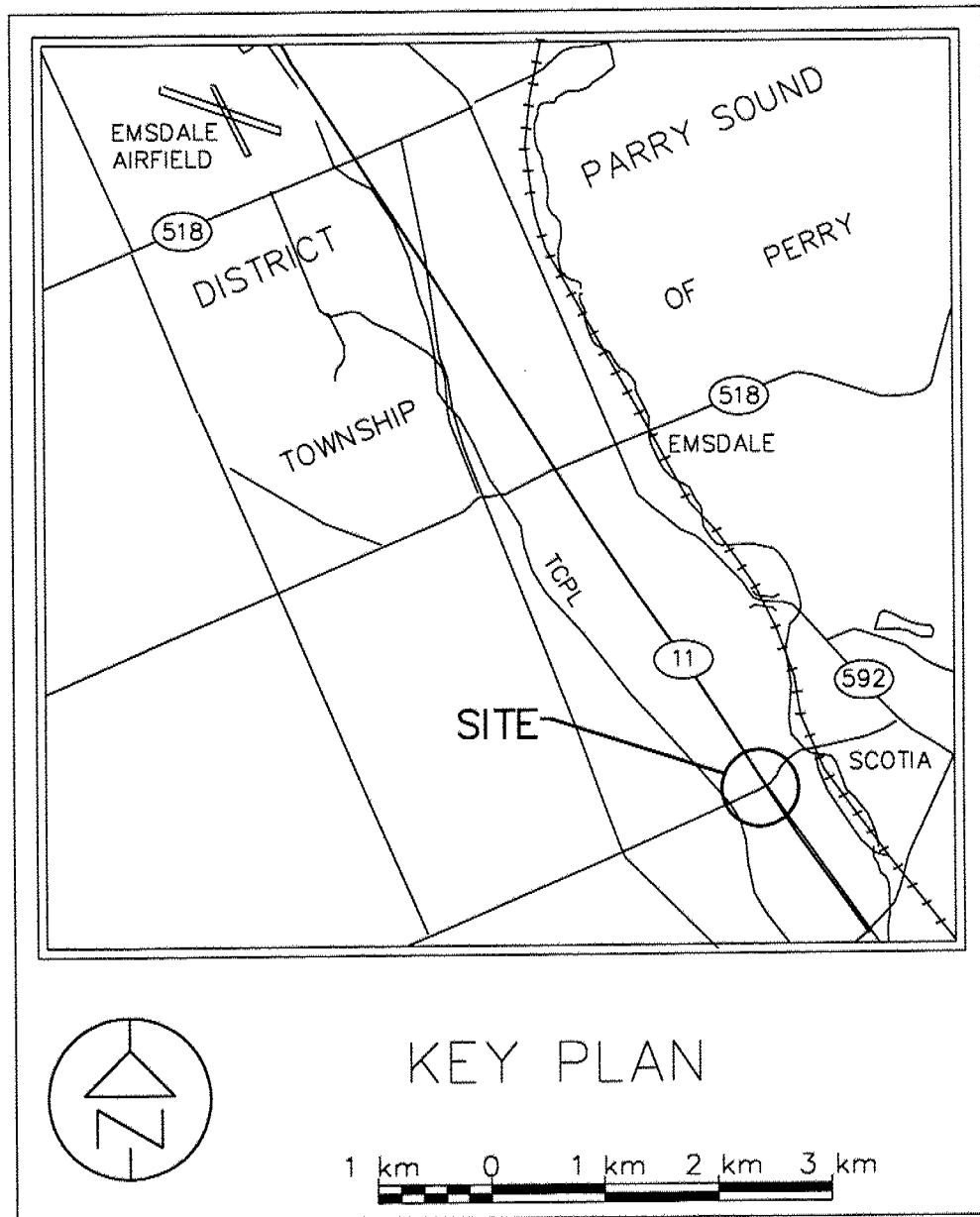
CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION
SANDY SILT SOME GRAVEL

FIG No 3
W P 466-93-00

ENCLOSURES



FERN GLEN ROAD UNDERPASS
KEY PLAN

Dwg. No 1



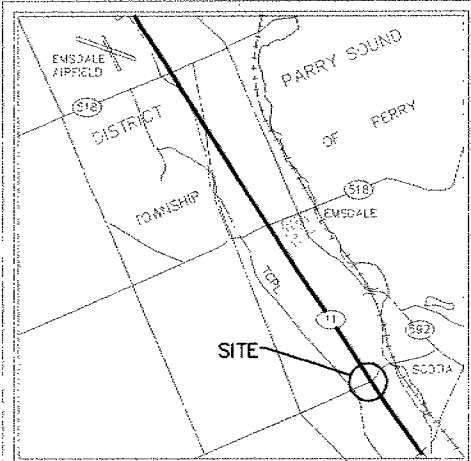


CONT. No.
W.P. No. 467-93-01

SHEET

FERN GLEN ROAD UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA

AGRA Earth & Environmental Ltd.



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation Dec. 98 - May. 99
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
FG1	341.8	5 041 311	319 924
FG2	342.2	5 041 321	319 917
FG3	341.0	5 041 331	319 955
FG4	340.8	5 041 323	319 959
FG5	342.0	5 041 340	319 979
FG6	342.3	5 041 306	319 884
FG7	341.8	5 041 296	319 895
FG8	342.5	5 041 295	319 869

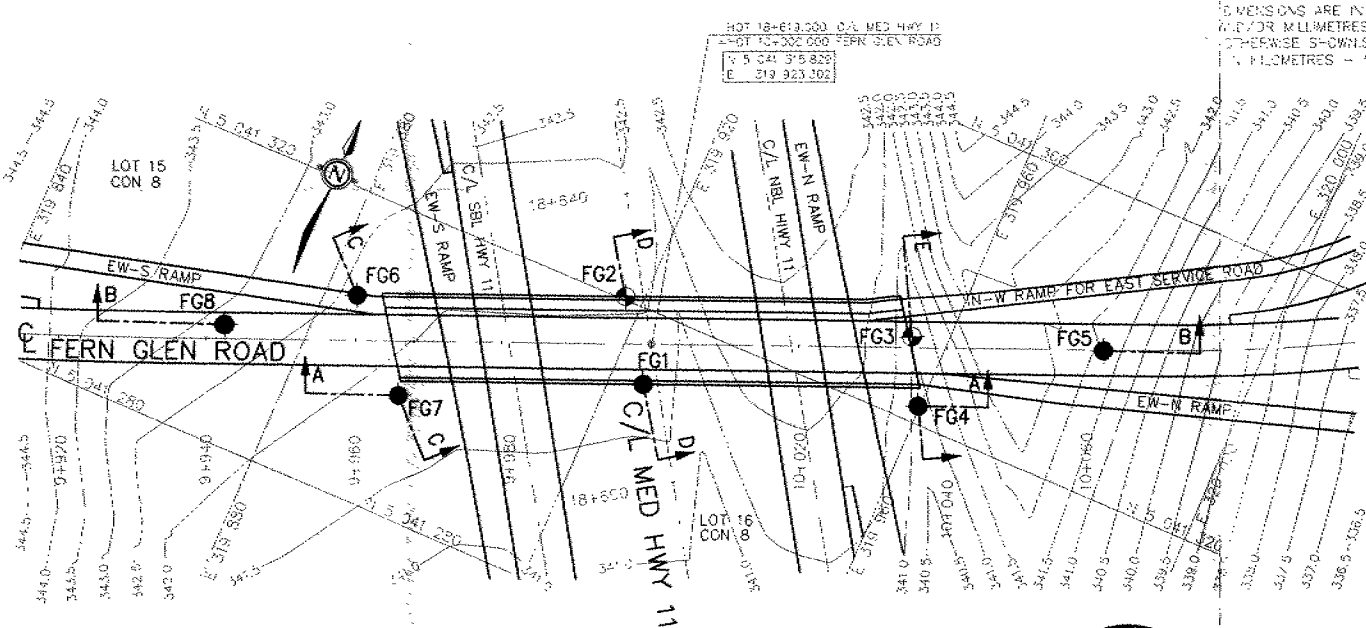
NOTE-
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Material's Office. Downview, information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 30.2.01 of GPS Gen.Cond.

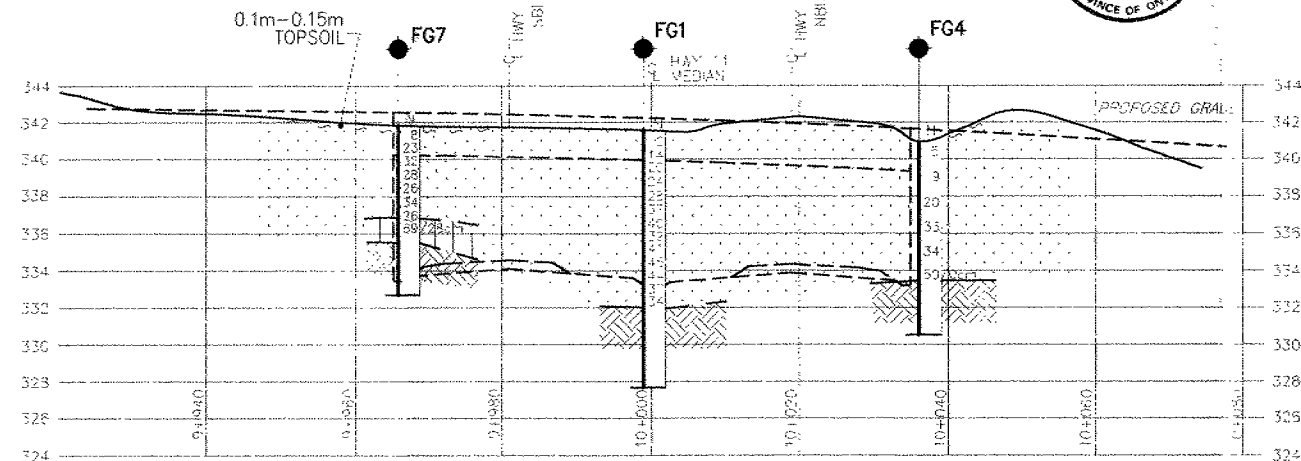
REV	DATE	BY	DESCRIPTION

Hwy No 11	SLWD TO	CHECKED AD	DATE June, 1999	DIST 52-HUNTSVILLE
DRAWN MA	CHECKED			SITE 44-381
				DWG 2

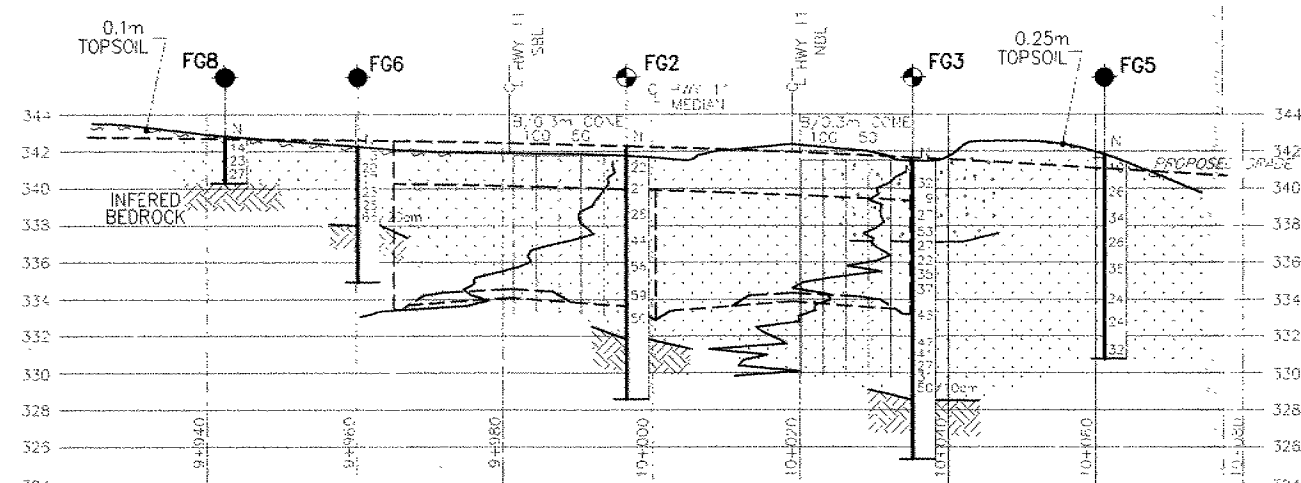
METRIC
DIMENSIONS ARE IN METRES
UNLESS OTHERWISE SPECIFIED
OTHERWISE DIMENSIONS ARE IN METRES



PLAN
5m 0 5m

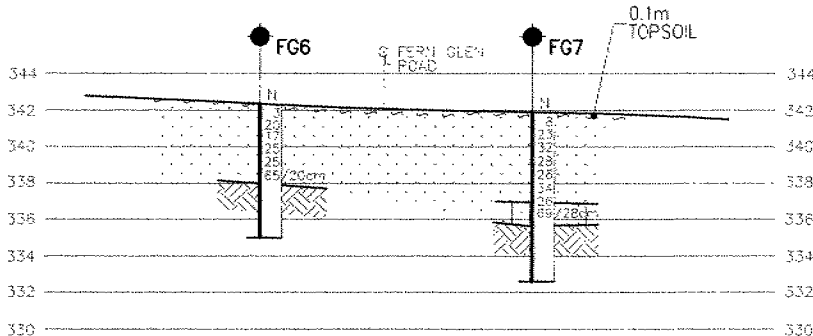


SECTION A-A

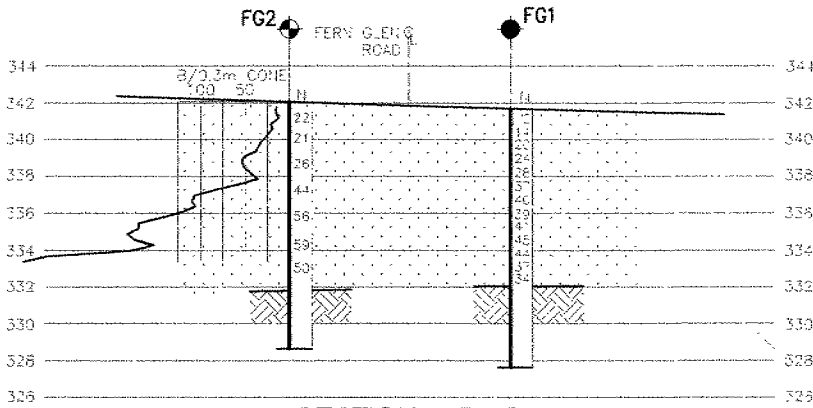


SECTION B-B

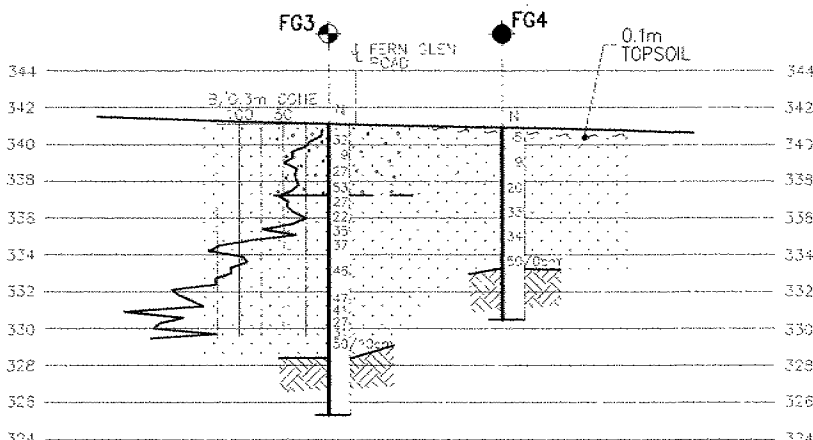
5m 0 5m HOR
2m 0 2m VER



SECTION C-C



SECTION D-D



SECTION E-E

2m 0 2m

SOIL STRATIGRAPHY LEGEND

	SAND FINE to MEDIUM Very Loose to Very Dense		SANDY SILT SOME GRAVEL Compact to Dense
	SAND to GRAVELLY SAND Loose to Compact		gneiss BEDROCK

REF: Hwy 11 Bridge Site Plan
Cwg. by: JTD, Jan. 1999

RECORD OF BOREHOLE No FG1

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041310.8 E 319924.2 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 16 December 1998 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					w _p w w _L				
341.8							20 40 60 80 100	20 40 60 80 100	10 20 30						GR SA SI CL		
0.0	brown SAND fine to medium dry	V.loose	1	SS	2											Station 9 + 999 5.0 Rt Fern Glen Rd. C/L	
			2	SS	14												
		compact	3	SS	20												
			4	SS	24												
		dense	5	SS	28												4 93 (3)
			6	SS	37												
			7	SS	40												0 96 (4)
			8	SS	39												

RECORD OF BOREHOLE No FG2

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041320.8 E 319917.2 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 16 February 1999 CHECKED BY ZSO


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
342.2							20 40 60 80 100	20 40 60 80 100	10 20 30					GR SA SI CL
0.0	frozen		1	SS	22									Station 9 + 996 6.9 Lt Fern Glen Rd. C/L
	brown SAND fine to medium dry		2	SS	21									
	compact		3	SS	26									9 86 (5)
	dense		4	SS	44									
	very dense		5	SS	56									4 91 (5)
			6	SS	59									
			7	SS	50									0 95 (5)
331.8	moderately weathered		8	RC										RC8: REC=97% RQD=52%
10.4														
	GNEISS BEDROCK		9	RC										RC9: REC=100% RQD=93%
	massive, moderately closely jointed													
			10	RC										RC10: REC=99% RQD=83%
328.6														
13.6	END OF BOREHOLE													
	WL on completion: none													

RECORD OF BOREHOLE No FG4

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041323.1 E 319959.4 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
 DATUM Geodetic DATE 11 May 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
340.8	0.15m TOPSOIL						20	40	60	80	100							
0.0	brown SAND fine to medium dry		1	SS	5											Station 10 + 035 7.0 Rt Fern Glen Rd. C/L		
			v. loose															
			loose															
				2	SS	9												
			compact															
				3	SS	20												
			dense															
			4	SS	33													
			5	SS	34													

RECORD OF BOREHOLE No FG5

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041340.1 E 319979.3 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 9 February 1999 CHECKED BY ZSO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)									
							20	40	60	80	100	20	40	60	80	100	10	20	30			
342.0	0.25m TOPSOIL (frozen)		1	SS	10																	Station 10 + 061 1.0 Lt Fern Glen Rd. C/L
0.0																						
			2	SS	26																	
			3	SS	34																	1 95 (4)
			4	SS	26																	
			5	SS	35																	0 95 (5)
			6	SS	24																	
			7	SS	24																	
			8	SS	32																	
330.8																						
11.3	END of BOREHOLE WL on completion: none																					Move borehole 2m Lt. auger refusal @ 0.1m. Move borehole 1m Lt. auger refusal @ 0.1m.

RECORD OF BOREHOLE No FG6

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041306.3 E 319883.9 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 14 April 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
WATER CONTENT (%)					10 20 30												
342.3	0.1m TOPSOIL		1	SS	3		342									Station 9 + 960 6.0 Lt Fern Glen Rd. C/L	
0.0	very loose		2	SS	20*		341									SS2: No recovery	
	compact		3	SS	17		340									2 89 (9)	
	brown SAND fine to medium dry		4	SS	25		339									Auger Refusal @ 4.1m Advance borehole by rock coring	
			5	SS	25		338										
	cobbles		6	SS	65/20		337										
338.0			7	RC			336									RC7: REC=100% RQD=100%	
4.3	GNEISS BEDROCK massive, moderately closely jointed		8	RC			335									RC8: REC=100% RQD=99%	
			9	RC												RC9: REC=100% RQD=75%	
			10	RC												RC10: REC=100% RQD=83%	
335.0																	
7.3	END of BOREHOLE WL on completion: none																

RECORD OF BOREHOLE No FG7

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041296.1 E 319894.6 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
 DATUM Geodetic DATE 14 April 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	x LAB VANE							
341.8							20	40	60	80	100							
0.0	0.1m TOPSOIL															GR SA SI CL		
		loose	1	SS	8											Station 9 + 965		
			2	SS	23											7.5 Rt Fern Glen		
			3	SS	32											Rd. C/L		
			4	SS	28													
			5	SS	26											4 92 (4)		
			6	SS	34													
			7	SS	26											Auger Refusal @		
336.8			8	SS	69/28											5.8m. Advance		
5.0	grey SANDY SILT some gravel compact to dense, moist															borehole by		
		boulder														triconing.		
335.6		boulder														20 40 40 0		
6.2			9	RC												RC9:		
	GNEISS BEDROCK		10	RC												REC=100%		
	massive, moderately closely jointed		11	RC												RQD=95%		
			12	RC												RC10:		
			13	RC												REC=100%		
332.6																RQD=91%		
9.2	END of BOREHOLE															RC11:		
	WL on completion: none															REC=100%		
																RQD=76%		
																RC12:		
																REC=100%		
																RQD=100%		
																RC13:		
																REC=100%		
																RQD=100%		

RECORD OF BOREHOLE No FG8

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041295.4 E 319869.2 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 14 April 1999 CHECKED BY ZSO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100					
342.5																
0.0	0.1m TOPSOIL		1	SS	14											
	brown SAND fine to medium compact, dry		2	SS	23											
			3	SS	27											
340.2																
2.3	END of BOREHOLE															
	AUGER REFUSAL on probable Bedrock															
	WL on completion: none															

**FOUNDATION INVESTIGATION REPORT
FOR
FERN GLEN ROAD UNDERPASS
STRUCTURE SITE NO. 44-391
DISTRICT 52, HUNTSVILLE
W.P. 467-93-01**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**September 1999
TT98820D**

August 31, 1999.
Ref. No.: TT98820D

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.

Dear Sir:

**Re: FOUNDATION INVESTIGATION REPORT
FOR
FERN GLEN ROAD UNDERPASS
STRUCTURE SITE NO. 44-391
DISTRICT 52, HUNTSVILLE
W.P. 467-93-01**

We take pleasure in enclosing seven (7) copies of our Foundation Investigation Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

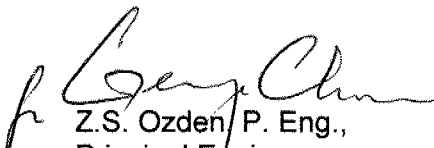

Z.S. Ozden, P. Eng.,
Principal Engineer.
ZSO/dee

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1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Delcan Corporation to conduct a foundation investigation at the site of a proposed bridge that will carry the proposed realigned Fern Glen Road over the proposed northbound and southbound lanes of Highway 11 and associated interchange ramps. The site is located near the Village of Emsdale, about 50 m south of the existing Fern Glen Road intersection with Highway 11, in the Township of Perry, Lot 16, Concession 8 in MTO District 52 - Huntsville (see Key Plan, Drawing No. 1). The proposed bridge will be an approximately 70 m long, 2-lane, two span structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the foundation design of the proposed structure and associated approach fills/cuts.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located about 50 m south of the intersection of Highway 11 and existing Fern Glen Road, near the Village of Emsdale. The ground elevation in the general area of the proposed bridge site is fairly level, ranging in Elevation from about 342.5 to 340.5 m. The ground rises mildly west of the proposed bridge site, and drops to the east. The surrounding area is sparsely wooded with sand exposed along highway cuts at the existing Highway 11 intersection.

Based on available geologic information, the site is in an area of glaciolacustrine sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin depositing sands, silts and clays in the low lying areas.

Published information show that the bedrock can be expected to be composed of strongly foliated, gneissic to migmatic rocks which form part of the Central Gneiss Belt of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out during the period of December 16, 1998, February 7 to 17, April 14, and May 10 to 11, 1999, and consisted of drilling and sampling eight boreholes (Borehole Nos. FG1 through FG8) to depths of 2.3 to 15.7 m below the ground surface, forty auger probes and two dynamic cone penetration tests.

.../...

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The delays in the field investigation were primarily due to permission to enter difficulties.

The plan locations of the boreholes and cone tests, along with stratigraphic sections are shown on Drawing No. 2. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, are presented on the Borehole Log Sheets. The results of the auger probes are tabulated in Appendix B.

The boreholes were advanced using solid and hollow stem continuous flight augers with a track-mounted power auger drilling rig (CME 75) owned and operated by Canadian Soil Drilling Inc., under the full-time supervision of a soils engineer from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

In addition, dynamic cone penetration tests were performed adjacent to two of the boreholes. This test consists of driving a 60° point, 50 mm diameter cone attached to the drill rod continuously into the undisturbed ground with a driving energy of 475 J (63.5 kg hammer falling freely a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Coring of the bedrock was effected by diamond drilling methods, using an NXL size core barrel.

The borehole locations were established in the field by our engineering staff, in relation to the already staked out proposed centre-line of Fern Glen Road (by Dearden and Stanton Limited). The borehole geodetic elevations and co-ordinates were subsequently taken by surveyors from Dearden and Stanton Limited and supplied to us.

The soil samples and rock cores were shipped to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also on Figure Nos. 1, 2 and 3.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. The boreholes were grouted on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at eight borehole locations (Borehole Nos. FG1 to FG8), and were inferred at the locations of two dynamic cone penetration tests and forty auger probes. The locations of the boreholes and cone penetration tests are shown on the Plan and Profile Drawing No. 2 and are also indicated on the individual Borehole Log Sheets. Cross sections of inferred subsurface stratigraphy are given on Drawing No. 2. The results of the auger probes are tabulated in Appendix B.

The ground surface at the proposed site is relatively level and the ground elevation at the proposed bridge location generally ranges from about 342.5 to 340.5 m.

Below a surficial topsoil layer, the overburden generally consists of a sand deposit to depths ranging from 2.3 to 15.3 m below existing grade. At Borehole FG3 overlying the sand deposit is a sand to gravelly sand layer which extends to a depth of 3.8 m below the ground surface while in Borehole FG7 the sand is underlain by a 1.2 m thick sandy silt layer immediately above the bedrock. Below the cohesionless overburden is the Precambrian bedrock. At the time of the investigation no groundwater was encountered within the overburden.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets. Descriptions of various strata are given in this section of the report.

4.1 TOPSOIL/FILL

A surficial topsoil layer, ranging in thickness from 0.1 to 0.25 m, was encountered at Boreholes FG4, 5, 6, 7 and 8. Measured natural moisture contents range from 12 to 18%.

At Borehole FG3, a 0.15 m thick layer of gravelly sand fill was encountered.

In our experience the thickness of topsoil and fill deposits frequently varies in between and beyond the borehole locations. In addition, at the time of our investigation the ground near the surface was frozen; therefore the soil conditions within the upper several decimeters could not be accurately determined and the descriptions given for this upper zone should be considered approximate only.

4.2 SAND TO GRAVELLY SAND

Underlying the surficial topsoil in Borehole FG3, a brown sand to gravelly sand layer was encountered to a depth of 3.8 m (Elevation 337.2 m) below existing grade. A grain size distribution analysis was conducted on a sample from this cohesionless (granular) deposit, and the resulting curve is presented in Figure No. 1. The analysis indicates:

Gravel:	29%
Sand:	68%
Silt and Clay:	3%

Measured 'N' values within this deposit range from 9 to 27 blows/0.3 m indicating a loose to compact condition. Higher values of 32 and 53 were also measured, but we believe these high values are due to the depth of frost and oversize gravel particles and are unreliable.

Measured natural moisture contents range from 3 to 5%.

4.3 SAND

Below the surficial topsoil, (gravelly sand in Borehole FG3), the predominant overburden in all of the boreholes consists of a brown sand deposit. At the borehole locations this granular (i.e. cohesionless) deposit extends to depths ranging between 2.3 m or Elevation 340.2 m (Borehole FG8) and 12.5 m or Elevation 328.5 m (Borehole FG3), generally to the surface of the bedrock. The grain size distribution of the material is generally in the fine to medium sand range. Grain size distribution analyses were conducted on twelve samples from the material and the range of particle sizes are presented in an envelope form in Figure No. 2. The analyses indicate:

Gravel:	0 - 9%
Sand:	86 - 98%
Silt and Clay:	2 - 9%

Measured 'N'-values within the surficial $2 \pm$ m, range from 2 to 32 blows/0.3 m, indicating surficially a very loose to dense condition (generally loose to compact). Below this depth, measured 'N'-values in this deposit generally range from 20 to 59 blows/0.3 m indicating a compact to very dense condition, but generally compact to dense. Measured natural moisture contents range from 1 to 10%.

The presence of occasional cobbles and boulders was noted immediately below the ground surface in Borehole FG5 where auger refusal was encountered. Auger refusal was also encountered due to cobbles (or boulders) immediately above the bedrock surface in Borehole FG6 and rock coring was used to advance through the cobbles (from 4.1 to 4.3 m below ground surface).

4.4 SANDY SILT

Underlying the sand deposit and overlying the bedrock in Borehole FG7, a grey sandy silt layer was encountered from a depth of about 5.0 m to 6.2 m below ground surface. This is a cohesionless material and contains some gravel size particles. One grain size distribution analysis was conducted within this deposit and the results are presented in Figure No. 3. The analysis indicated:

Gravel:	20%
Sand:	40%
Silt:	40%
Clay:	0%

Because of the presence of oversized particles, the recorded 'N'-values within this layer are generally considered to be unreliable but from the overall results and resistance to augering while drilling the material appears to be compact to dense. Measured natural moisture contents are 12 and 17%.

Within this deposit auger refusal was obtained immediately above the bedrock surface. In order to advance the borehole through cobbles and boulders, triconing had to be resorted to from 5.8 to 6.2 m below ground surface (i.e. to the surface of the bedrock).

4.5 BEDROCK

Below the overburden soils, bedrock was encountered and cored to obtain NXL size cores in Boreholes FG1, 2, 3, 4, 6 and 7 at depths of 4.3 to 12.5 m below existing ground surface (Elevations 338.0 to 328.5 m).

Auger probes were advanced at the site to attempt to delineate the undulating bedrock surface. Auger refusal on probable bedrock was encountered in Borehole FG8 and in the forty auger probes advanced at the site. The probe results are given in Appendix B. The results show that the recorded refusal depths range from 2.3 m or Elevation 340.2 m in Borehole FG8 (drilled for the west approach) to 15.3 m (Elevation 327.3± m) at a probehole put down 3.5 m left of proposed centre-line at Station 10+041 near (i.e. to the east) the proposed east abutment location. It is believed that while in most cases the refusals represent the depths to the surface of the bedrock, in some cases they may be due to cobbles and boulders in the overburden (probably immediately above the bedrock elevation).

From the results it can be surmized that in the general area of the proposed west abutment location the presumed bedrock surface elevation generally ranges from 337.5 to 336.0± m, dipping down from west to east. In the general area of the proposed central pier location it ranges generally from about Elevation 334 to 331 m, while in the area of the east abutment location a more complex picture emerges with rock elevations ranging from about 334 to 327.5± m. Here the rock surface appears to dip from about Elevation 334 m in the south-west area of the proposed abutment location to about 327.5± m towards the north and north-east of the abutment location.

The depth to the surface of the bedrock below the footprint of each foundation element will also depend on the width of the footing.

The rock was cored for a vertical distance of 3.0 to 4.2 m.

From the recovered rock cores the bedrock is a massive, moderately closely jointed, Precambrian gneiss. It is generally sound, while in some areas (Boreholes FG1 and 2) the upper 0.2 m is moderately weathered.

In Boreholes FG1 and 2, the upper 0.2 m has a rock quality designation of 0 to 52%, indicating a very poor to fair quality, and a core recovery of 31 to 97%. Below this upper weathered zone in Boreholes FG1 and 2, and from the surface of the bedrock in the remaining boreholes, a rock quality designation ranging from 42 to 100% was measured indicating the rock to be poor to excellent quality, but generally good to excellent. The core recovery was 97 to 100%.

From the results of the boreholes and auger probes, the bedrock surface in the general area appears to be dipping down from west to east. It should be noted that due to undulations in the surface of the bedrock, which is not uncommon in Northern Ontario, the bedrock elevation in between and beyond the boreholes may vary considerably.

4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. No groundwater was observed in the overburden of any of the boreholes drilled, which was confirmed by the measured natural moisture contents of the recovered samples. It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and in response to major weather events.

5.0 CLOSURE

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

Sincerely,



Andrew Drevininkas, P. Eng.

AD/dee



Z.S. Ozden, P. Eng.



APPENDIX A

AGRA
LIMITATIONS OF REPORT

The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX B

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	WEST ABUTMENT				
W1	9+960	Centerline	342.0 ±	4.5	337.5 ±
W2	9+960	6.0 RT of Centerline	342.0 ±	4.9	337.1 ±
W3	9+962	Centerline	342.0 ±	4.9	337.1 ±
W4	9+962	6.0 LT of Centerline	342.0 ±	4.5	337.5 ±
W5	9+962	6.0 RT of Centerline	342.0 ±	5.2	336.8 ±
W6	9+965	Centerline	342.0 ±	5.3	336.7 ±
W7	9+965	6.0 LT of Centerline	342.0 ±	5.5	336.5 ±
W8	9+968	Centerline	342.0 ±	5.8	336.2 ±
W9	9+968	6.0 RT of Centerline	342.0 ±	5.5	336.5 ±
W10	9+968	6.0 LT of Centerline	342.0 ±	6.7	335.3 ±

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	PIER				
P1	9+997	Centerline	342.0 ±	7.6	334.4 ±
P2	9+997	8.0 RT of Centerline	341.7	10.3	331.4 ±
P3	9+997	12.1 RT of Centerline	342.0 ±	9.9	332.1 ±
P4	9+997	6.0 LT of Centerline	342.0 ±	7.7	334.3 ±
P5	10+000	Centerline	342.0 ±	10.8	331.2 ±
P6	10+000	6.0 LT of Centerline	342.0 ±	10.9	331.1 ±
P7	10+000	12.2 RT of Centerline	342.0 ±	8.4	333.6 ±
P8	10+003	Centerline	342.0 ±	10.0	332.0 ±
P9	10+003	6.0 LT of Centerline	342.1	9.5	332.6 ±
P10	10+003	6.0 RT of Centerline	342.0 ±	9.9	332.1 ±
P11	10+003	9.0 RT of Centerline	342.0 ±	10.6	331.4 ±
P12	10+003	12.2 RT of Centerline	342.0 ±	10.0	332.0 ±

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
	EAST ABUTMENT				
E1	10+030	Centerline	342.0 ±	9.1	332.9 ±
E2	10+030	7.0 LT of Centerline	342.0 ±	14.6	327.4 ±
E3	10+031	7.0 RT of Centerline	342.0 ±	7.5	334.5 ±
E4	10+033	2.0 LT of Centerline	341.3	11.6	329.7 ±
E5	10+034	4.0 RT of Centerline	341.0	8.8	332.2 ±
E6	10+034	1.0 LT of Centerline	341.0	11.4	329.6 ±
E7	10+035	7.0 LT of Centerline	341.3	13.4	327.9 ±
E8	10+037	4.0 RT of Centerline	341.3	9.7	331.6 ±
E9	10+037	14.0 RT of Centerline	340.9	8.7	332.2 ±
E10	10+037	Centerline	341.7	11.7	330.0 ±
E11	10+037	7.0 LT of Centerline	341.9	13.9	328.0 ±
E12	10+039	14.0 RT of Centerline	342.0 ±	6.9	335.1 ±
E13	10+040	7.0 LT of Centerline	342.6	13.9	328.7 ±
E14	10+040	7.0 RT of Centerline	341.5	8.8	332.7 ±
E15	10+041	3.5 LT of Centerline	342.6	15.3	327.3 ±
E16	10+042	14.0 RT of Centerline	341.5	8.6	332.9 ±

PROBE NO.	STATION	OFFSETS	APPROXIMATE GROUND SURFACE ELEVATION (m)	DEPTH OF AUGER REFUSAL (m)	APPROXIMATE AUGER REFUSAL ELEVATION (m)
E17	10+042	1.0 LT of Centerline	342.4	12.4	330.0 ±
E18	10+043	6.0 RT of Centerline	342.1	10.6	331.5 ±

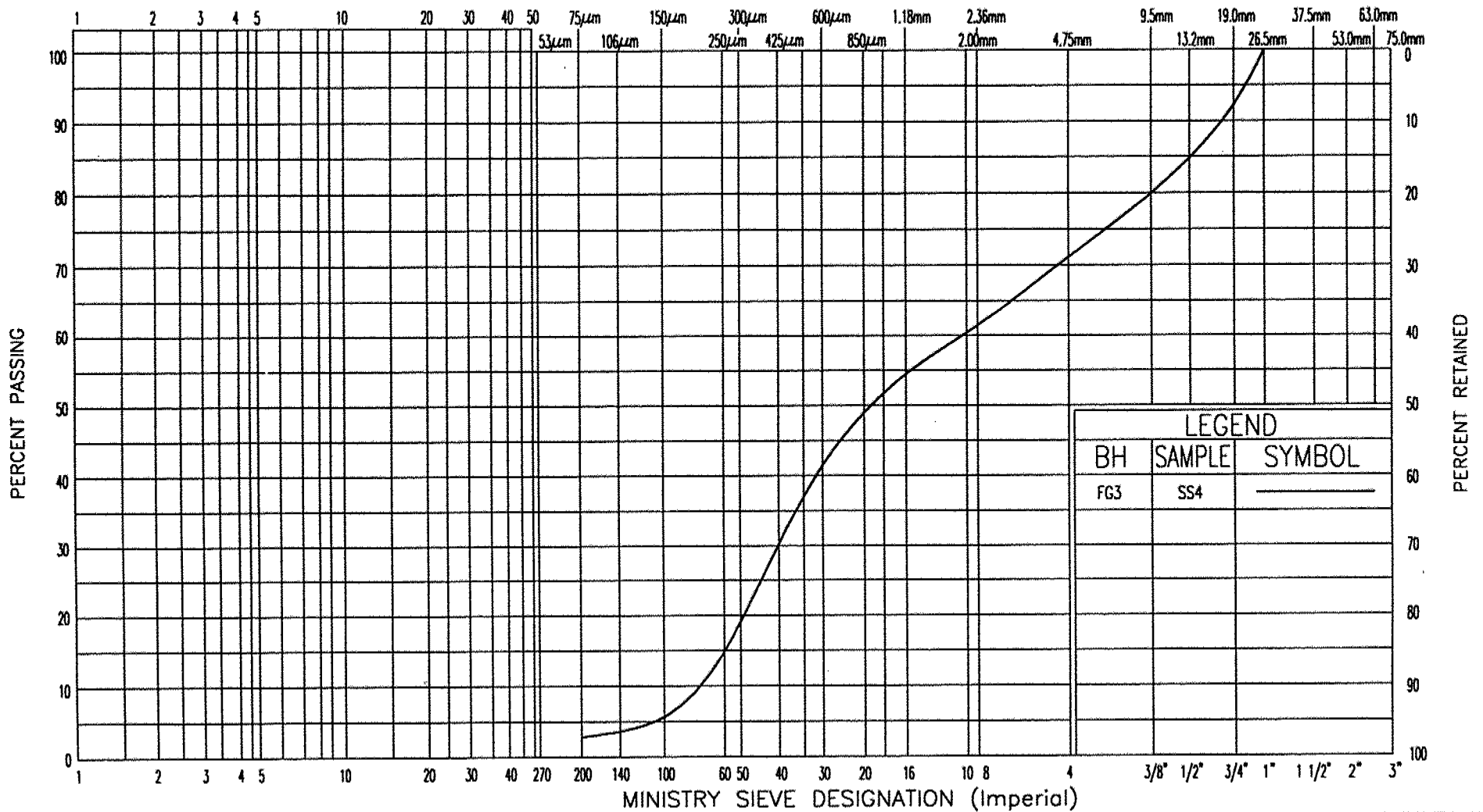
FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION
SAND to GRAVELLY SAND

FIG No 1

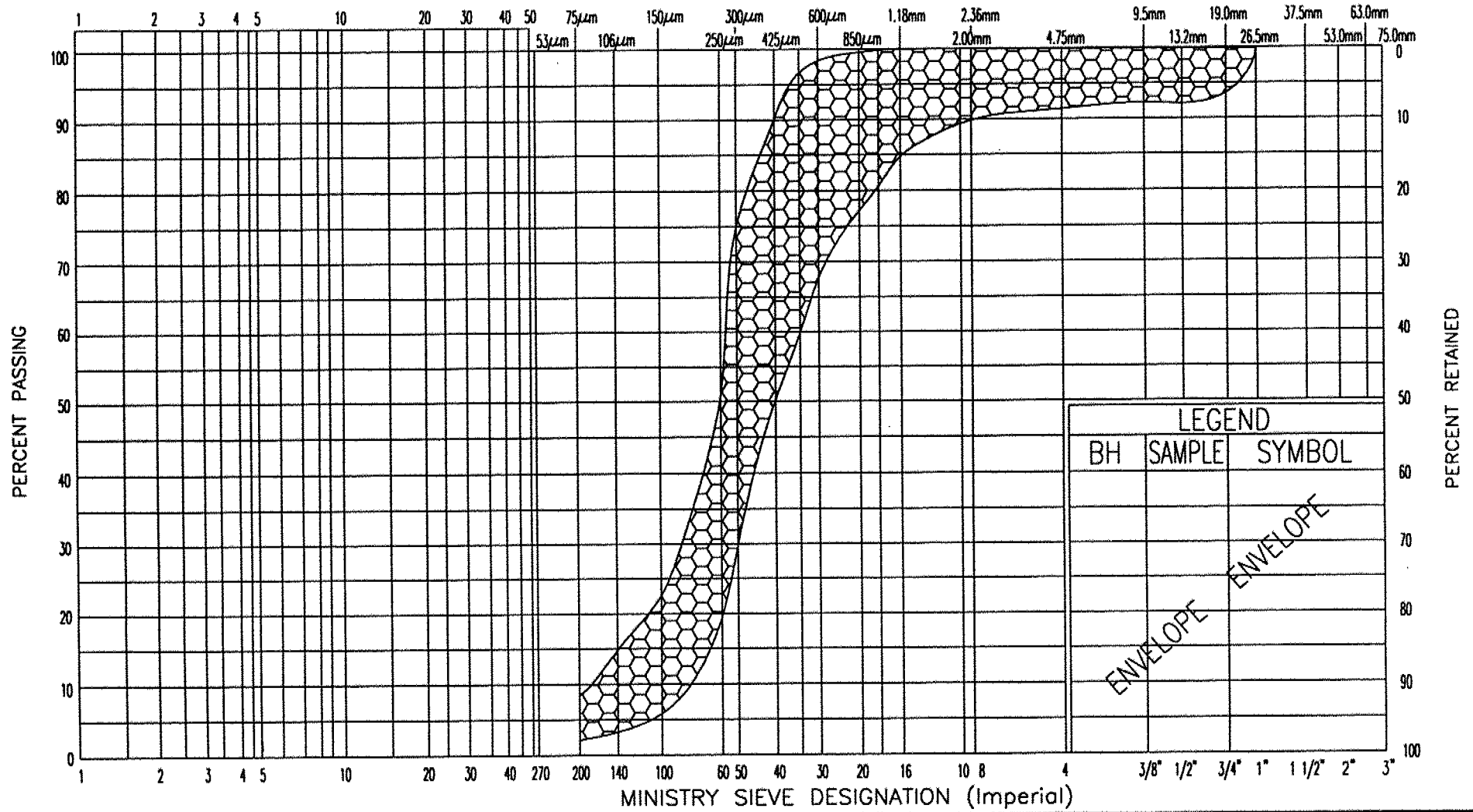
W P 466-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH SAMPLE SYMBOL

ENVELOPE ENVELOPE

GRAIN SIZE DISTRIBUTION
SAND

FIG No 2
W P 466-93-00

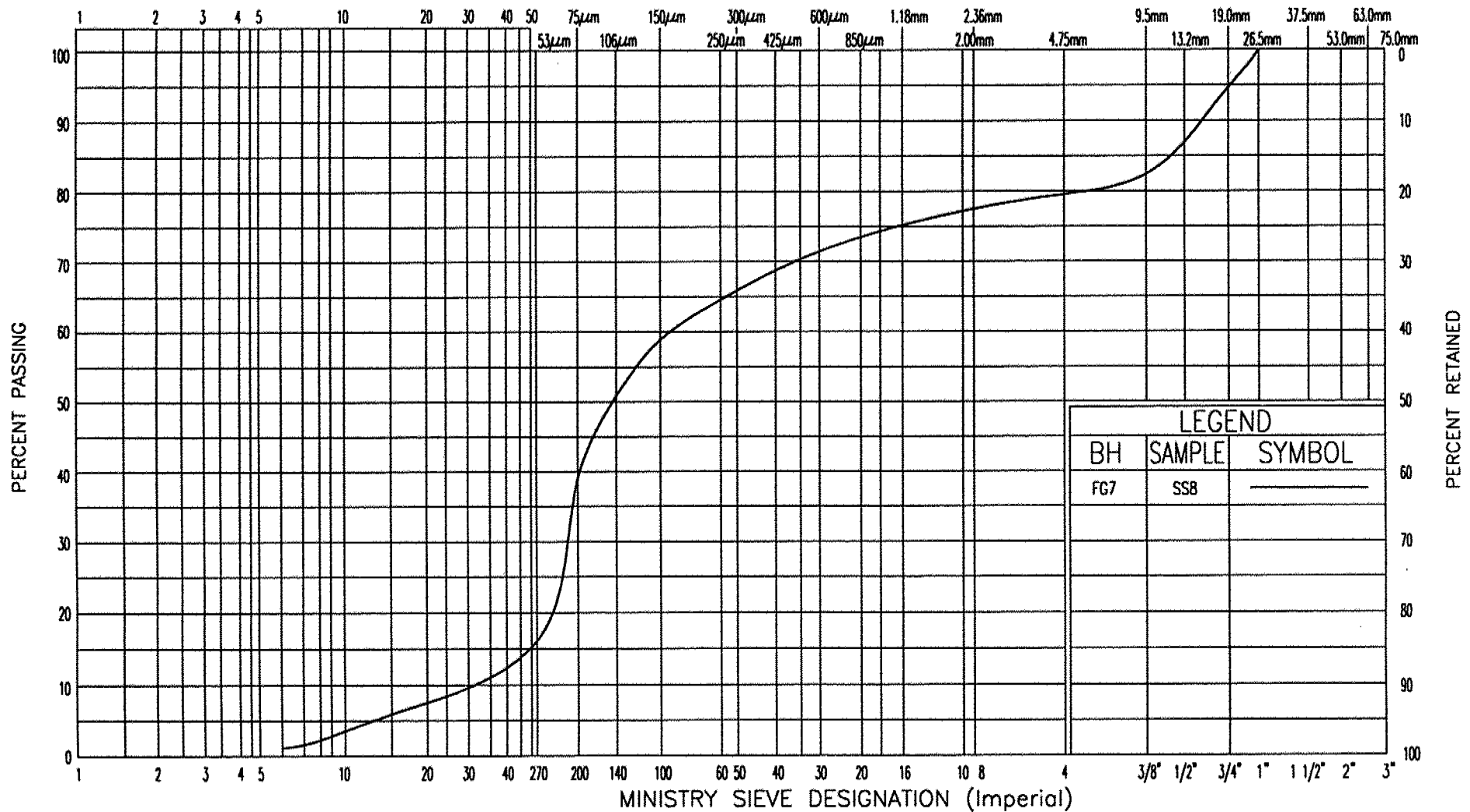


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



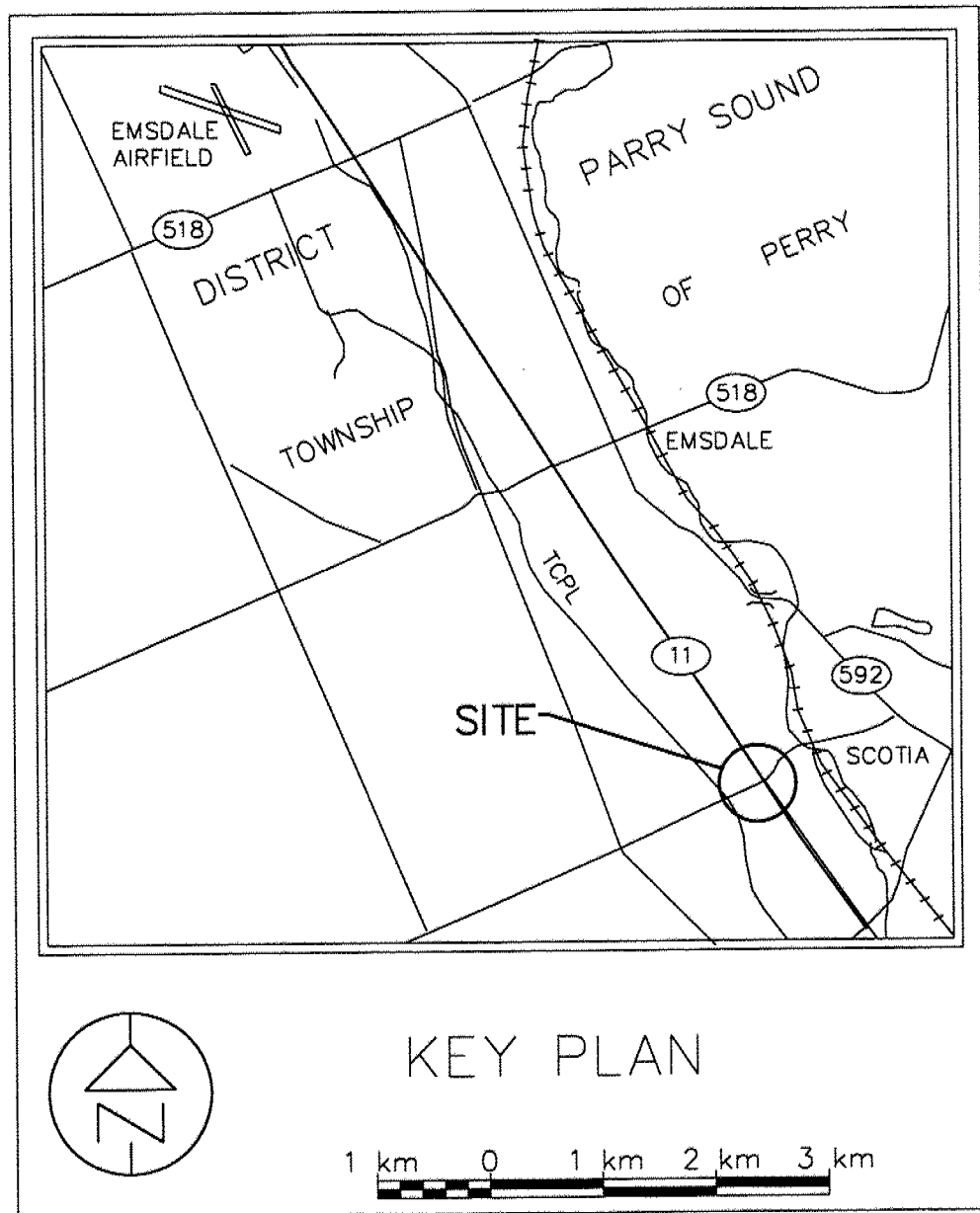
LEGEND		
BH	SAMPLE	SYMBOL
FG7	SS8	—



GRAIN SIZE DISTRIBUTION
SANDY SILT SOME GRAVEL

FIG No 3
W P 466-93-00

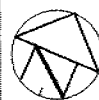
ENCLOSURES



FERN GLEN ROAD UNDERPASS
KEY PLAN

Dwg. No 1



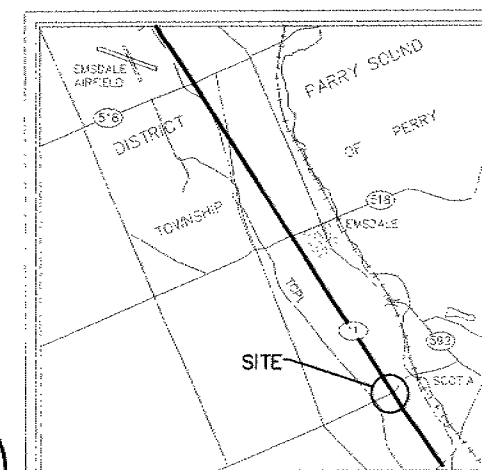


CONT. No.
W.P. No. 467-93-01

SHEET

FERN GLEN ROAD UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA

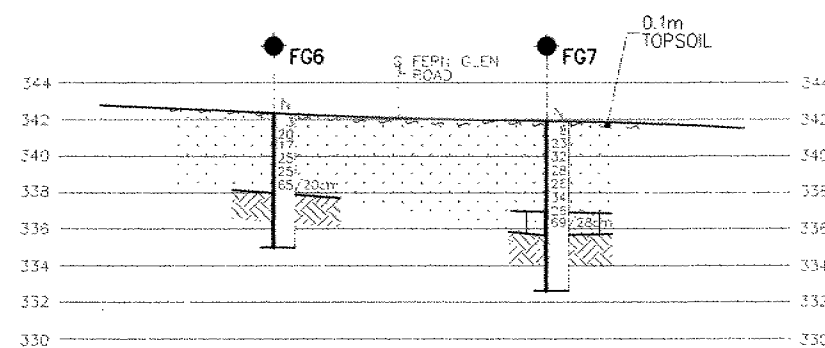
AGRA Earth & Environmental Ltd.



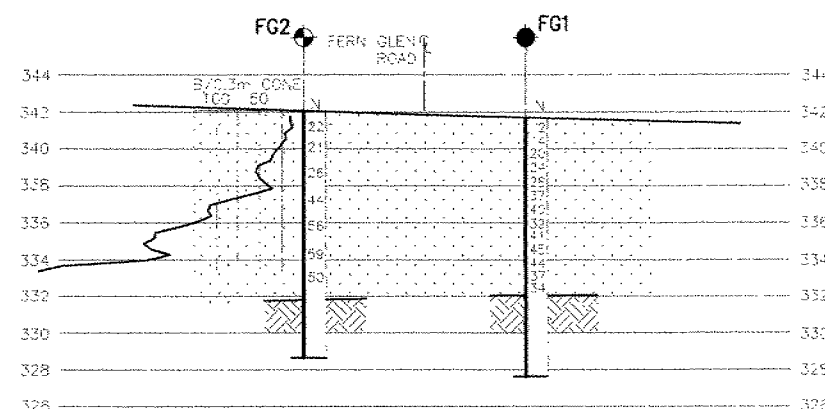
KEY PLAN
1 km 2 km 3 km

METRIC

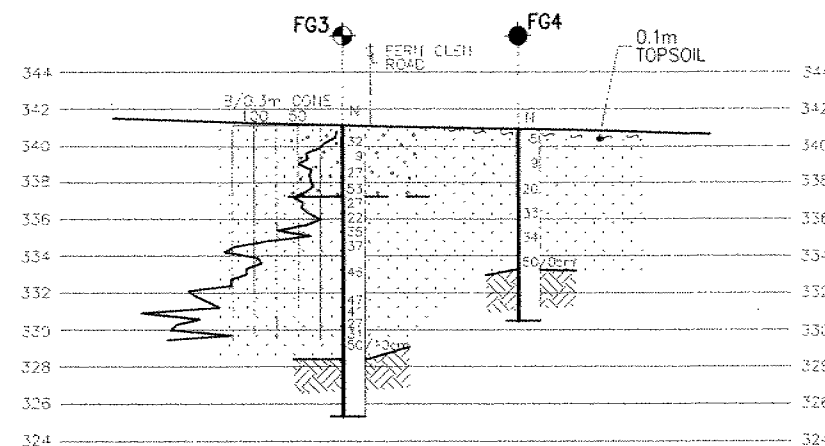
ALL DIMENSIONS ARE IN METRES
UNLESS OTHERWISE SHOWN
OTHERWISE SHOWN IN METRES



SECTION C-C



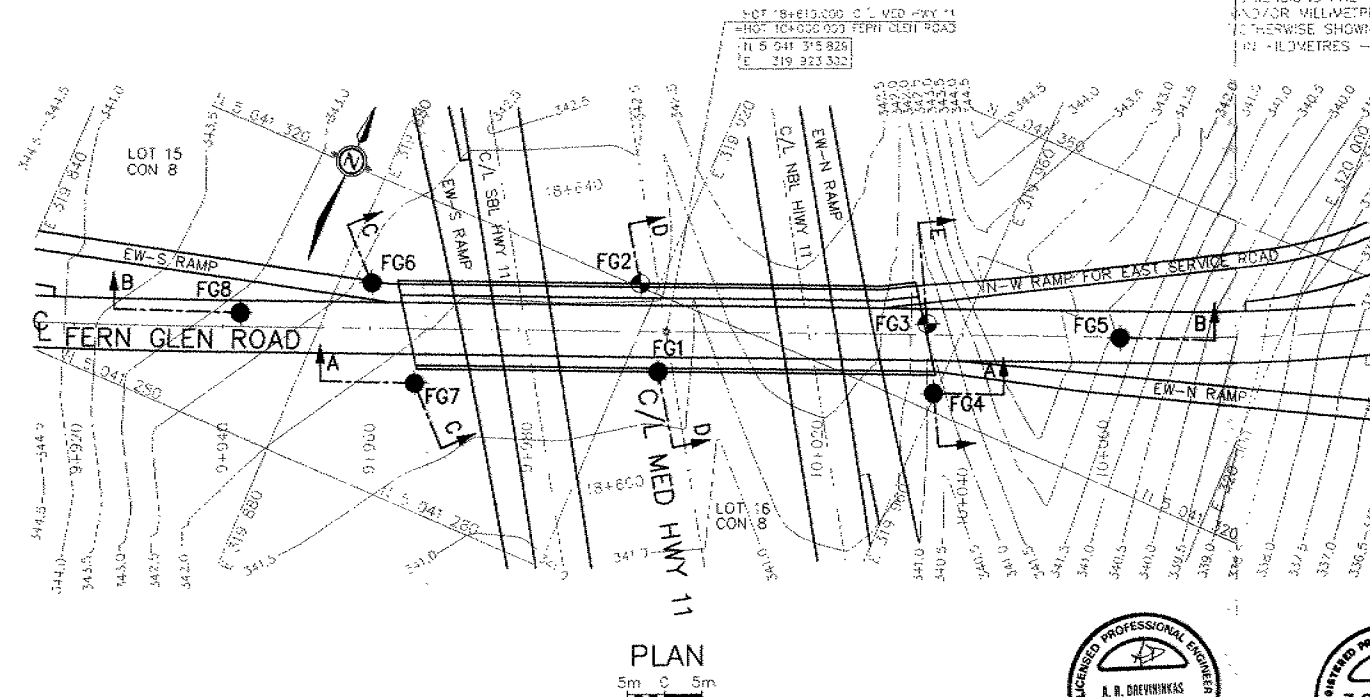
SECTION D-D



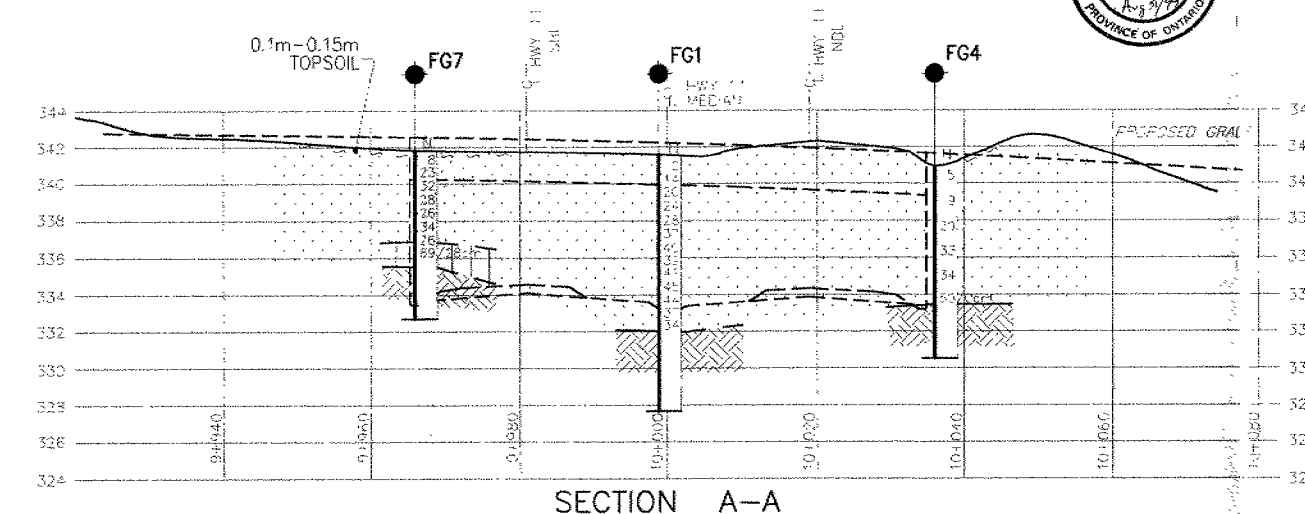
SECTION E-E

SOIL STRATIGRAPHY LEGEND

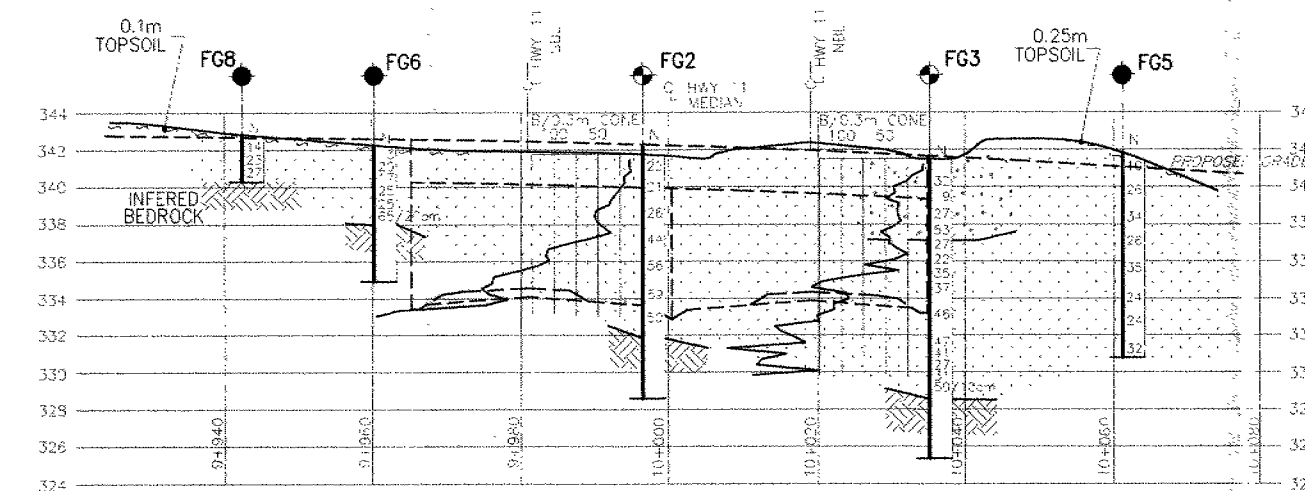
	SAND FINE to MEDIUM Very Loose to Very Dense		SANDY SILT SOME GRAVEL Compact to Dense
	SAND to GRAVELLY SAND Loose to Compact		GNEISS BEDROCK



PLAN
5m 0 5m



SECTION A-A



SECTION B-B

5m 0 5m HOR
2m 0 2m VER

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- 'N' Blows/0.3m (Sta Pen Test, 475 J/blow)
- CONC Blows/0.3m (60' Cone, 475 J/blow)
- WL at time of investigation Dec. 98
May. 99
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
FG1	341.8	5 041 311	319 324
FG2	342.2	5 041 321	319 917
FG3	341.0	5 041 331	319 955
FG4	340.8	5 041 323	319 959
FG5	342.0	5 041 340	319 979
FG6	342.3	5 041 306	319 884
FG7	341.8	5 041 296	319 895
FG8	342.5	5 041 295	319 869

-NOTE-

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 20.1 of CPS Gen. Cons.

REV	DATE	BY	DESCRIPTION
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REF. Hwy 11 Bridge Site Plan
Cwg. by AGRA Jan. 1999

HWY 11	DATE June 1999	DATE 44-391
SUBMIT TO CHECKED AD	DATE 44-391	DATE 44-391
DRAWN BY	CHECKED	DWG 2

RECORD OF BOREHOLE No FG1

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041310.8 E 319924.2 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 16 December 1998 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE												
341.8							20	40	60	80	100						GR SA SI CL			
0.0	brown SAND fine to medium dry	<div>V.loose</div> <div>compact</div> <div>dense</div>	1	SS	2												Station 9 + 999 5.0 Rt Fern Glen Rd. C/L			
			2	SS	14		341													
			3	SS	20		340													
			4	SS	24		339													
			5	SS	28		338											4 93 (3)		
			6	SS	37		337													
			7	SS	40		336											0 96 (4)		
			8	SS	39		335													
			9	SS	41		334													
			10	SS	45		333													
			11	SS	44		332													
			12	SS	37		331											1 92 (7)		
			13	SS	34		330													
332.1			14	RC												RC14: REC=31% RQD=0%				
9.8	moderately weathered		15	RC																
	GNEISS BEDROCK		16	RC												RC15: REC=100% RQD=100%				
	massive, moderately closely jointed		17	RC												RC16: REC=100% RQD=66%				
			18	RC												RC17: REC=100% RQD=82%				
327.8																RC18: REC=100% RQD=42%				
14.0	END of BOREHOLE																			
	WL on completion: none																			

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

RECORD OF BOREHOLE No FG2

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041320.8 E 319917.2 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 16 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
342.2 0.0	frozen		1	SS	22		342	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						Station 9 + 996 6.9 Lt Fern Glen Rd. C/L
	brown SAND fine to medium dry		2	SS	21		341							
							340							
	compact		3	SS	26		339							9 86 (5)
							338							
	dense		4	SS	44		337							
							336							
	very dense		5	SS	56		335							4 91 (5)
							334							
			6	SS	59		333							
							332							
			7	SS	50		331							
							330							
331.8 10.4	moderately weathered		8	RC			329							RC8: REC=97% RQD=52%
	GNEISS BEDROCK		9	RC										RC9: REC=100% RQD=93%
	massive, moderately closely jointed													
			10	RC										RC10: REC=99% RQD=83%
328.6 13.6	END of BOREHOLE													
	WL on completion: none													

RECORD OF BOREHOLE No FG3

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041331.4 E 319954.8 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 7 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
341.0														
0.0	0.15m Gravelly Sand FILL		1	AS										Station 10 + 035 2.5 Lt Fern Glen Rd. C/L
	frozen		2	SS	32		340							
	loose		3	SS	9		339							
	compact		4	SS	27		338							29 68 (3)
	brown SAND to GRAVELLY SAND dry		5	SS	53*		337							SS5: stone in sampler
337.2			6	SS	27		336							
3.8			7	SS	22		335							0 98 (2)
	compact		8	SS	35		334							
	dense		9	SS	37		333							
	brown SAND fine to medium dry		10	SS	46		332							
			11	SS	47		331							
			12	SS	41		330							
			13	SS	27		329							
			14	SS	31		328							
			15	SS	50/100		327							
328.5			16	RC			326							
12.5			17	RC										
			18	RC										
			19	RC										
325.4														
15.7	WL on completion: none													

RECORD OF BOREHOLE No FG4

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041323.1 E 319959.4 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 11 May 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
340.8	0.15m TOPSOIL		1	SS	5			20	40	60	80	100					
0.0																	

RECORD OF BOREHOLE No FG5

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041340.1 E 319979.3 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 9 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE												
342.0							20	40	60	80	100									
0.0	0.25m TOPSOIL (frozen)		1	SS	10												Station 10 + 061 1.0 Lt Fern Glen Rd. C/L			
			2	SS	26															
			3	SS	34												1 95 (4)			
			4	SS	26															
			5	SS	35												0 95 (5)			
		6	SS	24																
		7	SS	24																
		8	SS	32																
330.8																				
11.3	END of BOREHOLE WL on completion: none															Move borehole 2m Lt, auger refusal @ 0.1m. Move borehole 1m Lt, auger refusal @ 0.1m.				

RECORD OF BOREHOLE No FG6

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041306.3 E 319883.9 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 14 April 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
342.3	0.1m TOPSOIL		1	SS	3		342							Station 9 + 960 6.0 Lt Fern Glen Rd. C/L				
	very loose		2	SS	20*		341							SS2: No recovery				
	compact		3	SS	17		340							2 89 (9)				
	brown SAND fine to medium dry		4	SS	25		339							Auger Refusal @ 4.1m Advance borehole by rock coring				
	cobbles		5	SS	25		338											
338.0			6	SS	65/20		337							RC7: REC=100% RQD=100%				
4.3	GNEISS BEDROCK		7	RC			336							RC8: REC=100% RQD=99%				
	massive, moderately closely jointed		8	RC			335							RC9: REC=100% RQD=75%				
			9	RC										RC10: REC=100% RQD=83%				
335.0			10	RC														
7.3	END of BOREHOLE																	
	WL on completion: none																	

RECORD OF BOREHOLE No FG7

1 OF 1

METRIC

W.P. 467-93-01 LOCATION N 5041296.1 E 319894.6 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 14 April 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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341.8	0.1m TOPSOIL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														

RECORD OF BOREHOLE No FG8										1 OF 1		METRIC							
W.P. 467-93-01		LOCATION N 5041295.4 E 319869.2				ORIGINATED BY AD													
DIST 52 HWY 11		BOREHOLE TYPE Solid Stem				COMPILED BY CK													
DATUM Geodetic		DATE 14 April 1999				CHECKED BY ZSO													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										WATER CONTENT (%)	
342.5	0.1m TOPSOIL		1	SS	14		342											Station 9 + 942 1.5 Lt Fern Glen Rd. C/L	
	brown SAND fine to medium compact, dry		2	SS	23		341												
			3	SS	27														
340.2	2.3 END of BOREHOLE																		
	AUGER REFUSAL on probable Bedrock																		
	WL on completion: none																		