



Golder Associates Ltd.

CONSULTING ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION ONTARIO

GEOTECHNICAL INVESTIGATION

PROPOSED HAGER-RAMBO BOX CULVERT

FREEMAN INTERCHANGE

BURLINGTON, ONTARIO

W.P. 516-90-02

DISTRICT 4

Distribution:

13 copies - Ministry of Transportation Ontario
Downsview, Ontario

2 copies - Golder Associates Ltd.
Hamilton, Ontario

MARCH 1991

901-6039-1

TABLE OF CONTENTS

TABLE OF CONTENTS	i
1.0 INTRODUCTION	1
2.0 SITE AND PROJECT DESCRIPTION	1
3.0 INVESTIGATION PROCEDURE	2
4.0 SUBSURFACE CONDITIONS	4
4.1 General	4
4.2 Topsoil	5
4.3 Sand	6
4.4 Clayey Silt (Till)	6
4.5 Shale (Bedrock)	6
4.6 Groundwater Conditions	8
5.0 DISCUSSION	9
5.1 General	9
5.2 Foundations	9
5.3 Excavations	11
5.4 Backfilling	12
5.5 Embankments	13

Appendix "A"

Explanation of Terms used in Report,
Abbreviations and Symbols

Record of Borehole Sheets

Record of Test Pit Sheets

Figures 1 and 2

Drawing WP 5169002-A

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation Ontario (MTO) to carry out a series of site specific subsurface investigations for the design of structures for the proposed reconstruction of the Freeman Interchange in Burlington, Ontario. This report presents the results of a subsurface investigation carried out at the site of the proposed Rambo-Hager Creek box culvert to be constructed beneath the proposed 403 West - QEW East ramp as shown on the Key Plan, Drawing No. 5169002-A.

The purpose of the investigation was to determine the subsurface conditions at the site and to provide geotechnical engineering recommendations for the design of the box culvert. A proposal for carrying out the work was provided in our letter to the Ministry of Transportation Ontario (MTO) dated July 27, 1990.

2.0 SITE AND PROJECT DESCRIPTION

The site of the proposed culvert is located in a relatively flat, grassed area, west of Brant Street and between the existing hydro electric power corridor to the south and the eastbound lanes of the Queen Elizabeth Way (QEW) to the north, in Burlington, Ontario. The location of the site is shown on the Key Plan, Drawing 5169002-A.

The site is situated within the physiographic region of Southwestern Ontario known as the Iroquois Plain. Available geologic information indicates that the overburden in the general area of the site consists of a thin veneer of sands, glacial till and/or residual soil derived from the weathering

of the underlying shale bedrock. The Queenston shale formation which comprises the bedrock in the area generally consists of thinly bedded red shale with occasional bands of grey limestone.

It is understood that the proposed single cell reinforced concrete box culvert will direct water from the Rambo-Hager diversion channel southwards beneath the proposed 403 West and QEW East ramp. The proposed culvert cross-section is understood to be 4.3 metres by 4.3 metres. The length of the culvert will be about 100 metres. The proposed invert elevation will be about 4 metres below the existing ground surface to accommodate the Rambo-Hager diversion channel and the level of a future storm detention pond. Construction of the proposed ramp will result in some 11 metres of fill being placed over the culvert.

3.0 INVESTIGATION PROCEDURE

The field work for this investigation was carried out on August 14 and 15 and on October 12, 1990 during which time two test pits were excavated and seven boreholes were drilled. The locations of the boreholes and test pits are shown on Drawing 5169002-A.

The initial stage of the field work consisted of excavating test pits numbered 1 and 2 at the south and north limits of the proposed culvert respectively. The test pits were excavated using a "Cat 235" hydraulic backhoe supplied and operated by a local contractor. Chunk samples were obtained from the predominant soil strata exposed in the test pits. All of the test pits were loosely backfilled following sampling and logging.

The boreholes were drilled using a track mounted power auger equipped for rotary drilling which was supplied and operated by a specialist drilling contractor. Boreholes numbered 1, 3 and 5 were advanced to the bedrock, through about 1 to 2 metres of overburden, using nominal 150 millimetre outside diameter hollow stem augers. The shale bedrock encountered beneath the overburden in boreholes 1, 3 and 5 was core drilled in NQ size for 3 metres. Boreholes numbered 2, 4, 32 and 33 were drilled to practical auger refusal using hollow stem augers. Standard penetration testing and sampling was carried out within the overburden encountered in the boreholes using 35 millimetre inside diameter split spoon sampling equipment.

Samples of the overburden and the rock core recovered from the test pits and boreholes were taken to our Hamilton laboratory for examination and water content determinations. Grain size analyses and Atterberg limit determinations were carried out on selected samples of the overburden.

The soil and rock stratigraphy encountered in the boreholes and test pits are shown in detail on the Records of Boreholes and Records of Test Pits following the text of this report and on Drawing 5169002-A. The results of the field and laboratory testing are also shown on the Record of Borehole and Record of Test Pit sheets and on Figures 1 and 2.

Groundwater levels were observed in the open boreholes during drilling and in the test pits during and after excavation. A piezometer was installed in borehole 5 as detailed on the Record of Borehole sheet. Notes pertaining to the groundwater conditions observed in the boreholes and test pits are also shown on the Records of Boreholes and Test Pits

and on Drawing 5199002-A.

The locations and ground surface elevations at the borehole and test pit locations have been determined by Golder Associates staff with reference to site specific points and temporary bench marks provided by McCormick Rankin & Associates Limited. The final locations and ground surface elevations of the boreholes were subsequently verified by McCormick Rankin. The elevations provided are understood to be referred to geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes and test pits put down at the site are shown in detail on the Records of Borehole and Record of Test Pit sheets and in summary form on Drawing 5169002-A. The soil boundaries and rock stratigraphy indicated, particularly for the boreholes, are inferred from non-continuous sampling and resistance to auger advance. These boundaries typically represent a transition between one soil or rock type to another and are not intended to define an exact plane of geological change. Conditions will vary between and beyond the borehole and test pit locations.

The subsurface conditions encountered at the site generally consisted of topsoil, glacial till, and completely to highly weathered shale underlain by more competent shale bedrock, at depth.

The following discussion has been simplified in terms of

major soil and rock strata for the purposes of geotechnical design.

It should be noted that due to the relatively soft and weathered nature of the Queenston shale formation, particularly the upper zone, together with the effects of glacial overriding at the bedrock surface, it is difficult to accurately define the bedrock surface both from a geological and a contractual standpoint.

During the course this investigation, a stratigraphic unit directly overlying the shale bedrock, which in strictly geological terms is described as deformation till, has been encountered. This stratigraphic unit consists of an imbricate embedment of fragments of bedrock in a till matrix which has been formed from glacial overriding of the parent bedrock. This till is characterized by the presence of rounded and sub-angular clasts of the parent rock with non horizontal bedding. Based on the consistency, the relatively high penetration resistance encountered in the boreholes and the difficulty of excavation experienced in the test pits, this stratum could be contractually interpreted as a zone of the bedrock and for this reason has been referred to in this report as an upper zone of the bedrock formation.

4.2 Topsoil

Black sandy topsoil was encountered at the ground surface in all of the test pits and boreholes put down at the site, the thickness of the topsoil layer ranged from about 0.2 to 0.4 metres. Although not encountered in the boreholes and test pits, there is the possibility of the presence of localized surficial fill areas being encountered.

4.3 Sand

A surficial layer of sand was encountered beneath the topsoil in test pit 1 and borehole 1. The sand sampled from test pit 1 had a natural water content of about 4 percent. A single N value of 14 blows per 300 millimetres was determined in the sand in borehole 1.

4.4 Clayey Silt, trace to some sand, occ. gravel, occ. Cobbles (Till)

Glacial till was encountered below the surficial soils in test pit 1, and in boreholes 1, 2 and 32. The till typically had a clayey silt gradation. The N values determined in standard penetration testing carried out in the till ranged from 48 to 115 blows per 300 millimetres. The till has natural water contents of about 10 to 13 percent and plastic and liquid limits of about 17 and 26 percent respectively, based on a single Atterberg Limit determination as shown on the Plasticity Chart, Figure 2. A grain size distribution curve for a sample of the till is shown on Figure 1.

The till matrix as indicated by the gradation curve is fine grained in nature and major concentrations of coarse particles such as boulders were not encountered during this investigation. This does not necessarily mean that the coarser particle sizes are not present at random or in concentrations within the deposit since till is an inherently variable material.

4.5 Shale, Completely to Slightly Weathered (Bedrock)

At the borehole and test pit locations, the overburden

materials are underlain by shale bedrock of the Queenston Formation. Bedrock was encountered between about elevations 100.5 and 103.5 metres, or at depths of from about 0.3 to 2.3 metres below the existing ground surface. A bedrock surface sloping slightly downward toward the south is inferred.

In test pit 2 and in boreholes 3, 4, 5 and 33 a completely to highly weathered zone of shale, typically about 1 to 1.2 metres in thickness was encountered beneath the topsoil. The shale within this weak upper zone was generally completely decomposed; however, the rock fabric and structure were recognizable. The highly to completely weathered zone was penetrated by augering in boreholes 3, 4, 5 and 33 and by excavating in test pit 2.

The upper zone of bedrock encountered in boreholes 1, 2 and 32 and in test pit 1 is geologically referred to as deformation till, but has been described as an upper zone of the bedrock as outlined previously.

More competent shale was encountered in boreholes 1, 3 and 5, between about elevations 100 and 103 metres or at depths of about 1 to 2.3 metres below the existing ground surface. The bedrock core recovered from boreholes 1, 3 and 5, generally consisted of moderately to faintly weathered thinly bedded reddish brown shale, interbedded with weathered to slightly weathered thinly bedded light grey, fine grained argillaceous limestone up to about 0.5 metres in thickness.

The rock core recovered from the boreholes generally exhibited a relatively high degree of fracturing. However, the quality of the rock core recovered generally improved with depth. The total core recovery (TCR) ranged from about

83 to 97 percent. The solid core recovery (SCR) ranged from about 45 to 66 percent for the upper 1.5 metres of rock core, and from about 77 to 82 percent for the lower 1.5 metres of rock core. Similarly the rock quality designation (RQD) ranged from about 23 to 52 percent in the upper 1.5 metres of rock core compared to 56 to 80 percent for the lower 1.5 metres of rock core.

4.6 Groundwater Conditions

Groundwater was not encountered during the field drilling/digging operations in boreholes 4, 32 and 33 terminated at auger refusal, or in test pit 2 which was excavated into the highly weathered shale bedrock. Boreholes 1, 3 and 5 were dry to the depth of auger refusal, corresponding to about elevations 97.4, 101.5 and 102.2 metres respectively. Groundwater was encountered at about elevation 99.8 metres or about 3 metres below the existing ground surface in borehole 2. Groundwater seepage was observed at about elevation 100.3 metres in test pit 1, corresponding to a depth of about 2 metres below the existing ground surface. The groundwater level was measured at about elevation 101.3, or at a depth of about 2.5 metres below the existing ground surface, in the piezometer in borehole 5, about 1 month after the completion of drilling.

It should be noted that the piezometric groundwater level within the subsoil and underlying bedrock is subject to fluctuation not only due to precipitation conditions, but also due to seasonal variations.

5.0 DISCUSSION AND DESIGN RECOMMENDATIONS

5.1 General

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation. The geotechnical engineering parameters given in the following discussion are intended for design purposes only. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the works should make their own interpretation of the subsurface information provided as it affects their proposed construction methods, equipment selection, scheduling and the like.

5.2 Foundations

It is understood that a single cell reinforced concrete box culvert is proposed. The invert elevation will be at about elevation 99 metres. The cross sectional dimensions of the culvert will be 4.3 metres by 4.3 metres. The height of the proposed embankment fill above the culvert is some 11 metres.

Based on the seven boreholes and two test pits put down at the site, it is anticipated that the invert level for the culvert will be within the moderately to slightly weathered shale bedrock over the entire length.

It is considered that foundations constructed within the undisturbed moderately to slightly weathered shale, at or below elevation 99 metres, could be designed using a factored bearing capacity at ULS of as much as 1000 kilopascals for

footings at least 1 metre in width. For footings less than 1 metre in width the bearing capacity should be reduced proportionately.

Serviceability limit states is not relevant to spread footings placed on the shale bedrock since the stresses required to produce detrimental settlement of the structure founded on bedrock will be larger than the value given for the factored bearing capacity at ULS.

In order to achieve the design bearing pressure, it is essential that all material at the founding level which is loosened during excavation be removed from the base of the excavation. The shale is highly fractured and is particularly susceptible to weathering from exposure to air, water, and to any significant construction traffic. Effective control of groundwater during construction will therefore be a critical aspect to preserving the integrity of the bedrock surface.

To preserve the integrity of the bearing surfaces, a minimum 150 millimetre thick protective layer of lean concrete should be placed within four hours of final excavation. Any loosened or disturbed material must be removed prior to placing the protective layer of concrete.

The shale bedrock should be regarded as being frost susceptible. Care and adequate protection will therefore be required during winter construction to prevent any freezing of the rock below founding grade.

For design purposes, a minimum of 1.2 metres of cover should be provided to the bearing surface for frost protection. The

minimum cover can be achieved by increasing the thickness of the lean concrete layer such that the combined thickness of the culvert bottom and the underlying lean concrete is at least 1.2 metres.

Adequate provision must be made to control erosion and undermining beneath the culvert base at each end and along the channel bottom. This can be achieved by a concrete apron at each end or by a graded filter system with the appropriate larger sizes forming the channel bed in the area(s) adjacent to the box culvert.

5.3 Excavations

Based on the results of the current investigation, excavations for the culvert will extend through the various overburden materials and up to about 3 metres into the underlying shale bedrock. Some groundwater seepage into the excavation should be anticipated.

As discussed previously, the shale bedrock is highly prone to rapid deterioration in the presence of free water. Adequate control of groundwater is therefore considered essential during final excavation and preparation of the bearing surface for placement of the mud mat and prior to mass foundation concreting operations.

All excavations must conform to the current Occupational Health and Safety Act and care should be taken to direct surface runoff away from open excavations. Excavation side slopes within the overburden should not exceed an inclination of 1 horizontal to 1 vertical unless adequate temporary support is provided. For cuts of less than 3 metres below

the bedrock surface, near vertical side slopes are appropriate for temporary excavations in rock. However, the condition of the excavation side slopes should be monitored on a routine observation basis during construction as some ravelling of the slopes will develop due to deterioration of the exposed rock.

5.4 Backfilling

Backfill around the culvert and to 0.5 metres above the culvert should consist of free draining granular material such as Ontario Provincial Standard Specifications (OPSS) Granular "A" or Granular "B". The minimum limits of the granular backfill are as outlined in Section 6-9.6.1 of the Ontario Highway Bridge Design Code (OHBDC). Drainage of the backfill should be provided by filtered weep holes through the culvert. The granular backfill should be placed and compacted in accordance with the current OPSS practices and MTO directives.

The culvert should be designed for the full overburden pressure due to the embankment fill as well as any equivalent surcharge loading. Lateral pressures on the culvert walls will be strongly influenced by the deformation of the culvert walls during construction and the relative stiffness of the culvert walls, roof and corners. It is anticipated that the lateral earth pressures in terms of Ultimate Limit States could vary from 0.6 to 1.0 (depending on the stiffness of the culvert) and in terms of Serviceability Limit States could vary from 0.5 to 1.0 (depending on the stiffness of the culvert).

The following earth pressure parameters may be used in the

calculation of lateral earth pressures in accordance with the Ontario Hydro Bridge Design Code.

Granular "A"	
Unit Weight	22.8 kN/m ³
ϕ	35°
Granular "B"	
Unit Weight	21.2 kN/m ³
ϕ	30°

5.5 Embankments

It is understood that engineered fills of up to about 11 metres above the existing ground surface will be required for the proposed ramps.

Provided that the culvert is founded as outlined in section 5.2, and the embankment fill is constructed as outlined below, no deep seated stability problems are anticipated within the subsoil or in the embankment fill itself. The subgrade at the site is considered competent to support the embankment loading provided that all topsoil and fill materials and disturbed zones are removed prior to filling and that no softening of the subgrade occurs during construction.

The embankments may be constructed as detailed in the relevant OPSS specifications and MTO directives with the following exceptions:

- All topsoil and any existing fill materials should be stripped from all areas of embankment construction.
- The subgrade should be proof-rolled to identify any loose or softened areas which require subexcavation prior to placing fill.

For embankments constructed as outlined above and for embankment heights of up to about 11 metres, post construction settlement in the range of about 150 to 200 millimetres should be anticipated. The effect of the post construction deformation can be minimized by constructing the embankments as far in advance of pavement construction as practicable.

For the height of fills required and with the use of clayey silt material for construction of the embankments as outlined above, side slope inclinations no steeper than 2.5 horizontal to 1 vertical should be used to minimize the potential surficial instability. The completed fill side slopes should be blanketed with an appropriate vegetation cover.

It should be brought to the attention of the contractor by way of a special provision to the contract, that all of the test pits put down in this investigation and in any other preconstruction investigations, are to be located in the field to determine whether they are within the embankment alignment. Test pits within the embankment areas should be re-excavated and backfilled with appropriate material placed and compacted as outlined above.

GOLDER ASSOCIATES LTD.

J. G. Muckle, P. Eng.

J. L. Seychuk, P. Eng.

att.

For the height of fills required and with the use of clayey silt material for construction of the embankments as outlined above, side slope inclinations no steeper than 2.5 horizontal to 1 vertical should be used to minimize the potential surficial instability. The completed fill side slopes should be blanketed with an appropriate vegetation cover.

It should be brought to the attention of the contractor by way of a special provision to the contract, that all of the test pits put down in this investigation and in any other preconstruction investigations, are to be located in the field to determine whether they are within the embankment alignment. Test pits within the embankment areas should be re-excavated and backfilled with appropriate material placed and compacted as outlined above.

GOLDER ASSOCIATES LTD.

V. C. Hanemayer, P. Eng.

J. G. Muckle, P. Eng.

J. L. Seychuk, P. Eng.

att.

APPENDIX "A"

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

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_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

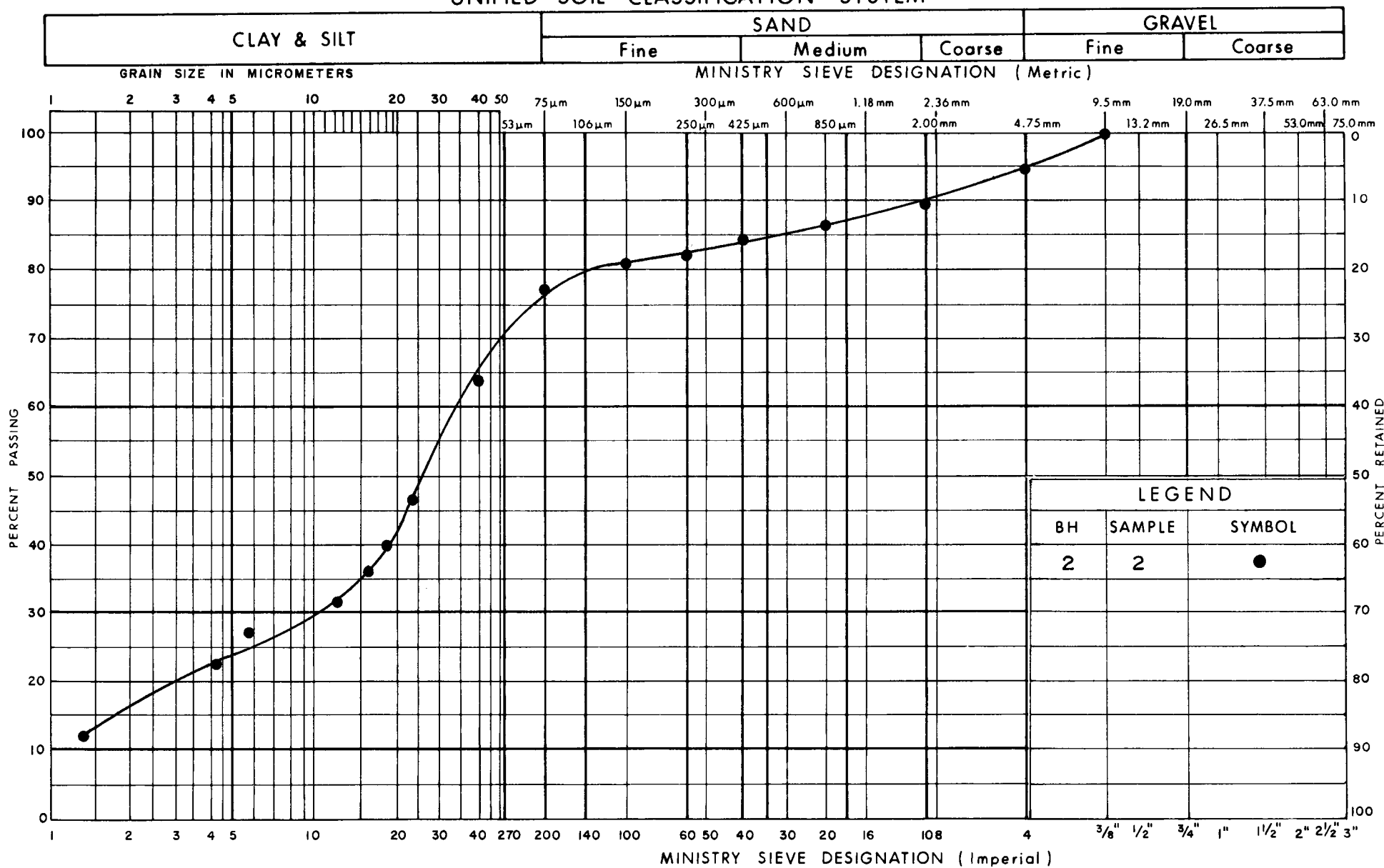
STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_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

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 = $\frac{w_L - w_p}{w - w_p}$	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						

UNIFIED SOIL CLASSIFICATION SYSTEM

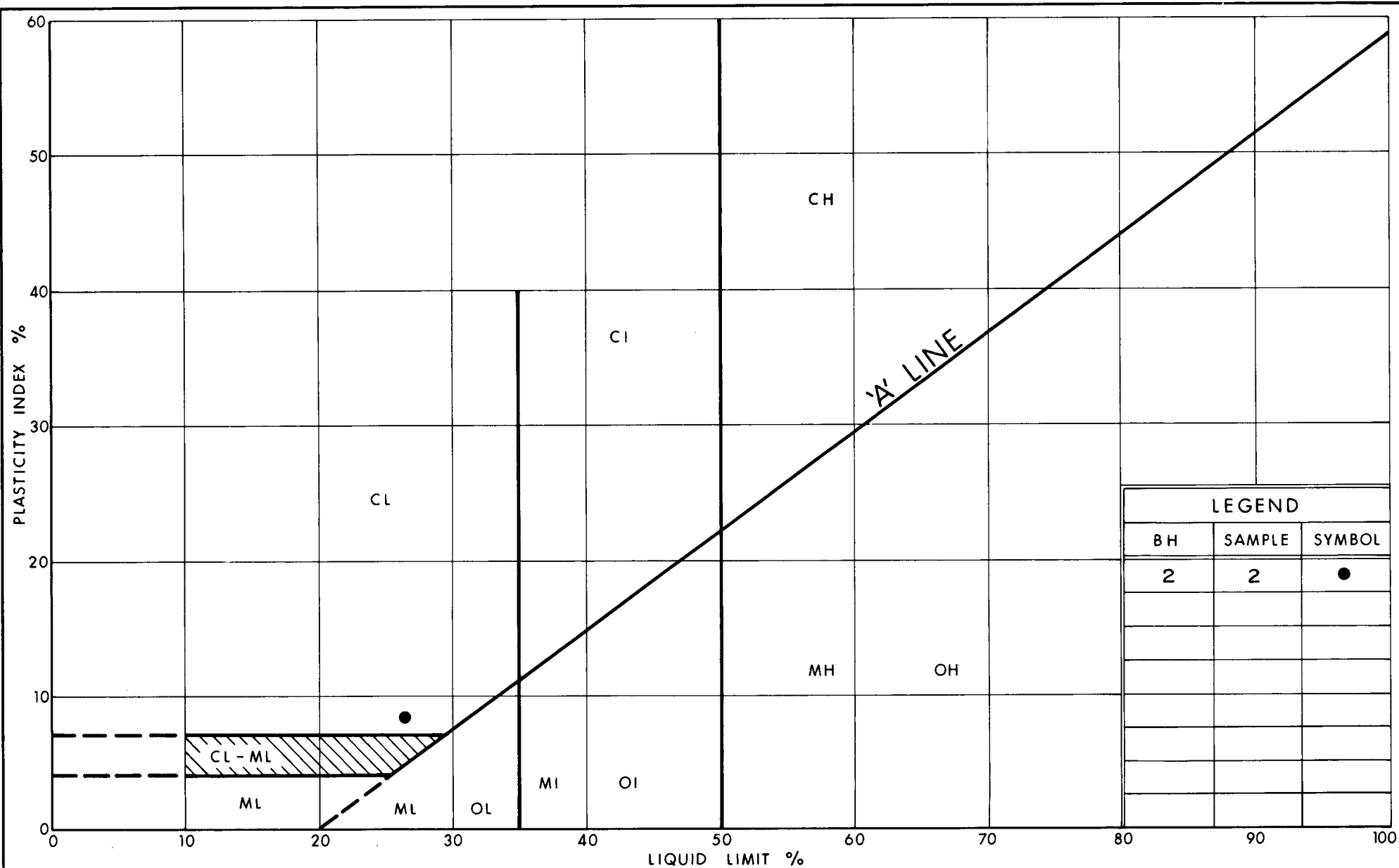


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

FIG No 1
WP 516-90-02



LEGEND		
BH	SAMPLE	SYMBOL
2	2	●



Ministry of
Transportation

Ontario

PLASTICITY CHART CLAYEY SILT TILL

FIG No 2
W P 516 - 90 - 02

METRIC

W P 516-90-02

LOCATION Co-ordinates N,4799,591 E278,352

ORIGINATED BY VCH

DIST 4 HWY QEW/403

BOREHOLE TYPE Hollow Stem Auger, NQ Rock Core

COMPILED BY MHW

DATUM Geodetic

DATE August 14, 1990

CHECKED BY YCH

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 (%)		
								20 40 60 80 100										10 20 30		
102.3	Ground Surface																			
0.0	Topsoil		1	SS	14															
0.2	Sand, Compact, Brown																			
0.5	Clayey Silt some gravel, some sand (Till)		2	SS	115															
100.5	Hard Reddish Brown																			
1.8	Shale. Highly to completely weathered (Deformation Till)		3	AS																
100.0	Reddish Brown																			
2.3	Shale Bedrock Moderate to slightly weathered thinly bedded, Reddish Brown interbedded with fine grained argillaceous limestone		4	NQ RC	* TCR= 93% SCR= 66% RQD= 52%															
			5	NQ RC	* TCR= 88% SCR= 82% RQD= 52%															
96.6																				
5.7	End of Borehole																			

⁺₃, x⁵ : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,621 E278,344 ORIGINATED BY VCH
DIST 4 HWY QEW/403 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MHW
DATUM Geodetic DATE August 14, 1990 CHECKED BY VCH

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					
102.8	Ground Surface															
0.0	Topsoil															
0.3	Clayey Silt (Till) some sand occasional gravel		1	SS	48									○		
			2	SS	100/180mm									○		5 18 62 15
100.5	Hard Reddish Brown				70/100mm											
2.3	Shale. Highly to completely weathered (Deformation Till)		3	SS												
99.7	Reddish Brown		4	SS										○		
3.1	End of Borehole				60/76mm											
					W.L. elev. encountered at 99.8 on Aug. 14/90											
						98										

OFFICE REPORT ON SOIL EXPLORATION

$+^3, x^5$: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 3

METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,648.3 E278,336.5 ORIGINATED BY VCH
DIST 4 HWY QEW/403 BOREHOLE TYPE Hollow Stem Auger, NQ Rock Core COMPILED BY MHW
DATUM Geodetic DATE August 14, 1990 CHECKED BY VCH

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	VALUES			20	40	60	80	100					
103.1	Ground Surface																
102.8	Topsoil																
0.3	Shale Bedrock. Highly to completely weathered Reddish Brown		1	SS	90		102										
					75/150mm												
101.3			2	SS													
1.8	Shale Bedrock Moderately to completely weathered thin to medium bedded Reddish Brown interbedded with argillaceous limestone		3	RC NQ	*TCR= 83% SCR= 47% RQD= 47%	Borehole Dry to Elev. 101.5 Aug. 14/90											
			4	RC NQ	*TCR= 93% SCR= 77% RQD= 60%		100										
98.5																	
4.6	End of Borehole						98										

* TCR: Total Core Recovery
SCR: Solid Core Recovery
RQD: Rock Quality Designation

OFFICE REPORT ON SOIL EXPLORATION



METRIC

W P	516-90-02	LOCATION	Co-ordinates N4,799,668.5 E278,331.5	ORIGINATED BY	VCH
DIST	4 HWY QEW/403	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MHW
DATUM	Geodetic	DATE	August 14, 1990	CHECKED BY	VCH

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

⁺₃, x⁵ : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 5						METRIC					
W P 516-90-02		LOCATION Co-ordinates N4,799,687 E278,325.5		ORIGINATED BY VCH							
DIST 4 HWY QEW/403		BOREHOLE TYPE Hollow Stem Auger; NQ Rock Core		COMPILED BY MHW							
DATUM Geodetic		DATE August 15, 1990		CHECKED BY VCA							
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	NUMBER	TYPE			'N' VALUES		20 40 60 80 100			
103.7	Ground Surface										
103.4	Topsoil										
0.3	Shale Bedrock completely to highly weathered										
102.6	Reddish Brown	1	SS	96	Bentonite Seal						
1.10	Shale Bedrock Moderately to slightly weathered thinly to medium bedded Reddish Brown interbedded with fine grained argillaceous limestone	2	NQ RC	* TCR= 92% SCR= 45% RQD= 23%	102						
		3	NQ RC	* TCR= 97% SCR= 98% RQD= 56%	100						
99.0					Piezo Tip						
4.7	End of Borehole				98						
* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation											

METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,593 E278,362.5 ORIGINATED BY VCH
DIST 4 HWY 403/OEW BOREHOLE TYPE Solid Stem Augers COMPILED BY MHW
DATUM Geodetic DATE October 12, 1990 CHECKED BY VCH

[illegible]

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 33

METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,684.7 E278,318 ORIGINATED BY VCH
DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Augers COMPILED BY MHW
DATUM Geodetic DATE October 12, 1990 CHECKED BY VCA

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									
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
WATER CONTENT (%)																	
103.9	Ground Surface																
103.7	Topsoil																
0.2	Shale Bedrock completely weathered		1	SS	62												
102.4	Reddish Brown																
1.5	End of Borehole																
							102										
							Borehole dry during drilling										

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF TEST PIT No. 1

METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,586.8 E278,347.5 ORIGINATED BY VCH
DIST 4 HWY QEW/403 BOREHOLE TYPE Backhoe Dug - Cat 235 COMPILED BY MHW
DATUM Geodetic DATE August 14, 1990 CHECKED BY VCH

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										SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		
102.3	Ground Surface																			
102.0	Topsoil		1	CS	--		102													
0.3	Sand Brown		2	CS	--															
0.5	Clayey Silt (Till) trace sand		3	CS	--															
101.2	Occ. Cobbles Brown																			
1.1	Clayey Silt, trace sand, some gravel																			
100.5	(Till) Reddish Brown																			
1.8	Shale, Highly to completely weathered		4	CS	--															
100.0	(Deformation Till)																			
99.9	Shale Bedrock						100													
2.4	End of Borehole																			

METRIC

W P 516-90-02 LOCATION Co-ordinates N4,799,795 E278,331 ORIGINATED BY VCH
DIST 4 HWY QEW/403 BOREHOLE TYPE Backhoe Dug - Cat 235 COMPILED BY MHW
DATUM Geodetic DATE August 14, 1990 CHECKED BY VCH

[illegible]

⁺₃, x⁵ : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10