

GEOCRES No;
31B-68

T11688A

REPORT TO

FENCO ENGINEERS INC.
WILLOWDALE ONTARIO

HIGHWAY 416
KEMPTVILLE ONTARIO
FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD 44 UNDERPASS
WP 372-89-02; Site 16-315
DISTRICT 9, KINGSTON
GEOCRES # 31B-68

Distribution:

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GEOCON (1991) INC.
December, 1991

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GENERAL CONDITIONS AND LIMITATIONS

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DRAWING NO. 3728902-A Stratigraphic Profile

1.0 INTRODUCTION

Presented herein are the results of a geotechnical subsurface investigation conducted at the above site to establish the prevalent subsurface geotechnical conditions for the design and construction of the proposed bridge, approach fills and Highway 44 realignment. Geocon (1991) Inc. (Geocon) was retained by Fenco Engineers Inc. to perform this work.

The field work for this project was conducted between November 12th and November 14th, 1990 using a CME 55 drill rig equipped with 200 mm diameter hollow stem augers. The investigation consisted of 5 boreholes of depths ranging from 3.5 m to 11.5 m. The soil overburden was augered then sampled using the Standard Penetration Tests (SPT) and the underlying bedrock was cored in NXL size. Two standpipe piezometers were installed to monitor the groundwater levels.

The locations of the boreholes are shown on Drawing 3728902-A. A record of the encountered subsurface conditions at each borehole and the single test pit, are given on the Record of Borehole Sheets in Appendix A.

2.0 SITE DESCRIPTION AND GEOLOGY

The proposed Highway 44 underpass is located just south of the intersection of Highway 44 and Highway 16 approximately 5 kilometers south of Kemptville, Ontario (Figure 1). The proposed underpass will comprise of two spans supported on a central pier and two abutments with approach fills of 4.0 to 6.0 m in height. The proposed underpass will traverse Highway 416 at a skewed angle trending southeast to northwest. The proposed east span of the underpass straddles the existing Highway 16 which is contained within a 3 m deep rock cut.

The proposed interchange is located on a bedrock high with bedrock outcrops at many locations. Based on site mapping of existing soil and rock cuts, the underlying bedrock gives way to glacial till deposits to the north near the intersection of County Road 44 and Highway 16. East of Highway 16 the site is typically grassland with some trees. The ground surface is undulating and generally slopes towards the south. To the west of Highway 16 the ground slopes towards the northwest and is densely covered with large trees. In the northwest quadrant of the proposed interchange a low lying poorly drained area with standing surface water may be found.

The proposed Highway 416 is located within the physiographic region of the Ottawa-St. Lawrence lowland. During the last Ice Age this area was glaciated which resulted in the deposition of a layer of till over much of the proposed alignment (Sharpe, 1979). In general the till comprises of a bouldery cobbly silty sand to sandy silt.

Subsequent to glaciation the Ottawa-St. Lawrence lowland was inundated by the Champlain Sea. At the location of the proposed Highway 416 alignment the depth of water was shallow and in general only resulted in minor wave action modification of the surface of the underlying till. However, in localized low lying areas deeper deposits (up to 10 m) of fine grained clayey silt material may be present.

Available surficial geology information (OGS Map 2387) indicates that the site is located on a bedrock outcrop of massive grey dolostone known as the Oxford Formation of the Beekmonton Group. The bedrock is overlain by a wave washed pebbly sandy layer generally less than 1 m thick (Sharpe 1979).

3.0 SUBSURFACE CONDITIONS

3.1 General

The subsurface conditions at the site of the proposed underpass are characterized generally by a thin cover of overburden overlying bedrock. However, between the proposed location of the central pier and west abutment the bedrock is deeper with corresponding thicker overburden.

The factual information which was used to interpret the soil conditions is given in Appendix A and B and Drawing No. 3728902-A. A summary of borehole depths and elevations is given in Table 1 of Appendix A. The subsurface conditions encountered within the proposed underpass area are described below:

3.2 Topsoil and Peat

A thin surficial layer of topsoil and peat, of up to 0.2 m in thickness, was encountered within the underpass area. However, peat thicknesses of up to 0.8 m were encountered at the location of the proposed western ramps.

3.3 Silty Sand to Sandy Silt and Sandy Gravel

Underlying the topsoil and peat is a thin layer of cohesionless material. This stratum consists of 0.5 m to 1.0 m of sandy silt with some gravel and occasional cobbles at the east abutment, about 0.4 m of gravelly sand some silt at the central pier, and 1.0 m of

silty sand with trace gravel, which in turn overlies 0.5 m of sandy gravel with some silt, at the west abutment.

3.4 Clayey Silt

The cohesionless stratum is underlain by a layer of grey clayey silt with trace to some sand, at the west abutment location only and ranged from 1.4 to 2.5 m in thickness. Based on the results of two grain size analyses performed on split spoon samples (Figure B2) the material may be described as clayey silt with trace to some sand.

SPT 'N' values within this layer vary from 20 to 31 inferring a strata of very stiff to hard consistency.

Atterberg Limits tests performed on two soil samples indicate the liquid limit to range from 27 to 29%, with plastic limits and natural water contents of 17% for both samples.

3.5 Glacial Till

Glacial till was encountered below the clayey silt layer at the west abutment location and ranged from 0.6 to 0.9 m in thickness. Based on a single grain size analysis the till may be described as a sand and silt with trace to some gravel and trace clay. SPT 'N' values within the layer ranged from 78 to refusal inferring a very dense stratum.

A single moisture content of 11% was determined for this layer.

3.6 Bedrock

Dolostone bedrock was encountered at about El. 109.0 m at the east end of the proposed underpass and dips to El. 103.4 m at the western end. These elevations are equivalent to about 0.6 m to 5.0 m below existing ground surface, respectively. The bedrock was proven in all five boreholes by coring from 2.9 m to 6.3 m into dolostone. The bedrock is judged by core recovery and RQD percentages, to be of good quality. However, at Borehole 8-3, the bedrock was of very poor to fair quality. Borehole 8-3 is located adjacent to the existing Highway 16 road cut. The lower rock quality at this location may in part be the result of blasting activities during excavation of the cut.

The bedrock is of dark grey fine to medium dolostone, characterized by very tight, very closely spaced to close spaced thin shale interbeds. In addition, randomly located oval shaped discontinuous vugs are present. Unconfined compressive strength tests performed on selected core samples indicate compressive strengths ranging from 49 MPa to 139 MPa.

3.7 Groundwater

Groundwater elevation was measured by means of two standpipe piezometers installed in Boreholes 8-1 and 8-4 at the east and west abutments, respectively. The groundwater levels were measured approximately 18 days after installation of piezometers and were observed to be at El. 106.6 m (2.7 m below ground surface) at Borehole 8-4 and El. 107.3 m (1.7 m below ground surface) at Borehole 8-1. Groundwater levels could be expected to vary seasonally.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 GENERAL

The proposed underpass will have two spans supported on two abutments and a central pier. At this site, it is anticipated that 4 to 6 m high approach fills will be adjacent to the abutments. As discussed in the following sections spread footings placed within the bedrock are recommended for the central pier and the east abutment. The use of spread footings placed on an engineered fill is recommended for the west abutment, however end bearing H-Piles may also be used to support the west abutment.

Slope stability analysis, indicates that embankments constructed with conventional 2 Horizontal to 1 Vertical side slopes will be stable.

4.2 UNDERPASS FOUNDATIONS

4.2.1 East Abutment and Central Pier

Sound bedrock was found to be within 1.0 m from the existing ground surface at the location of the east abutment. Therefore, design bearing pressures of 3 MPa at the ULS condition are recommended for spread footings placed at least 0.3 m below the bedrock surface. This value is conditional upon the leading edge of the footing being placed outside a failure line inclined at 45° to the horizontal and originating from the toe of the adjacent rock cut as shown in Figure 3. Recommendations for the SLS, Type II condition are not required as settlements are anticipated to be negligible. Footings placed on sound bedrock would obviate the need for frost protection measures.

It is understood that lowering of the existing ground surface by 3 to 4 m at the location of the proposed central pier will be required to accommodate the southbound lanes of the proposed Highway 416. Therefore, spread footings bearing on sound bedrock at or below El. 107.1 m (2.6 m below existing ground surface) may be utilized. Recommendations for the east abutment are also applicable for the central pier.

4.2.2 West Abutment

Bedrock is encountered at 3.7 m to 5.0 m from existing ground surface at the west abutment location. The proposed approach fills to the abutment are anticipated to be as much as 6.0 m high. Therefore, spread footings placed on engineered fill may be used to support the proposed abutment. Alternatively, end bearing H-Piles may be considered to create an unyielding foundation system similar in performance to that proposed for the central pier and east abutment.

4.2.2.1 Spread Footings Placed on Engineered Fill

For the assumed geometry of this foundation solution (Figure 2), the recommended bearing pressures at the Serviceability Limit State (SLS) and factored Ultimate Limit State (ULS) conditions are 400 kPa and 800 kPa, respectively.

The SLS design load is the load at which the estimated settlement of the footing will be of the order of 25 mm which for the purposes of this design has been assumed as the maximum settlement that may be tolerated. This settlement is comprised of 20 mm within the engineered fill and 5 mm within the natural overburden materials. It should be noted that both of these elements of settlement will be largely elastic and will occur mostly during initial loading of the foundations. An integral part of this proposed

foundation design is the construction of an engineered fill on which to place the footing (Figure 2). Frost protection measures for spread footing as engineered fill should be in accordance with Figure 2.

4.2.2.2 End Bearing H-Piles

In order to create a foundation system with similar deformation characteristics as the central pier and east abutment, it is recommended that end bearing steel H-Piles be driven to support the proposed perched abutment. The piles should be driven through the embankment fill and overburden to practical refusal into the underlying bedrock which occurs between El. 103.0 to 105.0 m. Allowable loads of 1150 kN and 1600 kN may be used at the SLS and ULS conditions respectively for HP 310 x 110 steel piles. These loads should, however, be checked against the structural capacity of the steel piles used. Settlements of the pile cap will be governed by the elastic compression of the pile units. It is recommended that the H-Piles be equipped with driving shoes. Pile design installation details such as termination resistance and the rated energy capacity of the pile hammer would largely depend on the pile type chosen. For preliminary design purposes, HP 310 x 110 steel piles may be driven to a set about 10 blows for the last 25 mm of penetration using a hammer transferring about 60 kilojoules of energy per blow to the pile. We would be pleased to review the pile installation details once the pile type has been chosen. Consideration should be given to re-striking the piles if relaxation is observed. Frost protection of up to 1.8 m of earthcover or equivalent will be required for the pile caps.

4.3 EMBANKMENT RECOMMENDATIONS

Based on the observed subsurface conditions encountered in the area of the proposed underpass abutments and an anticipated maximum embankment height of 8 m, it is concluded that embankments constructed with side slopes of 2 Horizontal to 1 Vertical will remain stable.

Within the area of the western ramps of the proposed interchange the subsurface conditions encountered comprise a surficial layer of organics, up to 0.8 m thick, overlying competent glacial till. Subject to the removal of all organics it is considered that conventional embankments constructed with side slopes of 2 Horizontal to 1 Vertical will remain stable. Details of design recommendations for the ramps and associated service roads are given in the companion Pavement Design Report for the project.

Embankment fill should meet the requirements of OPSS 212 for borrow material and should be placed and compacted in accordance with OPSS 206. Slopes of 2 Horizontal to 1 Vertical are applicable for sandy earth borrows, rock borrow or select subgrade fill material. If silty or clayey earth borrow is used, the embankment side slope should be 2.5 Horizontal to 1 Vertical or flatter and are to be confirmed by engineering analyses. Prior to the placement of any imported fill materials, the subgrade should be stripped of all topsoil and organics and any other deleterious material which may be present. The receiving subgrade, comprising glacial till or sand, should be proof-rolled and any soft areas excavated and replaced with compacted granular material. Settlements of the main approach embankments and the western ramps are estimated to be of the order of 10 mm which will primarily occur during initial loading of the embankments.

4.4 GENERAL DESIGN RECOMMENDATIONS

4.4.1 Dewatering

It is anticipated that excavation below groundwater level will be quite shallow and water inflow into the excavation can be handled by a system of ditches leading to a central sump and pump.

4.4.2 Excavations

Temporary excavations in bedrock are anticipated only at the central pier location and may be done concurrently with lowering of the ground surface for the construction of the southbound lanes of the proposed Highway 416. Bedrock may be excavated with side slopes of 4 Vertical to 1 Horizontal. The excavation of slopes shall be in compliance with the Ontario Health and Safety Act regulations or other governing regulations within the area.

4.4.3 Earth Backfill Pressures

The earth pressure for the design of the abutments should be computed as per Section 6.1.2 of the O.H.B.D.C., and an unyielding foundation condition may be assumed for the computations. If, however, movement of the top of the retaining walls is permitted and allowed to exceed 0.05% of the overall height of the wall, a yielding condition may be assumed for the computations. The Granular 'A' or 'B' backfill should be in accordance with the MTO Special Provision No. 109F03. The following parameters are recommended for the granular backfill:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction	$\phi = 35^\circ$	$\phi = 30^\circ$
Unit Weight (kN/m^3)	$\gamma = 22.8$	$\gamma = 21.2$

If the footings are placed on compacted granular backfill, an unfactored coefficient of friction value of $\tan 30^\circ$ may be assumed for the estimation of the sliding resistance. Alternatively, if footings are placed directly on bedrock, an unfactored coefficient of friction value of $\tan 35^\circ$ may be used.

4.4.4 Frost Penetration

The anticipated maximum depth of frost penetration at the site is 1.8 m (Canadian Foundation Engineering Manual). All foundation units should be provided with at least this depth of soil or equivalent cover below finished grade if not founded on bedrock. In addition, where approach fill embankments are less than the anticipated depth of frost penetration additional design measures will be required to ensure the satisfactory performance of the pavement. This aspect of the design will be addressed in more detail in the Pavement Design Report.

4.4.5 Site Supervision

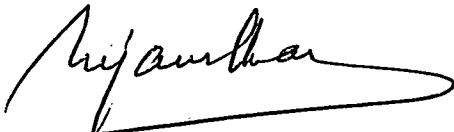
The recommendations given in this report are based on the assumptions that the assumed soil conditions will be verified in any engineered fill and excavations and that all construction recommendations are followed. It is recommended, therefore, that the foundation and earthworks construction be carried out under suitably qualified geotechnical engineering supervision.

5.0 CLOSURE

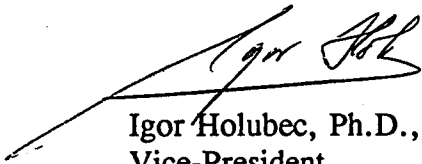
The field work portion for the investigation was done under the supervision of Mr. G.C. Yule. The report was written by Mr. I. Corbett, P.Eng. and Mr. N. Khan, P.Eng., and reviewed by Dr. I. Holubec, P.Eng.

This report is subject to the attached General Conditions and Limitations.

Yours very truly
GEOCON (1991) INC.

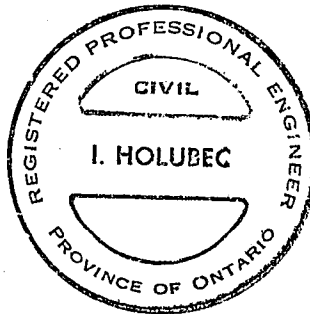


N. Khan, P.Eng.
Project Engineer



Igor Holubec, Ph.D., P.Eng.
Vice-President

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GEOCON (1991) INC.

GEOTECHNICAL REPORT

GENERAL CONDITIONS AND LIMITATIONS

A. USE OF THE REPORT

- A.1 The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation or if the project is not initiated within eighteen months of the date of the report Geocon (1991) Inc. (Geocon) should be given an opportunity to confirm that the recommendations are still valid.
- A.2 The comments given in this report are intended only for the guidance of the design engineer. The number of test holes to determine all the relevant underground conditions which may affect construction costs, techniques and equipment choice, scheduling and sequence of operations would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual test hole data, as to how subsurface conditions may affect their work.

B. FOLLOW-UP

- B.1 All details of the design and proposed construction may not be known at the time of submission of Geocon's report. It is recommended that Geocon be retained during the final design stage to review the design drawings and specifications related to foundations, earthworks, retaining systems and drainage, to determine that they are consistent with the intent of Geocon's report.
- B.2 Retention of Geocon during construction is recommended to confirm and document that the subsurface conditions throughout the site do not materially differ from those given in Geocon's report and to confirm and document that construction activities did not adversely affect the design intent of Geocon's recommendations.

C. SOIL AND ROCK CONDITIONS

- C.1 Soils and rock descriptions in this report are based on commonly accepted methods of classification and identification employed in professional geotechnical practice. Classification and identification of soil and rock involves judgement and Geocon does not guarantee descriptions as exact, but infers accuracy only to the extent that is common in current geotechnical practice.
- C.2 The soils and rock conditions described in this report are those observed at the time of the study. Unless otherwise noted, those conditions form the basis of the recommendations in the report. The condition of the soil and rock may be significantly altered by construction activities (traffic, excavation, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil and rock must be protected from these changes or disturbances during construction.

D. LOGS OF TEST HOLES AND SUBSURFACE INTERPRETATIONS

- D.1 Soil and rock formations are variable to a greater or lesser extent. The test hole logs indicate the approximate subsurface conditions only at the locations of the test holes. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of boring, the frequency of sampling, the method of sampling and the uniformity of subsurface conditions. The spacing of test holes, frequency of sampling and type of boring also reflect budget and schedule considerations.
- D.2 Subsurface conditions between test holes are inferred and may vary significantly from conditions encountered at the test holes.
- D.3 Groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities on the site or adjacent sites.

E. CHANGED CONDITIONS

- E.1 Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the use or reliance by the client of this report that Geocon is notified of the changes and provided with an opportunity to review the recommendation of this report. Recognition of changed soil and rock conditions requires experience and it is recommended that an experienced geotechnical engineer be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

F. DRAINAGE

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage can have serious consequences. Geocon can take no responsibility for the effects of drainage unless Geocon is specifically involved in the detailed design and follow-up site services during construction of the system.

REFERENCES

Canadian Foundation Engineering Manual, 1985. Second Edition. Part 1: Fundamentals; Part 2: Shallow Foundations; Part 3: Deep Foundations; Part 4: Excavations and Retaining Structures Part 5: References. Canadian Geotechnical Society, Technical Committee on Foundations, 456 pp.

Sharpe, D.R., 1979. Quaternary Geology of the Merrickville Area, Southern Ontario. Ontario Geological Survey, Report 180, 54P. Accompanied by Maps 2387 and 2388, scale 1:50,000.

Totten Sims Hubicki Associates (1981) Limited Consultants 1990 Structure Data Report, Highway 416 from 0.7 km north of Hwy 401 northerly to 1.0 km north of Hwy 43, District 9 Ottawa. Report prepared for Ministry of Transportation, Ontario.

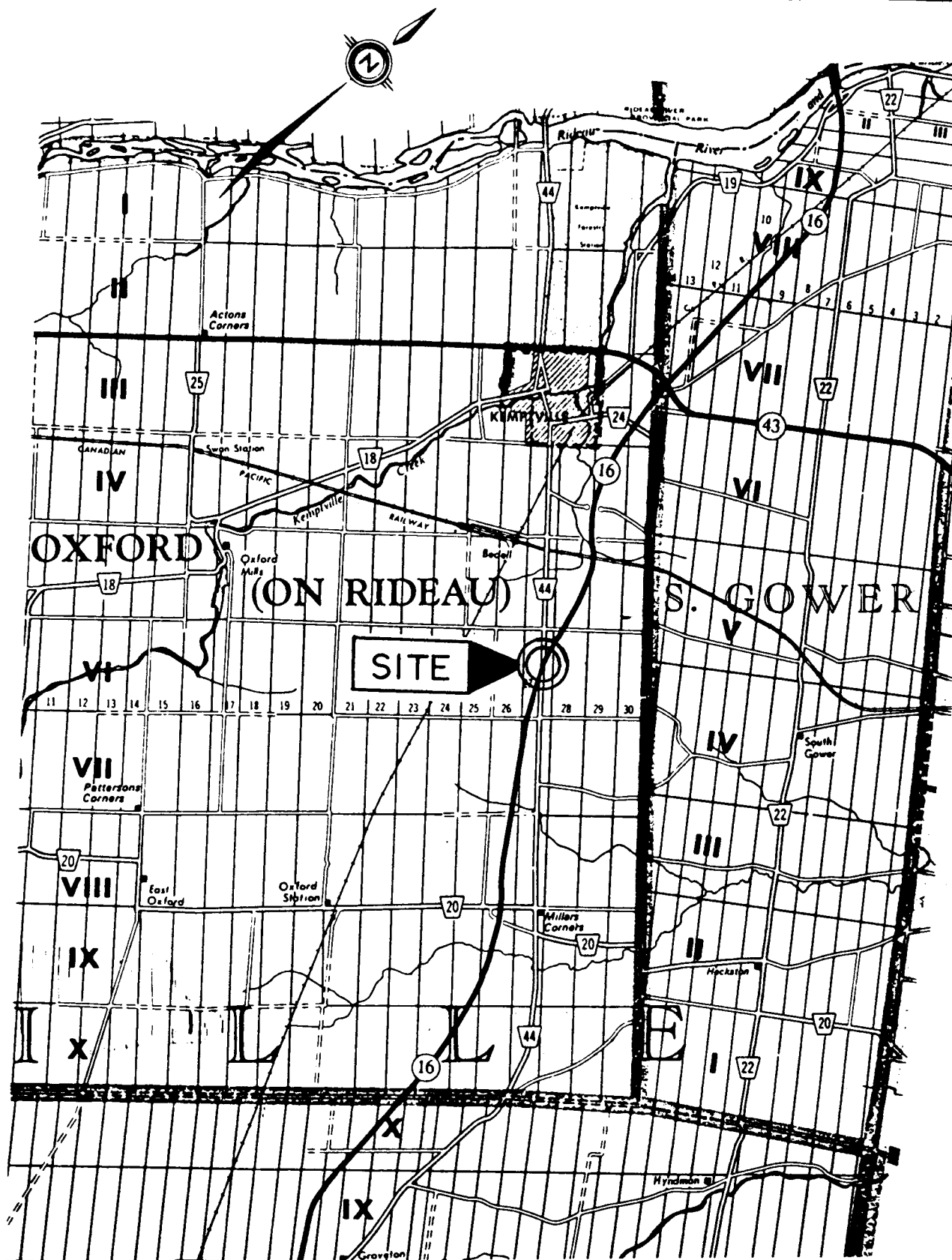
KEY PLAN

APPENDIX

FIGURE

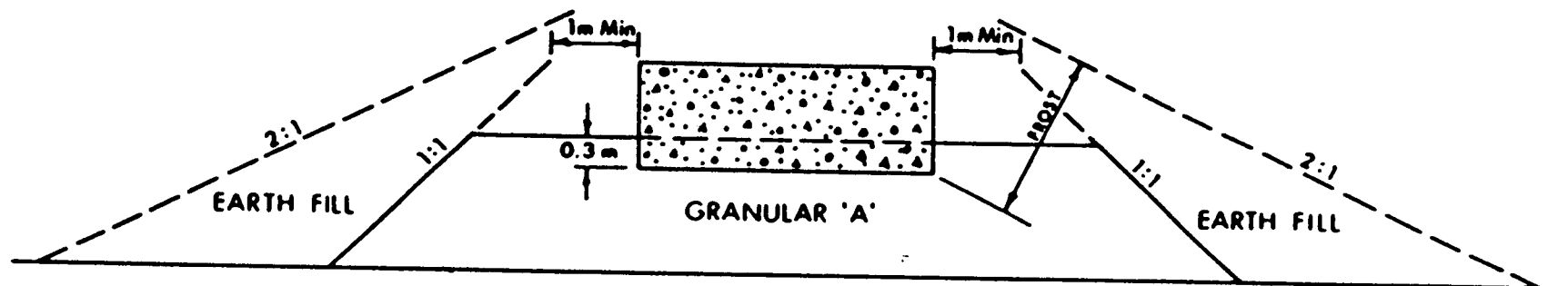
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PROJECT WP 372-89-02

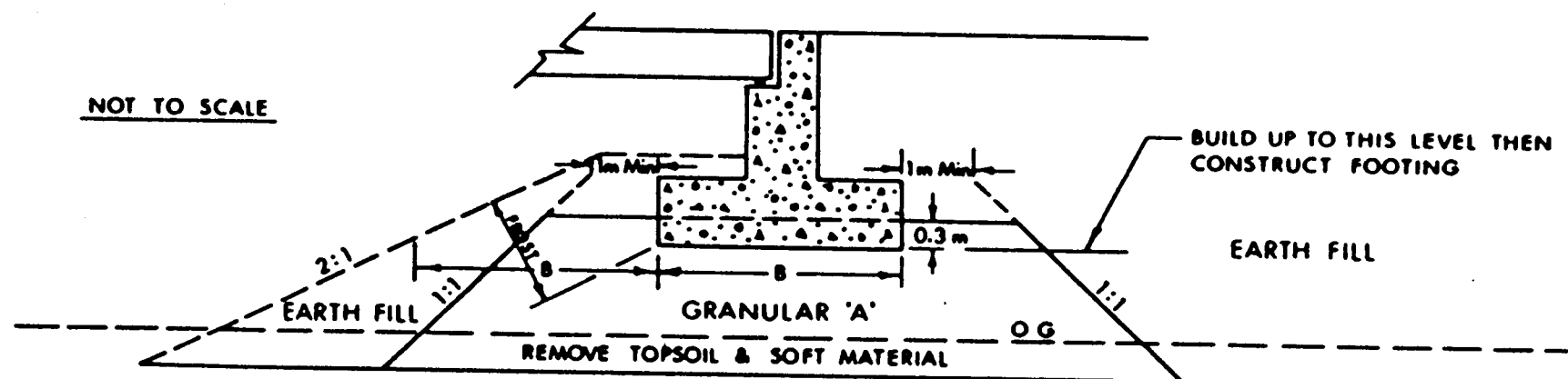


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GEOCON



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

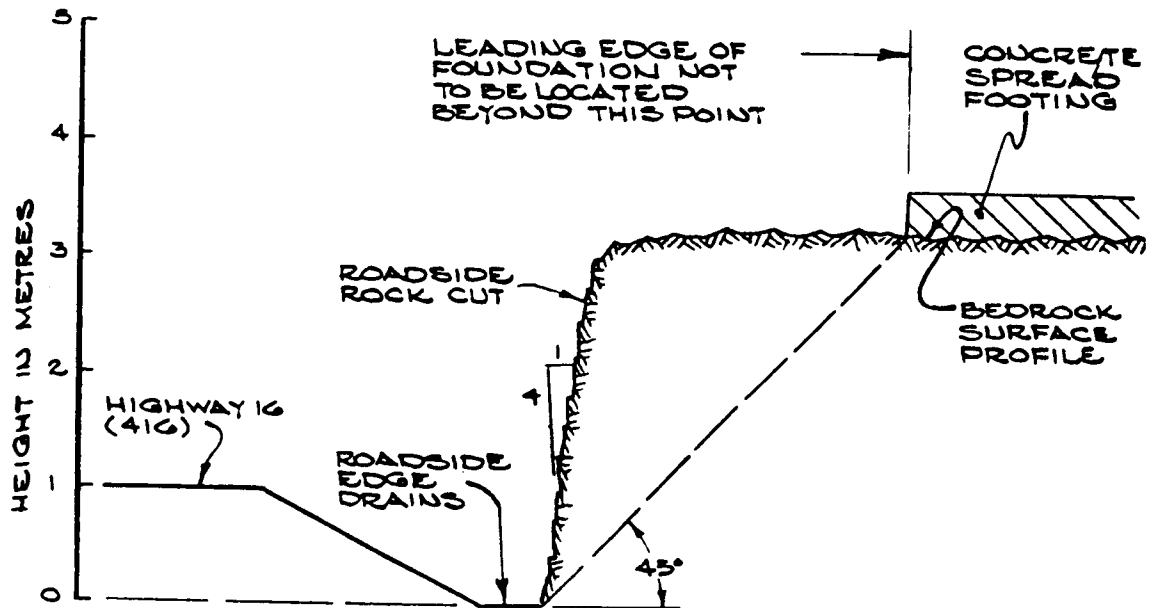


Ministry of
Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 2

W P 372-89-02



APPENDIX A

Borehole Information

Explanation of Terms used in this Report

Explanation of Term Rock Quality Designation (RQD)

Table 1 - Summary of Borehole Information

Record of Borehole Sheets (Boreholes 8-1 to 8-5)

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O D 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $\frac{w_L - w_p}{I_p}$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

EXPLANATION OF THE TERM

ROCK QUALITY DESIGNATION (RQD)

The description of bedrock quality for engineering purposes can be inferred from a modified core recovery logging procedure designated as RQD, developed by D.U. Deere.* This classification is based on a modified diamond drill core recovery percentage in which only the pieces of sound core over 4 inches (10 cm) long are counted as recovery. The core must be carefully examined to discount fresh irregular breaks caused by the drilling process (fresh broken pieces are fitted together and counted as one piece). The remaining fragments less than 4 inches (10 cm) length are considered to be due to very close bedding, jointing, fracturing, shearing, or weathering in the rock mass and are not counted. The procedure penalizes the rock where recovery is poor. This is appropriate because poor core recovery usually depicts poor quality rock. In the case of certain shaley sedimentary or thinly foliated metamorphic rocks, the method is not as exact as for other rock types and rock quality requires interpretation by a specialist for the particular engineering application. To minimize the occurrence of core breaks from drilling procedures RQD logging is normally run on core obtained by double or triple tube core barrels and generally of "N" size or greater.

The table below may be used as a general indicator to correlate (RQD) and rock mass quality.

RQD	DESCRIPTION OF ROCK QUALITY
90 - 100	Excellent - intact, very sound, massive
75 - 90	Good - moderately jointed or sound
50 - 75	Fair - blocky and seamy, fractured
25 - 50	Poor - shattered and very seamy or blocky, severely fractured
0 - 25	Very poor - crushed, very severely fractured

*See, for instance:

K.G. Stagg and O.C. Zienkiewicz, "Rock Mechanics in Engineering Practice". New York, Wiley, 1968, Chapter I.

TABLE 1

**HIGHWAY 416 - FOUNDATION INVESTIGATION
COUNTY ROAD 44 UNDERPASS
SUMMARY OF BOREHOLE INFORMATION**

Borehole No.	Borehole Location		Borehole Details					Piezometer Details	
			Ground Surface Elevation (m)	Total Depth (m)	Overburden Depth (m)	Bedrock Surface Elevation (m)	Depth of Bedrock Drilled (m)	Tip Elevation (m)	Groundwater Elevation (m)
8-1	10+026.7	3.5 Right	108.5	11.4	5.0	103.4	6.3	97.1	106.6
8-2	10+026.6	3.1 Left	108.4	6.9	3.7	104.7	3.1	Not installed	
8-3	9+976.9	0.0	109.7	3.5	0.6	109.1	2.9	Not installed	
8-4	9+927.3	3.4 Right	110.0	4.0	1.1	108.9	2.9	106.9	107.3
8-5	9+926.9	3.4 Left	109.5	6.9	0.7	108.8	6.3	Not installed	

Notes:

1. Station values refer to those of the proposed County Road 44 re-alignment.
2. For more detailed borehole information refer to the Record of Borehole Sheets.
3. Groundwater elevations were recorded on December 4, 1990.
4. Quoted elevations are geodetic.

RECORD OF BOREHOLE No 8-1

METRIC

W P 372-89-02 LOCATION Co-ords: 4,982,678.7 N; 374,923.5 ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, BXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE November 12 and 13, 1990 CHECKED BY I.G.

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
108.5	Ground Surface																GR SA SI CL
0.0	Black Fibrous Peat																
108.2																	
0.2	Silty Sand, fine Trace Gravel																
107.4	Brown																
1.1	Sandy Gravel, Some Silt																
106.9	Brown																
1.6	Clayey Silt. Some Sand		1	SS	20												
	Very Stiff to Hard																
	Grey																
			2	SS	31												2 16 66 16
104.4																	
4.1	Silty Sand. Some Gravel, trace clay (Glacial Till)		3	SS	78												
103.4	Very Dense				Rec%												
5.1	Dolostone Dark Grey		4	RC NXL	100												
	Fine to medium grained with closely to moderately spaced thin (1mm) black shale interbeds. Core readily breaks along the shale interbeds to expose a smooth, highly irregular interface.		5	RC BXL	100												UCS = 139MPa
	Vugs with calcite crystals at: 6.3m; 6.4m; 6.6m; 6.7m; 8.8m; 9.1m; 10.1m.																
	Vugs are generally small with some contained wholly within the dimensions of the core.		6	RC BXL	100												
	Small discontinuous solution cavities between 8.7 - 8.8 m.																
			7	RC BXL	100												
			8	RC BXL	93												UCS = 49MPa
97.1																	
11.4	End of Borehole																
	Water level in piezometer measured at elevation 106.56m on December 4, 1990.																
	* Denotes unconfined compression strength.																

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 8-2

METRIC

W P 372-89-02 LOCATION Co-ords: 4,982,672.9 N; 374,920.3 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, BXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE November 13, 1990 CHECKED BY I.C.

OFFICE REPORT ON SOIL EXPLORATION

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					
108.4	Ground Surface																
0.00	Black Fibrous Peat					Not											
0.2	Sand, medium. Some Silt Red/Brown					Noted	108										
107.2																	
1.2	Sandy Gravel																
106.7	Loose Brown		1	SS	31												0 4 83 13
1.7	Clayey Silt, Trace Sand Hard Grey						106										
105.3																	
3.1	Sand and Silt. Tr Gravel Tr Clay (Glacial Till)		2	SS	40/	150 mm											8 45 43 4
104.7	Very Dense Grey				RecX			RQD%									
3.7	Dolostone Dark Grey Fine to medium grained with closely to moderately spaced thin (1mm) black shale interbeds. Core breaks readily along the shale interbeds to expose a smooth highly irregular surface		3	RC BXL	98		104	98									
			4	RC BXL	57			57									
101.5							102										
6.9	End of Borehole																
	Note: Core lost down the borehole at the end of the second core run due to malfunctioning retainer spring.																

+3, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 8-3

METRIC

W P 372-89-02 LOCATION Co-ords: 4,982,651.1 N; 374,965.1 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Washboring, BXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE November 12, 1990 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
109.7	Ground Surface																
0.00	Topsoil Black					Not											
109.5	Gravelly Sand, Some					Not											
0.2	Silt Brown				Rec%												
109.1																	
0.6	Dolostone Dark Grey		1	RC BXL	82												
	Fine to medium grained highly fractured. Reddish brown staining on sub-vertical fractures. Between 1.2m and 1.5m core has many small discontinuous solution cavities. Rock becoming sound at 2.6m.		2	RC BXL	100												
			3	RC BXL	100												
106.2	13mm diameter Vug infilled with calcite at 3.4m.																
3.5	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 8-4

METRIC

W P 372-89-02 LOCATION Co-ords: 4,982,629.7 N; 375,010.0 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Washboring, BXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE November 14, 1990 CHECKED BY I.C.

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
110.0	Ground Surface																
0.0	Silty Sand Topsoil						100										
0.2	Sandy Silt. Some Gravel, occasional cobble.						Bentonite Seal										
108.9	Brown				Rec%												
1.1	Dolostone Dark Grey Fine to medium grained, with closely spaced thin (1mm) shale interbeds. Core breaks readily along the shale interbeds to expose a smooth highly irregular surface. Joint infilled with Sand at 1.8m.		1	RC BXL	96		108										
			2	RC BXL	101		Backfill										
106.0	End of Borehole						106										
4.0	Water level in piezometer measured at elevation 107.3m on December 4, 1990.						Piezometer										

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 8-5

METRIC

W P 372-89-02 LOCATION Co-ords: 4,982,623.5 N; 375,006.9 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Washboring, BXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE November 14, 1990 CHECKED BY I.C.

OFFICE REPORT ON SOIL EXPLORATION

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					
109.5	Ground Surface																
0.0	Silty Sand Topsoil					Not											
109.3	Sandy Silt. Some Gravel					Not											
0.3	Occasional Cobble Brown				Rec%												
108.8																	
0.7	Dolostone Dark Grey		1	RC BXL	75												
	Fine to medium grained with very close to closely spaced thin (1mm) shale interbeds. Core breaks readily along shale interbeds to expose a smooth, highly irregular surface.		2	RC BXL	89		108	83									
	Rock is light Grey between 5.9 - 6.5 m.		3	RC BXL	100		106	100									
	Approximately 13mm diameter vugs with some calcite infilling at: 2.9m; 4.3m; 4.9m; 6.3m; 6.9m.		4	RC BXL	100		106	100									
			5	RC BXL	97		104	97									
102.6																	UCS = 79MPa
6.9	End of Borehole																
	* USC denotes unconfined compressive strength																

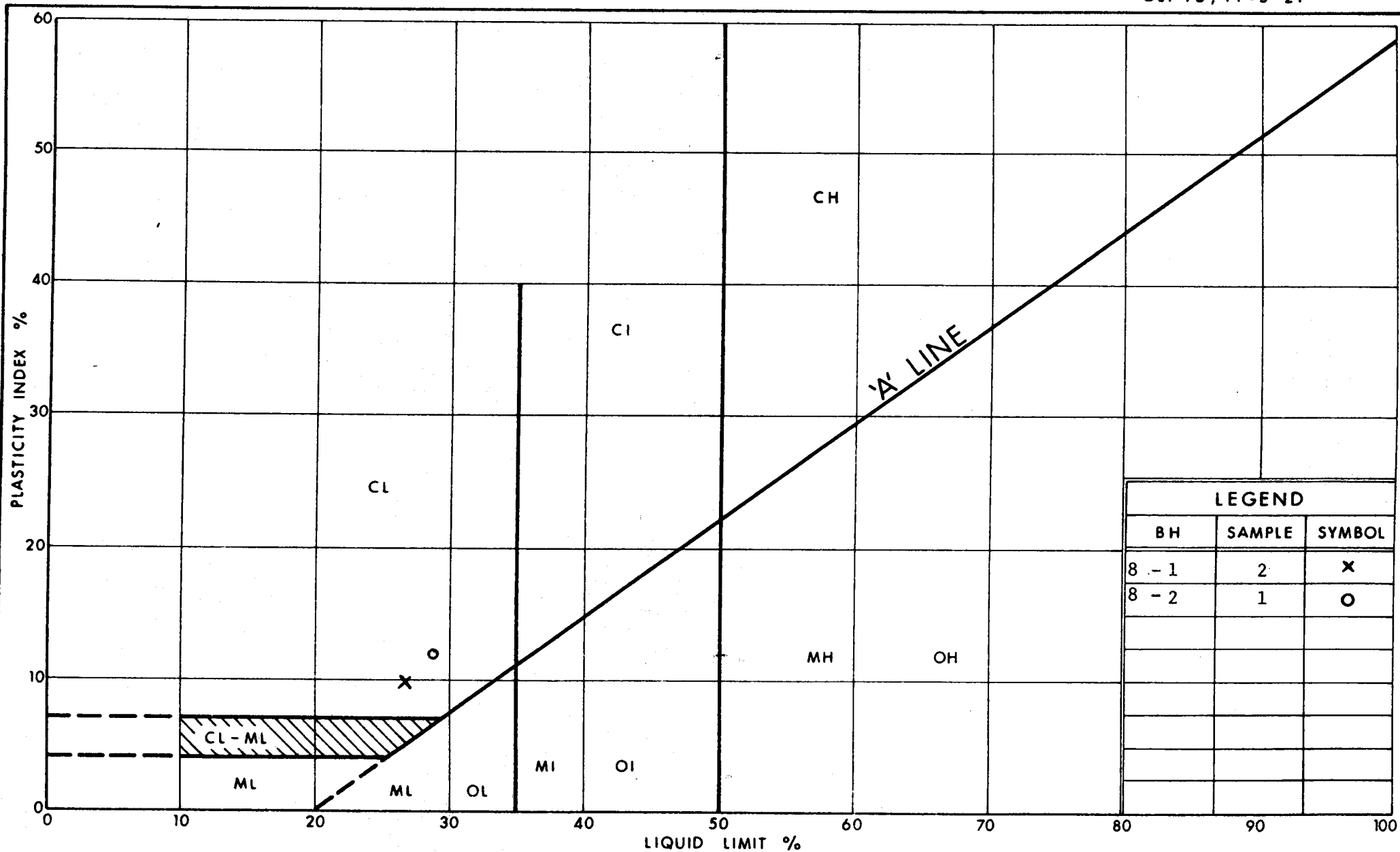
APPENDIX B

Laboratory Test Data

Figure B1: Plasticity Chart - Clayey Silt

Figure B2: Grain Size Distribution - Clayey Silt

Figure B3: Grain Size Distribution - Glacial Till



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PLASTICITY CHART

Clayey Silt

FIG No B1

W P 372-89-02

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

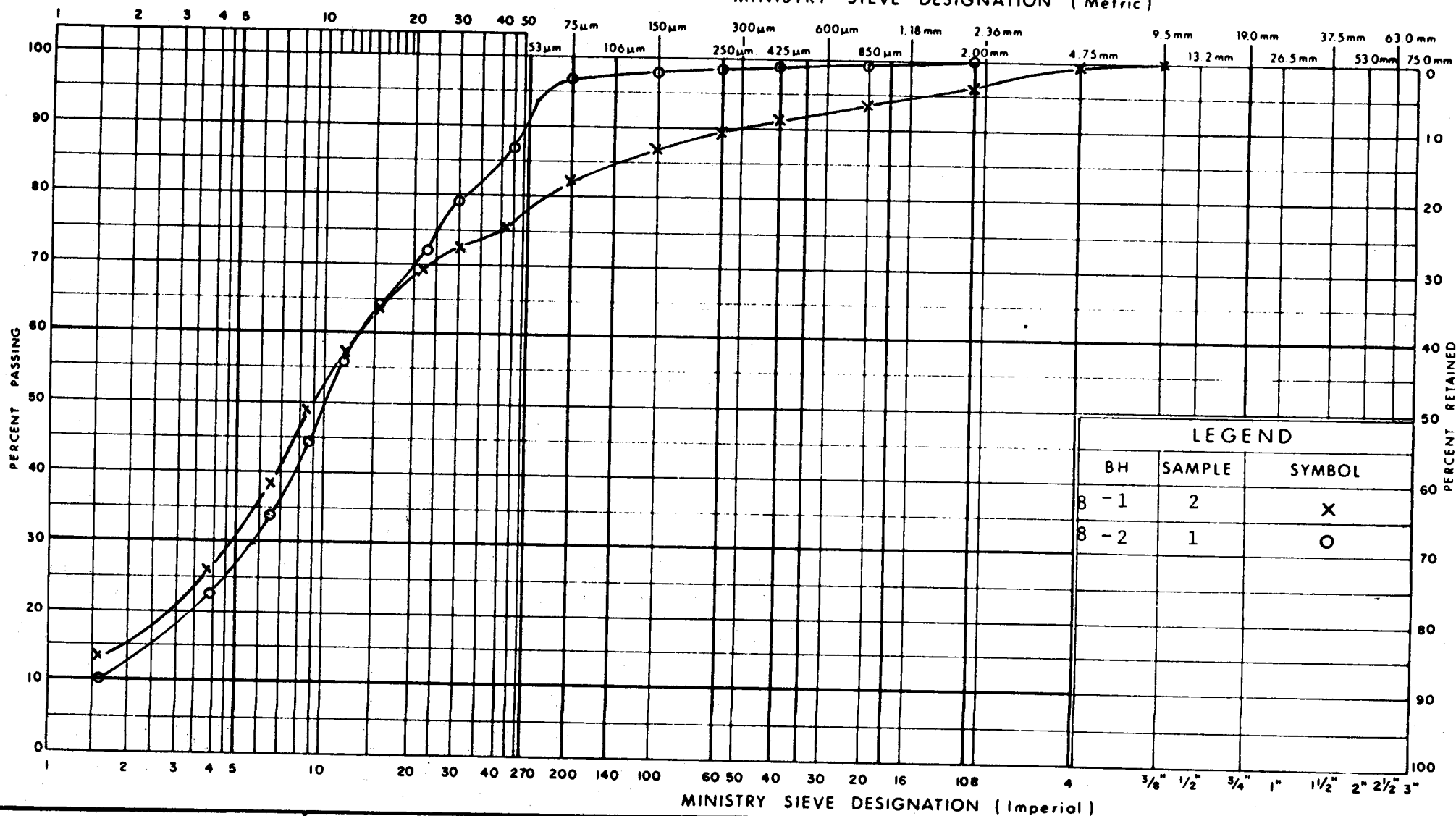
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH	SAMPLE	SYMBOL
B -1	2	x
B -2	1	o

GRAIN SIZE DISTRIBUTION

Clayey Silt, Trace to Some Sand

FIG No B2

W P 372-89-02

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UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

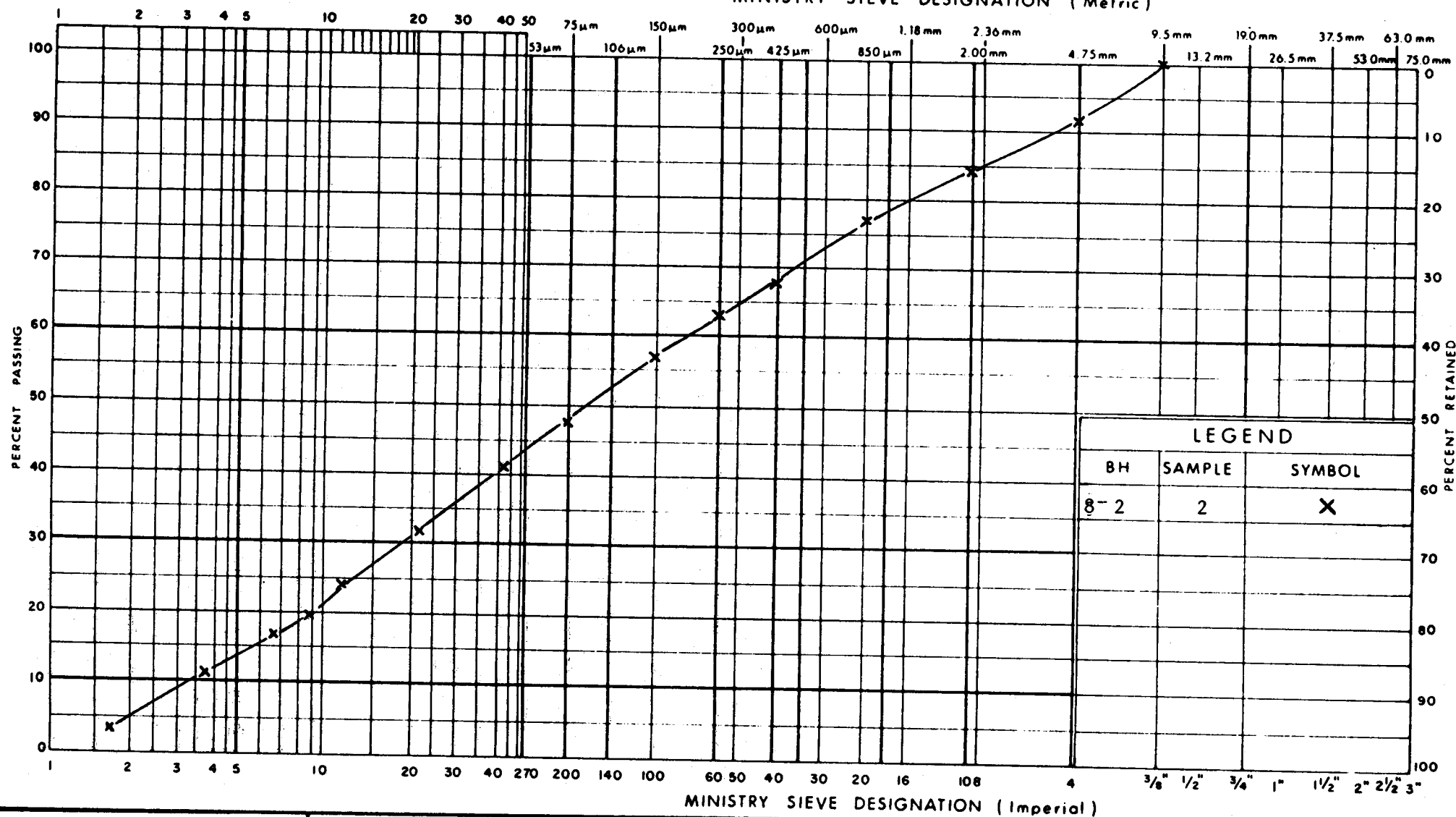
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH	SAMPLE	SYMBOL
8-2	2	X



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Communications

GRAIN SIZE DISTRIBUTION

Glacial Till

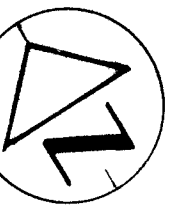
FIG No B3

W P 372-89-02

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

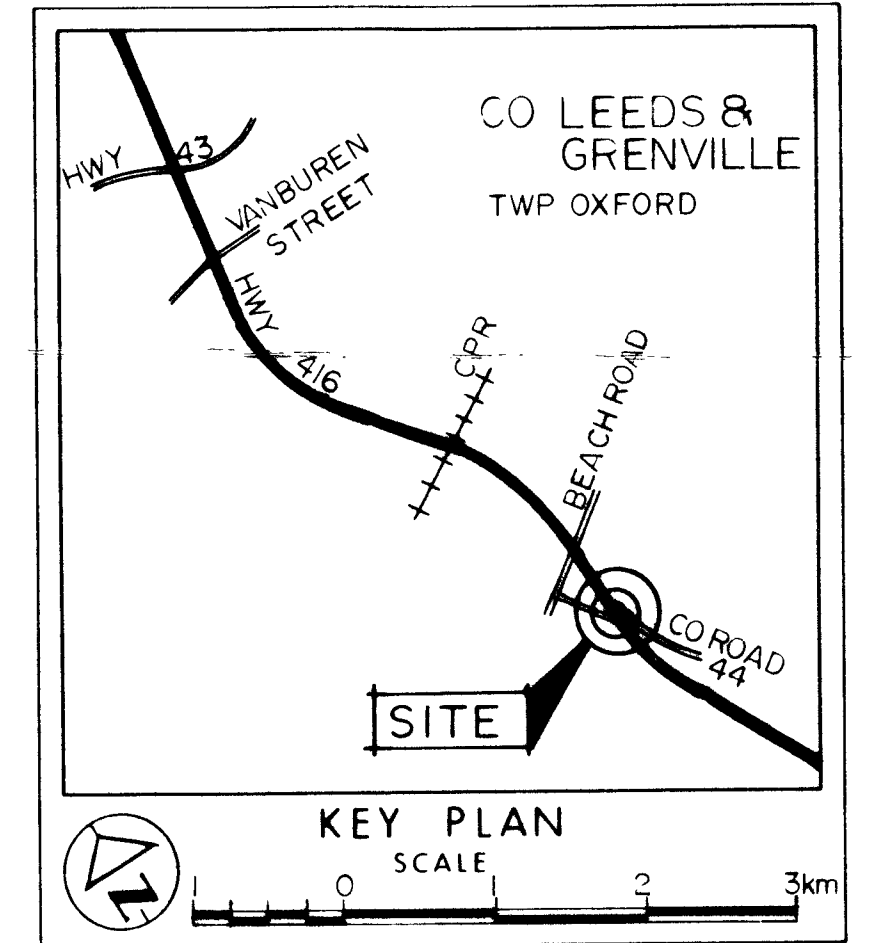
CONT No
WP No 372-89-02

HIGHWAY 416 UNDERPASS
AT COUNTY RD 44
BORE HOLE LOCATIONS & SOIL STRATA



SHEET
A

GEOCON (1991) INC



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
- Piezometer

No	ELEVATION	NORTH	EAST
1	108.5	4982078.7	374923.5
2	106.4	4982672.0	374920.3
3	109.7	4982051.1	374965.1
4	110.0	4982629.1	375010.0
5	109.5	4982623.5	375006.9

NOTE

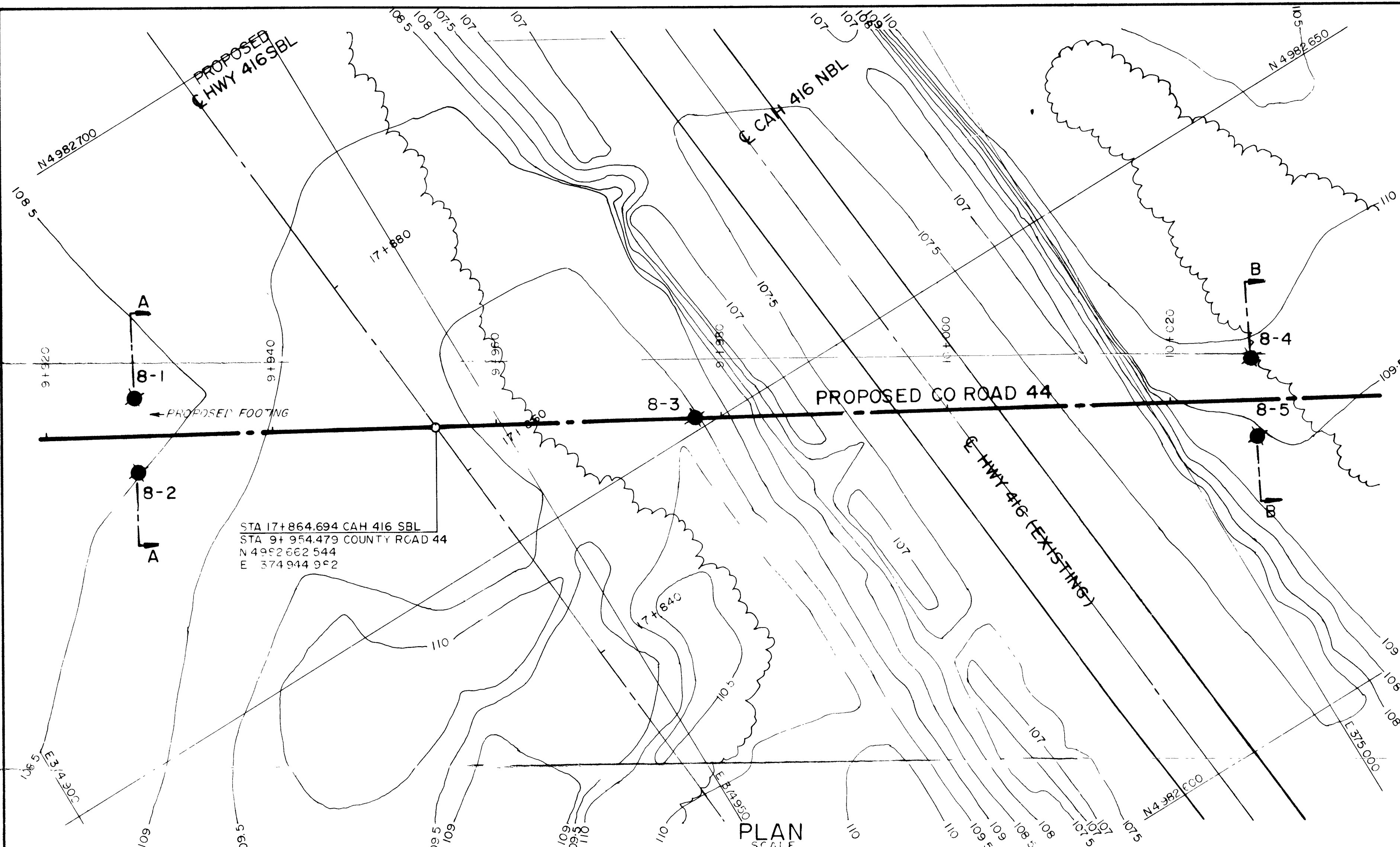
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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 102-2 of Form 100.

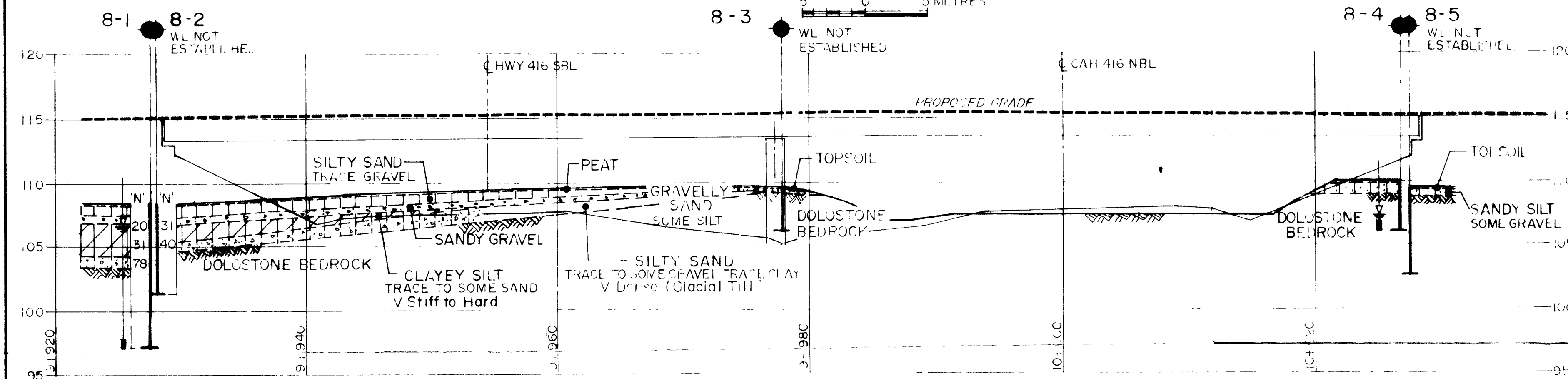
DATE	BY	DESCRIPTION

Geocres No GEOCREs # 31B-6B

HWY No. 416	CHECKED RB	DATE 1991 08 26	DIST 9
SUBMD NK	CHECKED NK	APPROVED	SITE 16-315
DRAWN MZ	CHECKED NK		DWG 3728902-A

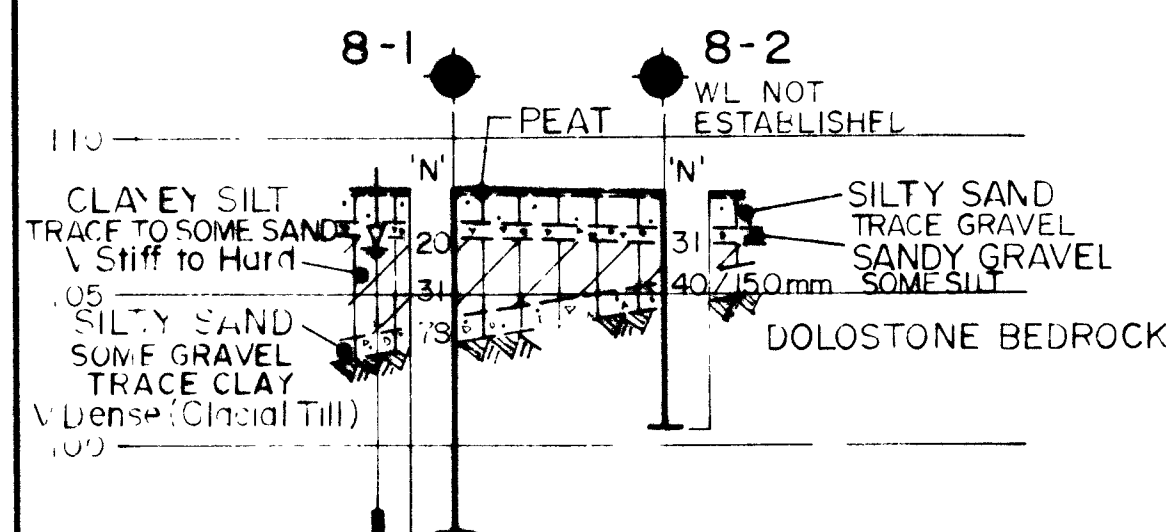


PLAN
SCALE
5 METRES

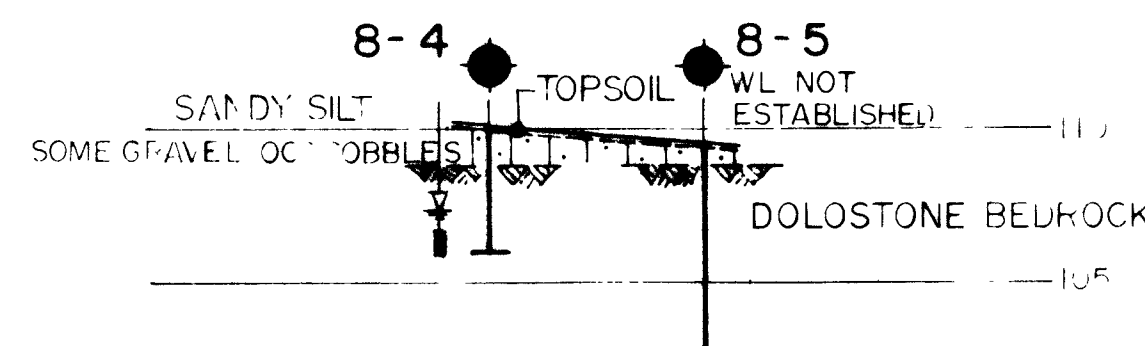


PROFILE CO ROAD 44

SCALE
10 METRES



SECTION A-A



SECTION B-B

SECTIONS
SCALE
10 METRES