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STR. SITE No. 3-357

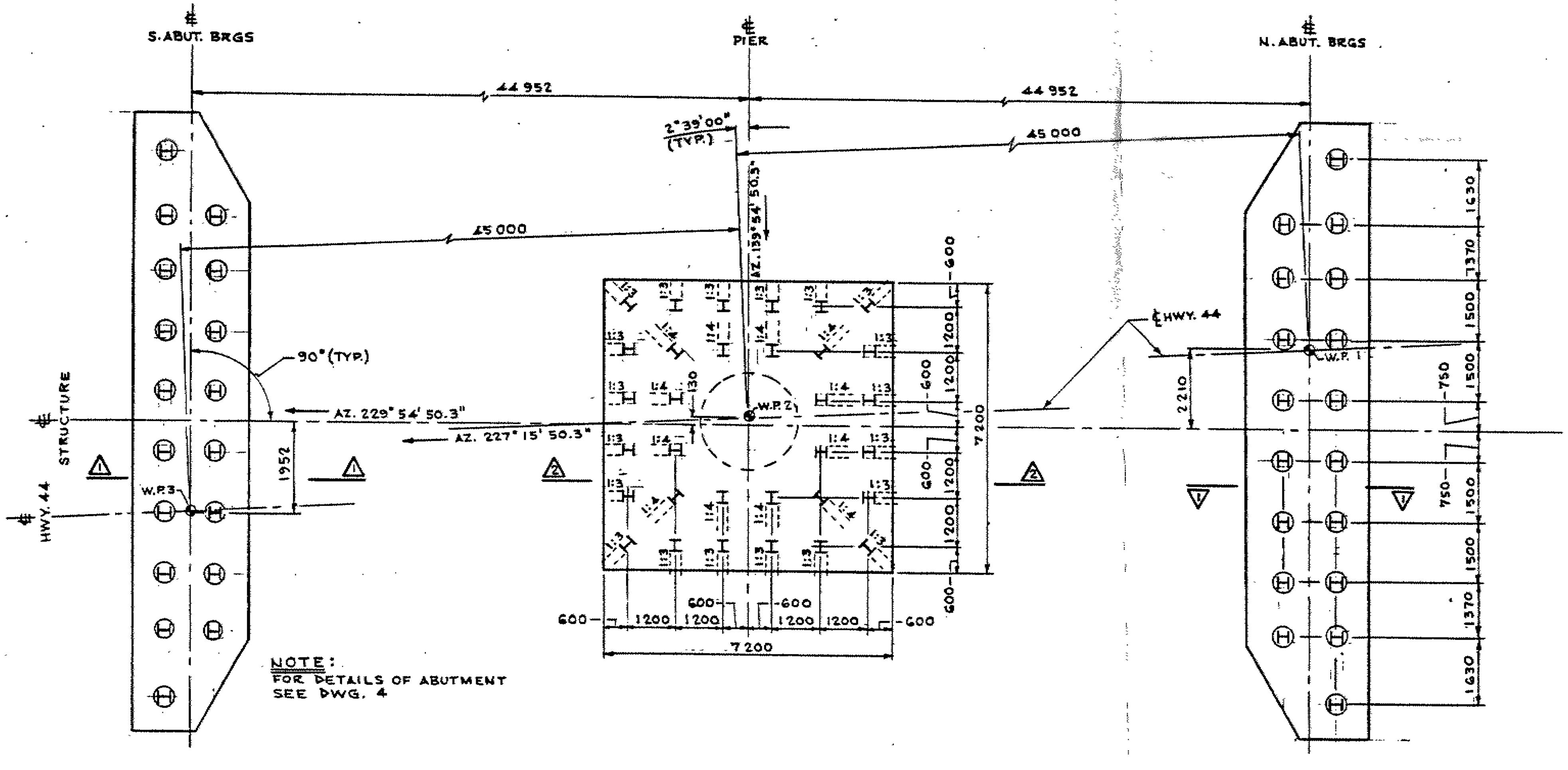
HWY. No. 17

LOCATION Hwy 17 & Hwy 44 Interchange

No. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



FOOTING LAYOUT
1:75

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
WP No 34-81-02

HWY. 44 INTERCHANGE UNDERPASS

FOOTING LAYOUT

SHEET

NOTES:

1. ALL PILES ARE HP 310 x 110.
2. PILE SPACING FOR NORTH & SOUTH ABUTMENTS ARE TYPICAL.
3. PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
4. DRIVING SHOES TO BE PROVIDED ON ALL PILES.
5. PILES TO BE DRIVEN TO BEDROCK.
6. PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTH BELOW CUT-OFF.

PILE DESIGN DATA

MAX. COMBINED FACTORED LOADS =

ABUTMENTS

NORTH	SLS II	=	710 kN
	ULS	=	990 kN
SOUTH	SLS II	=	850 kN
	ULS	=	1200 kN

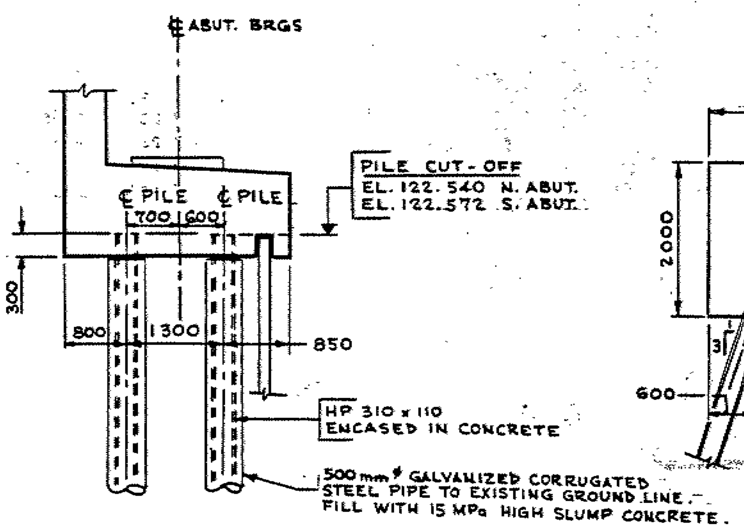
PIER

SLS II	=	1150 kN
ULS	=	1600 kN

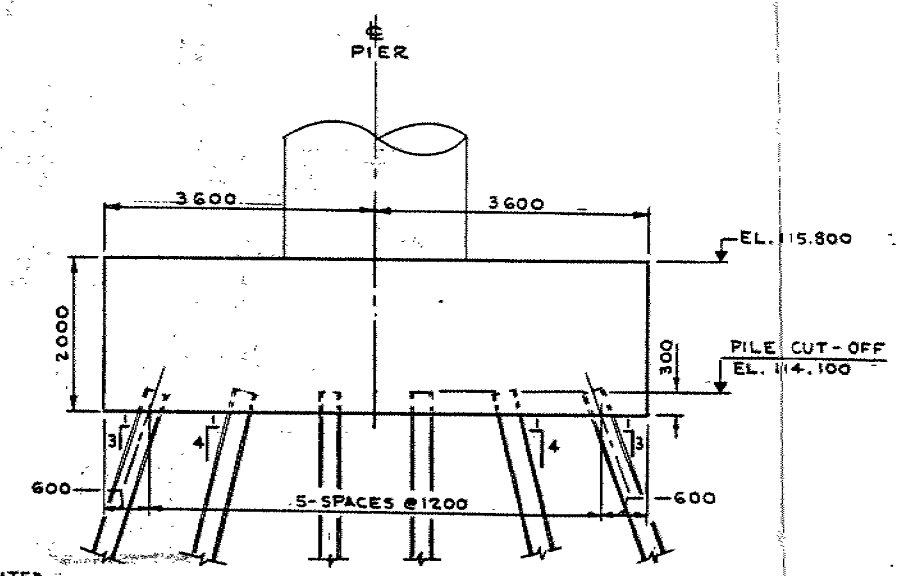
POINT	STATION	CO-ORDINATES	
		NORTH	EAST
W.P. 1	9+955.000	5 019 131.778	339 559.852
W.P. 2	10+000.000	5 019 101.243	339 526.801
W.P. 3	10+045.000	5 019 070.702	339 493.748

LOCATION	ROW	No	*LENGTH (m)	BATTER
NORTH ABUTMENT	FRONT ROW	8	29.0	VERT.
	BACK ROW	10	29.0	VERT.
PIER	SOUTH SIDE	6	21.0	1:3
	NORTH SIDE	6	21.0	1:3
	EAST SIDE	4	21.0	1:3
	WEST SIDE	4	21.0	1:3
	INSIDE	12	20.5	1:4
SOUTH ABUTMENT	FRONT ROW	8	25.5	VERT.
	BACK ROW	10	25.5	VERT.

* SEE NOTE 6



1 1:50



2 1:50

LICENSED PROFESSIONAL ENGINEER
S. S. BHATIA
ONTARIO

APPLICABLE STANDARD DRAWINGS

DD-3301 SPLICE AND DRIVING SHOE DETAILS

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN S.B.	CHK	CODE OHBDC-83	LOAD CLASS A	DATE NOV./90
DRAWN G.F.Mc	CHK S.B.	SITE 3-357	STRUCT	SCHEME	DWG. 3

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 91-52

Ministry of Transportation
Ontario

INDEX

Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

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

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

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

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 $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

REPORT TO

MINISTRY OF TRANSPORTATION OF ONTARIO
FOUNDATION DESIGN SECTION

SUBSURFACE GEOTECHNICAL INVESTIGATION
PROPOSED HIGHWAY 17 AND 44, UNDERPASS
OTTAWA ONTARIO

WP 34-81-02 SITE 3-357

1.0 INTRODUCTION

The Ministry of Transportation of Ontario, propose to upgrade the existing Highway 17, west of Ottawa from two lanes to a four lane expressway. In order to achieve this work, several regional roads in the area require grade separations over the proposed expressway. Geocon Inc. was requested to conduct a subsurface geotechnical investigation for one of these proposed grade separations, located at the intersection of Regional Road 44 and Highway 17.

2.0 SITE GEOLOGY

The site is located adjacent to the Ottawa River on a sand plain along the near shore area of the historical Champlain Sea (Chapman and Putnam 1983). Expected soil conditions at site consist of a thin veneer of cohesionless material overlying significant depths of marine clay. These deposits are in turn underlain by glacial till which is overlying bedrock. These general soil conditions have been confirmed by previous subsurface investigations in the area (Ministry of Transportation of Ontario - Geo-Cres library).

The bedrock geological mapping at the proposed site, as presented in the Ontario Geological Survey (O.G.S.) Map P 2726, indicates that the underlying bedrock consists of limestone of the Verulam Formation of the Middle Ordovician Period. Typically, this formation consists of fine grained limestone bedrock interbedded with shale layers up to 100 mm thick. Outcrops of the formation occur approximately 2 km west of the proposed site.

3.0 SITE AND PROJECT DESCRIPTION

The site of the proposed overpass structure is located at the intersection of Highway 44 and 17, approximately 35 km west of Ottawa, Ontario. The site is located on a generally flat plain locally traversed by three low level embankments.

The largest of the embankments is that associated with the north-west trending Highway 17 (Drawing T11600.1) which is approximately 2.0 m above the existing ground level. In the east-west direction, two embankments intersect obliquely with the larger Highway 17 embankment. The smallest of these embankments, associated with the previous Highway 44 alignment, is generally less than 1.0 m above the existing ground level. The second, associated with the diversion of Highway 44 to a location some 60 m north of the previous location is slightly higher as it grades up from surrounding ground level to meet Highway 17 at grade. A drainage ditch traverses the site in a roughly north easterly direction.

The proposed grade separation of the two highways will be achieved using a new overpass structure located on the old alignment of Highway 44.

4.0 INVESTIGATION PROCEDURE

The field work for this investigation was conducted in two phases, the first between December 4, 1989 and December 18, 1989 and the second between January 22 and January 25, 1990. During that time, a total of 9 boreholes was drilled at the locations. In addition, one dynamic cone penetration test was conducted from surface. Details of the drilling program are summarized in Table 1. The borehole logs and the results of dynamic cone penetration tests are presented in Appendix I.

The boreholes were advanced using 200 mm diameter hollow stem augers. The drill type used for this investigation was a Bombardier mounted CME 55, owned and operated by Johnson Drilling, Ottawa, Ontario. During advance of the boreholes, sampling of the subsurface materials was performed at regular intervals. Generally, in the upper 7.5 m of each borehole , sampling was performed at 0.75 m intervals and thereafter at 1.5 m intervals. Sampling was generally achieved using a split spoon sampler associated with the Standard Penetration Test. At selected locations in the cohesive units, undisturbed thin-walled Shelby tube samples were taken. All recovered samples were initially examined and logged in the field and thereafter transported to our Mississauga office for detailed visual and tactile examination. Sample types and locations are presented on the borehole logs (Appendix I).

In addition to sampling the subsurface materials, field insitu undrained vane tests were performed at regular intervals throughout the cohesive strata. Generally, the spacing of field vane determinations was 0.75 m in the upper 7.5 m and 1.5 m there-after. Also, as a general rule in the upper 6.0 m of the borehole, a split spoon sample was taken for material identification purposes after each field vane test was performed. Between 6.0 m and 15.0 m field vane tests and insitu sampling were spaced at approxi- mately 0.75 m. At depths in excess of 15.0 m, field vane tests and sampling were performed successively at 1.5 m intervals to depth. At Boreholes 4A and 8, drilled to penetrate the lower units, no sampling or insitu testing of the overburden cohesive strata was performed. Measured field insitu undrained vane strengths are presented on the borehole logs (Appendix I).

At Borehole 2, drilling and sampling of the underlying silty sand till layer was achieved by advancing B-size casing whilst washboring using a bentonite based drilling mud. This system was used to counteract the excess hydrostatic water pressures within the till layer which caused difficulty with material blowing up in the augers when this layer was penetrated using hollow stem augers.

The underlying bedrock was cored using a BX size core barrel at the locations of Borehole 2, 4A and 8 for total depths of 1.6 m, 6.2 m and 3.5 m, respectively.

In addition to monitoring the groundwater conditions during drilling, a total of four piezometers was installed in Boreholes 2, 3, 4A and 5. The installation details of these piezometers and the recorded water levels are presented on the borehole logs (Appendix I) and summarized in Table 1.

The field work was supervised at all times by a member of our engineering staff who supervised the drilling and sampling operations, ensured proper procedures for field vane tests were adhered to, logged the boreholes, cared for the samples obtained and supervised the installation of the piezometers.

The locations and ground surface elevations of the boreholes were confirmed in the field by the survey department, Ministry of Transportation of Ontario, Ottawa, Ontario prior to the commencement of drilling. It is understood that the elevations are related to geodetic datum.

5.0 LABORATORY TESTING

Laboratory testing on the recovered samples taken from within the overburden materials consisted of routine index testing on the disturbed split spoon samples and consolidation testing on selected Shelby tube samples.

Index testing consisted of moisture content determinations, grain size analysis, and Atterberg limits. The results of these tests are presented on the borehole logs (Appendix I). The results of 5 grain size analyses are presented on Figures 1 to 3 of Appendix II.

Four consolidation tests were carried out on Shelby tube samples taken from within the overburden materials. The locations of the tests are indicated on the borehole logs (Appendix I) with the test results presented on Figures 4 to 5 of Appendix II. An incremental load ratio of 2 was used for the consolidation test performed on Sample No. 4, Borehole 6. The remaining tests were conducted with an incremental load factor of 1.5.

One unconfined compression test was performed on a sample of the bedrock taken from Borehole 4A at a depth of 26.2 m (elevation 92.0 m). The test result is discussed in Section 6.0.

6.0 SUBSURFACE GROUND CONDITIONS

Based on the soil conditions encountered at the location of the boreholes, stratigraphic conditions at the proposed site generally consist of fill overlying a layer of sandy silt which in turn overlies silty clay. The silty clay was found to be underlain at depth by a silty sand till which in turn was underlain by limestone bedrock. A summary of the borehole information is presented on the stratigraphic section (Drawing No. 348102-A*). More detailed information on the stratigraphy at the borehole locations is presented on the borehole logs (Appendix I). A description of these individual units is presented below.

Fill

A layer of fill, associated with the construction of the previous alignment of Highway 44 and the present Highway 17, was encountered in all the boreholes. The fill generally consists of a brown, medium sand. Standard Penetration "N" values within the fill ranged from 9 to 48 blows and the layer can generally be described as having a compact relative density.

At the locations of Boreholes 1, 2, 3 and 7, drilled from the surface of the previous alignment of Highway 44, the thickness of this layer is of the order of 1.2 m. At the locations of Boreholes 4, 5 and 6, drilled through the higher Highway 17 a maximum thickness of 2.1 m was recorded.

Silty Fine Sand - Sandy Silt

A layer of silty fine sand to sandy silt with trace clay, was encountered in all of the boreholes. At the location of Borehole 4 the thickness of this layer was determined at 4.1 m with an associated lower interface elevation of 111.9 m. However, at the

* DWG NO 2 OF THE CONTRACT DWG'S

remaining boreholes the thickness varied from 1.6 m to 2.4 m with an associated range in elevation of the underside of this layer from 113.2 m to 114.0 m. The reason for the observed increase in thickness of this layer at the location of Borehole 4 is not known.

Measured Standard Penetration Test "N" values within the layer varied from 3 to 28 indicating a very loose to compact relative density.

Tactile investigation of disturbed samples taken from within this layer indicate that the sand content decreases with depth while the silt and clay contents increase. Occasional small white shells were present within this layer. The results of two grain size analyses from samples taken from within this layer are presented on Figure 1 of Appendix II.

Moisture content determinations from within this layer with the exception of a value of 35 percent at Borehole 1, Sample 3 show values which range from 17 to 23 percent. One determination of Atterberg limits from a sample taken from within this layer gave plastic and liquid limit values of 17 and 19 percent respectively with an associated plasticity index of 2. The layer can generally be described as being non-plastic.

The results of one consolidation test performed from an undisturbed Shelby sample taken from within this layer (Borehole 6; Sample 4) is presented on Figure 4 of Appendix II with key parameters summarized in Table 2. The coefficient of recompression is estimated at 0.005. The test was not continued sufficiently far beyond the preconsolidation pressure to permit an accurate estimate of the coefficient of compression. The pre-consolidation pressure of this sample is estimated at to be at least 480 kPa with an associated over-consolidation ratio of at least 11.7.

Silty Clay

Present in all boreholes, the measured thickness of this layer was found to reduce across the site towards the west. Measured thickness varied from 8.0 m at Borehole 1 to 17.1 m at Borehole 6. The associated elevations of the underside of this layer were 105.9 m at Borehole 1 and 96.5 m at Borehole 6.

Tactile inspection of split spoon samples taken from within the layer indicated that the layer generally consists of silty clay material interbedded with clayey silt layers (varves). The thickness of the clayey silt varves are of the order of 3 to 5 mm with spacings of the order of 25 to 30 mm. Within the bottom 3 to 4 m of the layer the thickness of the varves increases with an associated reduction in spacing.

The results of 2 grain size distribution tests on material from this layer are presented on Figure 2 of Appendix II. The gradations of these two samples include clay contents varying from 18 to 46 percent. This is believed to be indicative of the local gradation variances throughout this layer as a result of its varved nature.

- Insitu Undrained Vane Shear Strength

Individual measurements of the undrained shear strength with depth as determined in the field are presented on the borehole logs (Appendix I). The minimum observed value is of the order of 40 kPa and occurs at an elevation of approximately 113.0 m or 1.5 m to 2.0 m below the underside of the upper sandy silt layer. Below the minimum value, the measured values show a general increase with depth, estimated to be of the order of 4 kPa per metre. The layer can be described as having a firm to stiff consistency.

- Sensitivity

Observed sensitivity values from within the silty clay layer, defined as the ratio of the peak undrained shear strength and the vane remoulded strength as determined after 10 revolutions of the vane, are presented on the borehole logs (Appendix I). In general, the sensitivity shows no discernible trend with depth, with values ranging from a minimum of 3 to a maximum of 31. However, the majority of the values fall in the range between 5 and 20 and the material can be described as being highly sensitive and is typical of Champlain Sea deposits of eastern Canada.

- Moisture Content

Observed moisture content from within the upper sandy silt and the silty clay are presented on the borehole logs (Appendix I). Generally, the upper sandy silt material has a constant moisture content of the order of 20 percent. From the underside of the upper layer, at approximate elevation 114.0 m, values show an increasing trend up to maximum of the order of 50 percent at elevation 106.0 m. Below this elevation, values are generally constant until the bottom of the layer where moisture content values tend to decrease.

The results of 5 Atterberg limits from within the layer show a plastic limit range of between 15 and 34 percent with liquid limit values varying from 20 to 37 percent. The associated plasticity index varies from 5 to 13, and based on these values, the layer can be described as being of low to medium plasticity. The higher plasticity index values are presumed to be associated with the more clay rich samples. Also, it is expected that the silty clay varves have significantly higher plasticity values than those obtained from the "bulk" samples which were tested. Insitu moisture content values throughout the layer are generally higher than the measured liquid limit values.

- Consolidation Test Data

Four consolidation tests were performed on undisturbed Shelby samples taken from Borehole 6 and 7 (Appendix II - Figures 4 and 5). A summary of the test data is presented in Table 2.

The silty clay deposit is highly overconsolidated with Over Consolidation Ratios (OCR) of the order of 3.2 to 3.4.

However, the observed preconsolidation pressure profile is linear with depth indicating a consistent trend within this deposit. Mesri (1975) relates the undrained shear strength of clay deposits to preconsolidation pressure. Therefore, the observed linear increase of preconsolidation pressure with depth is in keeping with an observed similar trend for the field insitu undrained vane shear strength.

Compression Index values for the tests vary from 0.305 to 1.06 with re-compression index values ranging from 0.013 to 0.040. As presented in Table 2, the compression and recompression indices of the deepest test are significantly higher than the other two tests performed within this layer. This indicates that the material at depth within this layer may have a more brittle structure than those closer to the surface. This conclusion is supported by the higher observed natural moisture content in this region of the layer.

Coefficient of Consolidation values (C_v), are significantly higher in the pressure range up to the preconsolidation pressure than thereafter. Typical C_v values up to the preconsolidation pressure are of the order of 25 to 50 m^2/yr. Within the virgin consolidation range, typical C_v values are of the order of 3 to 15 m^2/yr.

Till

As encountered at the boreholes, the underlying till layer becomes deeper from west to east across the site. The surface of this layer was at elevation 105.9 at Borehole 1 and at elevation 95.8 m at Borehole 8.

The thickness of this layer in Boreholes 2, 4A and 8 was 4.2 m, 4.6 m and 2.0 m, respectively. At Borehole 6, auger refusal within this layer suggests a minimum thickness of 3.1 m. Further, dynamic penetration tests performed at the bottom of Boreholes 1 and 3 infer minimum thicknesses for this layer of 4.5 m and 7.2 m, respectively, for these locations. Hence, it can be concluded that the thickness of this layer is quite variable although it does appear to increase in thickness towards the west.

Visual and tactile examination of recovered samples indicate the till consists primarily of a mixture of silty sand with some gravel. Boulders, up to at least 0.5 m diameter (Borehole 4) were confirmed to exist within the deposit. A grain size analysis, performed on a sample taken from within this layer is presented on Figure 3 of Appendix II.

Measured "N" values within the deposit are quite variable and may be unreliable due to the "blowing up" of sand within the hollow stem augers while drilling within the layer. Also, as previously discussed, the deposit contains boulders which may also influence the measured "N" values. However, based on the measured "N" values as well as dynamic cone penetration tests conducted at Boreholes 1 and 3 and adjacent to Borehole 6, the upper 3.0 m of this layer appears to be generally in a loose state of relative density. Beyond 3.0 m depth into the layer, the measured "N" values tend to show a general increase.

Bedrock

The surface of the bedrock, as confirmed at Boreholes 2, 4A and 8, tends to rise from east to west across the site. Specifically at Boreholes 2, 4A and 8 the surface of the bedrock was determined to be at elevations 97.3 m, 94.4 m and 93.8 m, respectively. At these boreholes, the total lengths of cored bedrock was 1.5 m, 6.1 m and 3.3 m, respectively.

The bedrock consists of unweathered fresh limestone with thin (<1.0 mm) very closely spaced shale partings. Occasionally thicker horizons of shale up to 30 mm thickness were found in the deposit. The thin shale partings within the rock are generally tight but have a low tensile strength and hence splitting on these features occurs quite readily, especially during drilling. This is believed to be a major reason for the observed low RQD values reported on the borehole logs (Appendix I). Further, the shale partings are primarily non linear features and in some instances within the dimensions of the the core, the observed surfaces were quite irregular.

The shale on the thin partings is estimated to be of very soft rock strength. However, where the thickness of these features increases beyond 1 to 2 mm their strength was indistinguishable from the host limestone rock.

One Unconfined Compression Test strength determination, on a sample taken from Borehole 4A, gave a value of 48 MPa. The tested sample was representative of the general observed rock formation with very closely spaced thin shale partings as described above, present along its entire length.

Groundwater

Four standpipe piezometers were installed at the proposed site to monitor the ambient groundwater regime. One of the piezometers (Borehole 5) was sealed within the upper silty sand while another (Borehole 2) was sealed into the silty clay layer. The remaining piezometers were sealed into the underlying silty sand till.

Groundwater water levels within the upper sandy silt and silty clay appear to be hydrostatic with a groundwater elevation of approximately 116.0 m or approximately at the level of the existing ground surface. The piezometric head within the underlying till is below this level as measured at 113.8 m in Borehole 3. Unfortunately, it was not possible to obtain a reading on the piezometer installed in Borehole 4A because of inclement weather after its installation. However, based on the available information to date, it appears that a situation of vertical drainage to the underlying silty sand till layer exists. This may explain in part, the observed linear increase in depth of moisture

CLOSURE

The fieldwork for this report was performed under the supervision of Mr. I. Corbett, P.Eng. who also wrote this report. This report was reviewed by Mr. R.D. Powell, P.Eng. and Mr. M.A.J. Matich, P.Eng.

NOTE: The preceding report is a copy of the factual information from the Foundation Investigation Report prepared by Geocon Inc. (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.

TABLE 1 - BOREHOLE SUMMARY

Borehole No.	Location				Stratigraphic Lower Elevations (Layer Thickness in Brackets)					Groundwater		
	General	Chainage (km+m)	Offset (m)	Ground Elevation (m)	Fill (m)	Silty Sand (m)	Silty Clay (m)	Till (m)	Bedrock Surface Elevation	Piezometer Installed	Piezometer Tip Elevation (m)	Groundwater Elev. (m)
1	West Approach Fill	10+090	0.0	116.84	115.47 (1.37)	113.91 (1.56)	105.87 (8.04)	104.65* (1.22)	Not Confirmed	No		
2	West Abutment	10+046	8.7 Right	116.62	115.40 (1.22)	113.72 (1.68)	101.53 (12.19)	97.31 (4.22)	97.31 [1.52]	Yes	107.48	115.78
3	West Abutment	10+037	5.2 Left	116.81	115.59 (1.22)	113.76 (1.83)	103.40 (10.36)	102.64* (0.76)	Not Confirmed	Yes	103.40	113.81
4	Central Pier	10+000	0.0	118.17	116.04 (2.13)	111.92 (4.12)	98.97 (12.95)	96.83* (2.14)	Not Confirmed	No		
4A	Central Pier	10+000	1.0 Right	118.17	116.04 (2.13)	111.92 (4.12)	98.97 (12.95)	94.40 (4.57)	94.40 [6.14]	Yes	95.32	Not Measured
5	East Abutment	9.963	5.0 Right	117.46	115.33 (2.13)	113.19 (2.14)	96.74 (16.45)	95.67 (1.07)	Not Confirmed	yes	113.65	116.06
6	East Abutment	9+954	9.0 Left	117.23	115.71 (1.52)	113.57 (2.14)	96.51 (17.06)	93.46* (3.05)	Not Confirmed	No		
7	East Approach Fill	9+910	0.0	116.94	115.72 (1.22)	113.28 (2.44)	103.99* (9.29)	-	Not Confirmed	No		
8	East Abutment	9+957	5.0 Right	117.16	-	-	95.83 ?	93.79 (2.04)	93.79 [3.30]	No		

Notes

1) Asterisk indicates that layer was not fully penetrated
Elevation given is lowest elevation confirmed by drilling

2) Elevations given are assumed to be Geodetic

3) Square brackets indicate depth of bedrock drilling

4) Consult borehole logs for more detailed information

TABLE 2

Consolidation Test Data

Sample Location					Test Data					
Test #	Borehole No.	Sample No.	Depth (m)	Elevation (m)	σ'_v (kPa)	P_c (kPa)	O.C.R.	C_c	C_r	e_o
1	6	4	3.27	113.96	41	>480	>11.7	-	0.005	0.455
2	7	6	4.87	112.07	56	182	3.25	0.360	0.015	0.934
3	6	8	7.92	109.31	77	259	3.36	0.305	0.013	0.975
4	6	11	12.50	104.73	112	383	3.42	1.060	0.040	1.442

Definitions

σ'_v - Existing Vertical Effective Stress
P_c - P_c Estimated Preconsolidation Pressure
O.C.R. - Over Consolidation Ratio
C_c - Compression Index
C_r - Reload Compression Index
e_o - Initial Void Ratio

APPENDIX I

Borehole Logs

RECORD OF BOREHOLE No 1

METRIC

W P 34-81-02 LOCATION CH 10 + 090 (Hwy. 44) Centre Line ORIGINATED BY R.K.

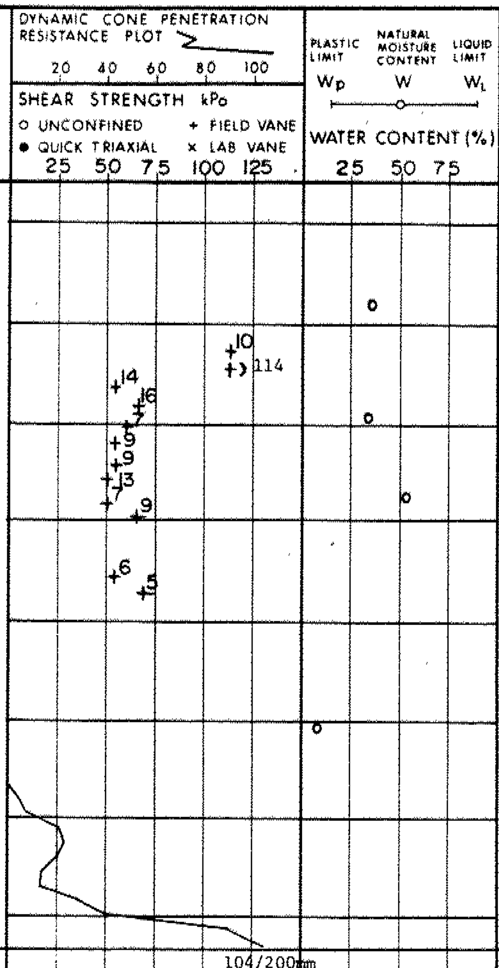
DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger & Penetration Test COMPILED BY I.C.

DATUM Geodetic DATE December 14, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES
116.84	Ground Level			
0.00 / 115.47	Compact, brown sand. Fill	1	SS	18
1.37 / 113.91	Compact to loose, grey silty sand to sandy silt. Tr. Clay.	2	SS	10
		3	SS	3
2.93 / 105.87	Stiff, grey silty Clay with 3 mm thick clayey silt varves. Thickness and frequency of varves increasing with depth.	4	SS	PM*
		5	SS	PM*
		6	SS	PM*
		7	SS	PM*
		8	SS	PM
10.97 / 104.65	Grey, silty sand. Some gravel. Occ boulder. Till	9	SS	4
12.19 / 101.40	End of Borehole			
15.44	End of penetration test			

Note
After completion of Penetration Test the rods were pulled back 0.9 m. A total of 16 blows were required to re-advance the rods.

PM* - Sample was taken from disturbed ground.



UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C

$+^3, \times^5$: Numbers refer to Sensitivity

20 / 15 -◇- 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 2

METRIC

W P 34-81-02 LOCATION CH 10 + 045.9 - 8,7 RT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 15.09 m. COMPILED BY I.C.

DATUM Geodetic DATE December 11, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE
116.62	Ground Level				
0.00	Loose, brown sand. Some silt. Fill	1	SS	9	116
115.40 / 1.22	Compact to loose silty fine sand. Tr clay. Silt and clay. Content increase with depth.	2	SS	12	
		3	SS	4	114
113.72 / 2.90	Stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	4	SS	PM*	
		5	ST	-	112
		6	SS	PM*	
		7	ST	-	110
		8	SS	PM	108
		9	ST	-	
		10	SS	PM	106
		11	SS	PM	104
101.53 / 15.09	Loose to compact, grey silty sand. Tr clay, some gravel. Occ boulder. Till	12	SS	3	102
		13	SS	26	100
		14	SS	18	
97.31 / 19.31	Fresh, grey, medium grained limestone bedrock with dark grey, closely spaced, dark grey partings (below 10mm) of shale 50mm fractured zone at 20.2 m.	15	BQ		
95.79 / 20.83		16	BQ		

Ground water conditions: standpipe water level near 115.8; Seal at about 108.5; piezometer tip below seal.

Shear strength (kPa), field vane (+): 114 (at approx. elev. 113); 9, 9, 9, 12 (approx. 112–111); 9, 16 (approx. 109.5); 24, 31 (approx. 108); 6, 7 (approx. 106.5); 20, 13 (approx. 105); 9, 10 (approx. 102.5)

Rec %	RQD %	Water Return
90	65	100
95	80	100

Unit weight: 98, 96

Remarks & grain size distribution (%) GR SA SI C: 0, 40, 53, 7 (sample 2); 12, 44, 20, [illegible] (sample 13)

End of Borehole

Notes

Water level in standpipe Piezometer at elevation 115.78 m on 22/12/89.

PM* - Sample taken from disturbed ground.

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$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⟡ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 34-81-02 LOCATION CH 10 + 037.2 - 5.2 LT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger & Penetration Test COMPILED BY I.C.

DATUM Geodetic DATE December 13-14, 1989 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
116.81	Ground Level					
0.00	Compact, brown sand Fill	1	SS	13		116
115.59 / 1.22	Compact to loose sandy Silt. Tr Clay. Silt and clay contents increase with depth.	2	SS	11		
		3	SS	5		114
113.76 / 3.05	Stiff grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	4	SS	1/50cm		
		5	ST	-		
		6	SS	PM		112
		7	SS	PM		110
		8	SS	PM		108
		9	SS	PM		106
103.40 / 13.41	Loose, grey silty sand Some gravel.Till.	10	SS	5	Seal	104
102.64 / 14.17	End of Borehole					102
						100
						98
96.16 / 20.65	End of penetration test					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125

Field vane values (sensitivity): 11, 10, 11, 9, 13, 22, 26, 16, 6, 11

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C

Redrive values after pulling back 0.9 m

Notes

Water level in stand-pipe at elevation 113.81 m on 24/01/90

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$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 -ó- 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

W P 34-81-02 LOCATION CH 10 + 000 (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 19.61 m COMPILED BY I.C.

DATUM Geodetic DATE December 8-11, 1989 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES
118.17	Ground Level			
0.00	Dense to compact, brown sand. Tr gravel Fill	1	SS	48
		2	SS	24
116.04 / 2.13	Compact to loose, grey sandy silt. Tr Clay. Silt and clay contents increase with depth.	3	SS	24
		4	SS	3
		5	SS	28
		6	SS	28
		7	SS	6
111.92 / 6.25	Very stiff becoming stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30mm spacings.	8	ST	-
		9	SS	PM
		10	SS	PM
		11	SS	PM
		12	ST	-
		13	SS	PM
		14	SS	PM
98.97 / 19.20	Grey silty sand. Some gravel. Occ. boulder. Till Boulder 19.61 m-20.11 m	15	BQ	
96.83 / 21.34	End of Borehole Auger Refusal at 19.61m Started coring (BQ size) at this depth			

GROUND WATER CONDITIONS | ELEVATION SCALE

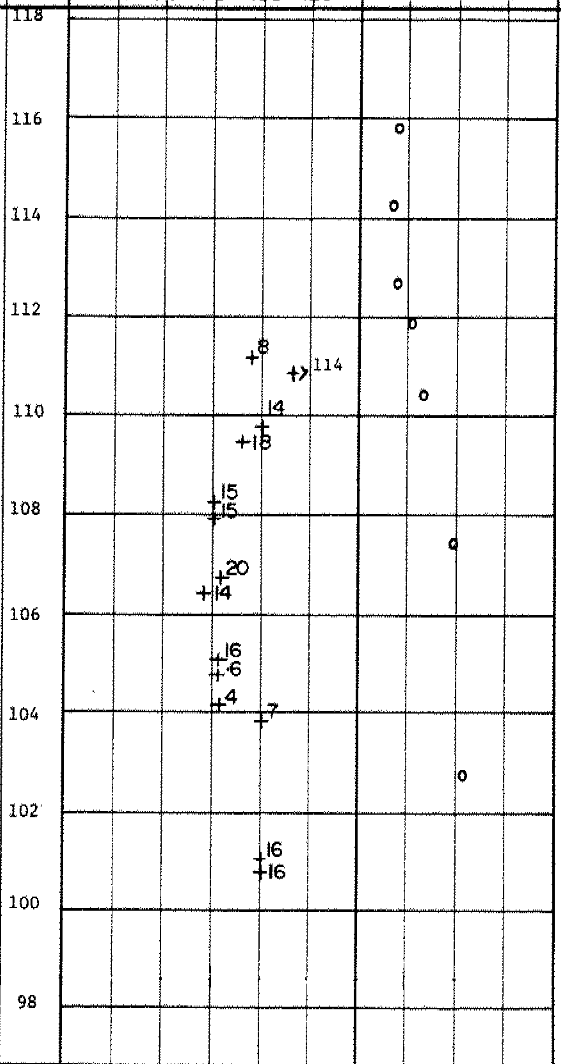
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C



+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 4A

METRIC

W P 34-81-02 LOCATION CH 10 + 000 - 1.0 Rt (Hwy 44) ORIGINATED BY MK

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) 23.77 m COMPILED BY IC

DATUM Geodetic DATE January 24 & 25, 1990 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	Rec %	RQD %	Water Return %	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
118.17	Ground Level										
0.00	Overburden material not sampled (For stratigraphy see Borehole No. 4)					118					
98.97 / 19.20	Probably, grey silty sand, some gravel. Occ. boulder. Till				Seal						
94.40 / 23.77	Fresh, sound, grey limestone bedrock with very closely spaced thin, (<1 mm) tight shale partings. Closely spaced shale bands (10-20 mm). Shale partings generally have irregular surface.	1	BQ		R1		95	61	100	94	
		2	BQ		R2		95	39	100	92	UCS @ 91.88-91.9 = 48 MPa
	Core breaks readily on shale partings	3	BQ		R3	90	92	34	100	90	
		4	BQ				0	0	100		
88.25 / 29.92		5	BQ		R4		87	24	100		

End of Borehole

Note: Water level in piezometer not measured.

Elevation scale: 118, 116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96, 90

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%)

$+^3, \times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 5

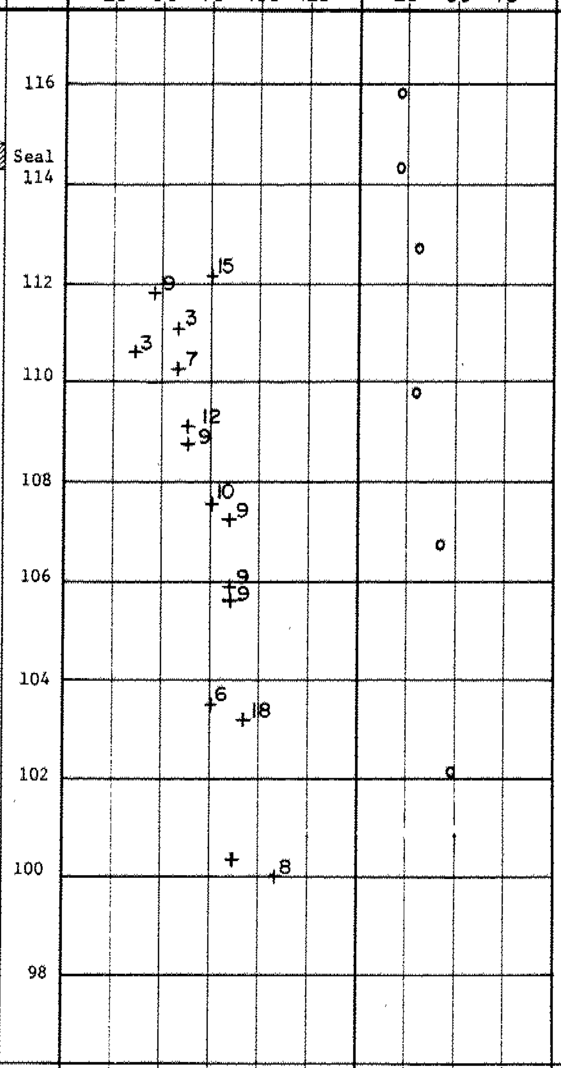
METRIC

W P 34-81-02 LOCATION CH 9 + 962.8 - 5.0 RT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger COMPILED BY I.C.

DATUM Geodetic DATE December 4, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS
117.46	Ground Level				
0.00	Compact to loose brown sand. Occ organics. Fill.	1	SS	13	
		2	SS	9	
115.33 / 2.13	Compact, grey silt. Tr. sand and clay. Occ. shells	3	SS	17	
		4	SS	16	Seal
		5	SS	10	
113.19 / 4.27	Stiff to firm, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	6	SS	2	
		7	SS	PM*	
		8	SS	PM	
		9	SS	PM	
		10	SS	PM	
		11	SS	PM	
		12	SS	PM	
		13	SS	PM	
96.74 / 20.72	Loose, grey silty sand Some gravel. Till	14	SS	7	
95.67 / 21.79	End of Borehole				



DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 25 50 75 100 125; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C

Note

Piezometer installed a short distance away from Borehole 5.

Water level in standpipe at elevation 116.06 m on 22/12/89.

PM* - Sample taken from disturbed ground.

+3, x5 : Numbers refer to Sensitivity

15 -ф- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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RECORD OF BOREHOLE No 6

METRIC

W P 34-81-02 | LOCATION CH 9 + 954.1 - 9.0 LT (Hwy. 44) | ORIGINATED BY R.K.

DIST 9 HWY 44 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY I.C.

DATUM Geodetic | DATE December 5, 6, 1989 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.23	Ground Level						
0.00 / 115.71	Loose, brown sand. Tr Silt Fill	1	SS	10			
1.52 / 113.57	Compact to loose, grey sandy silt. Tr. clay. Occ. shells	2	SS	18			
		3	SS	7			
		4	ST	–		21.1	0, 54, 45,
3.66 / 96.51	Stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30mm spacings.	5	SS	1/50 cm			
		6	SS				
		7	SS	PM			
		8	ST	–		18.4	
		9	SS	PM			
		10	SS	PM			
		11	ST	–		16.8	0, 2, 52, 4
		12	SS	PM			
		13	ST	–			
		14	SS	PM			
20.72 / 93.46	Compact, grey silty sand and gravel. Tr clay. Occ boulder. Till.	15	SS	12			
23.77	End of Borehole Auger refusal						

ELEVATION SCALE: 116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96, 94

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

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

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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OFFICE REPORT ON SOIL EXPLORATION

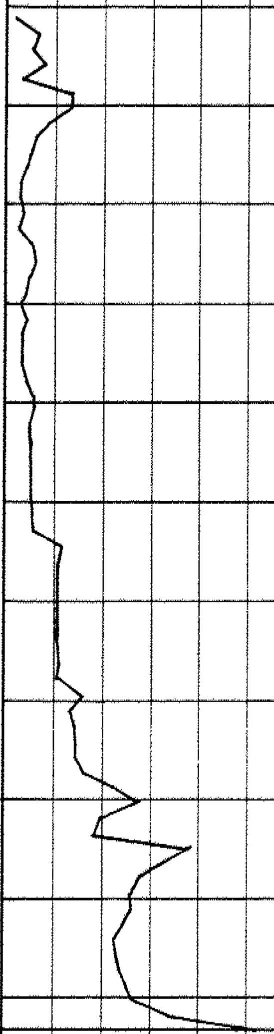
RECORD OF BOREHOLE No 6A

METRIC

W P 34-81-02 LOCATION CH 9 + 954.1 - 2.0 LT (Hwy. 44) ORIGINATED BY RK

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Penetration Test @ 1.5 m COMPILED BY IC

DATUM Geodetic DATE December 18 , 1989 CHECKED BY

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
117.23	Ground Level										
0.00	Overburden materials not sampled. For stratigraphy see Borehole 6						116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96				
95.03 / 22.20	End of Penetration Test							104/50 mm			

0 10 20 30
1:500

$+^3$, $\times^5$: Numbers refer to Sensitivity

20
15 -○- 5 (%) STRAIN AT FAILURE
10

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RECORD OF BOREHOLE No 7

METRIC

W P 34-81-02 LOCATION CH 9 + 910 (Hwy.44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger COMPILED BY I.C.

DATUM Geodetic DATE December 7, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C	UNIT WEIGHT γ kN/m³
116.94	Ground Level						
0.00	Loose, brown sand Tr silt. Fill	1	SS	9	116		
115.72 / 1.22	Compact, grey sandy silt Tr clay. Occ shells. Silt and clay contents increase with depth.	2	SS	21			
		3	SS	12	114		
		4	SS	12			
113.28 / 3.66	Stiff to firm, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	5	SS	PM*			
		6	ST	-	112	0, 10, 72,	18.5
		7	SS	PM*	110		
		8	ST	-	108		
		9	SS	PM			
		10	SS	PM	106		
103.99 / 12.95	End of Borehole						

PM* - Sample taken from disturbed ground.

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125. Field vane values with sensitivity: +9, +4, +5, +3, +×114, +8, +9, +7, +8, +9.

Plastic limit W_p, Natural moisture content W, Liquid limit W_L. Water content (%) 25 50 75.

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

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

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OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 8

METRIC

W P 34-81-02 LOCATION CH 9 + 957 - 5.0 Rt (Hwy 44) ORIGINATED BY MK

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 23.37 m COMPILED BY IC

DATUM Geodetic DATE January 22, 23 and 24, 1990 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	Recovery %	RQD %	Water Return %	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI
117.16	Ground Level												
0.00	Overburden not sampled to 21.33 m. For stratigraphy see Borehole 5.						116 … 96						
95.83 / 21.33	Compact to dense, grey silty sand. Some gravel, occ. boulders. Till		1	SS	38								
			2	SS	29		94						
93.79 / 23.37	Fresh, sound dark grey limestone bedrock with very closely spaced thin (<1 mm) shale partings. Core splits readily on shale partings. Parted surfaces generally very irregular		3	BQ				99	54	100			
			4	BQ			92	100	59	100			
90.49 / 26.67	End of Borehole												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

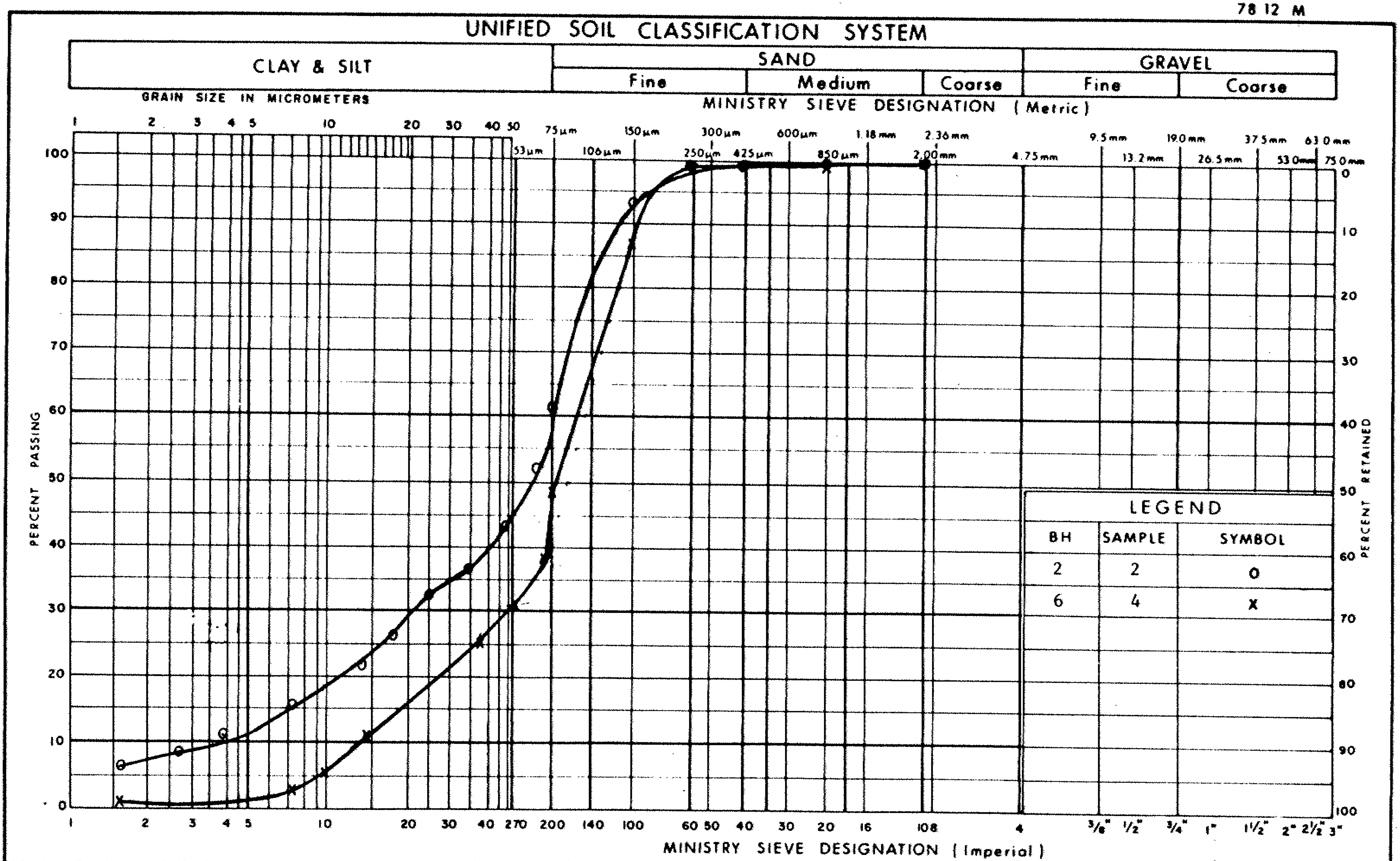
$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

APPENDIX II

Laboratory Test Data

78 12 M



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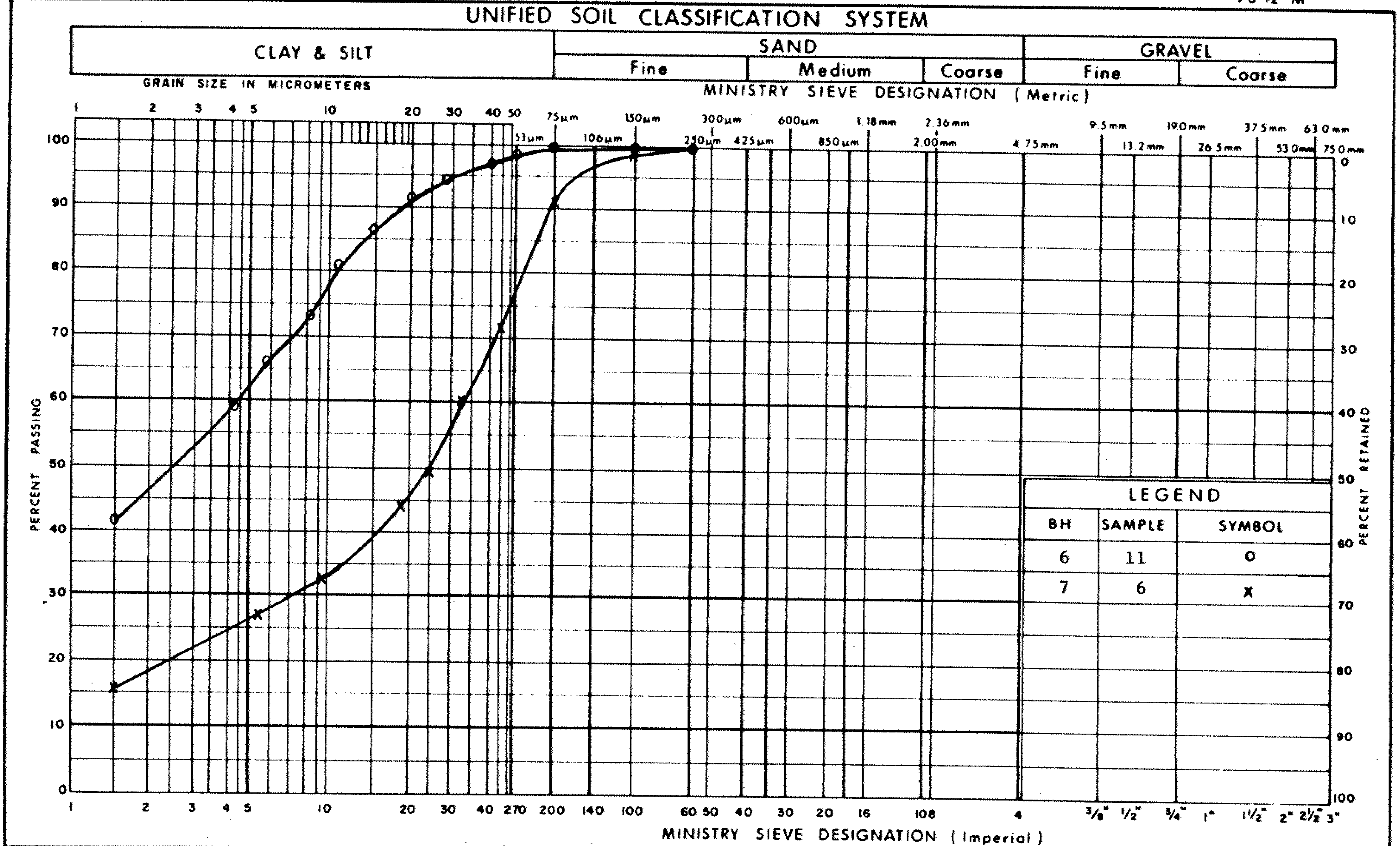
GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Silt, Trace Clay

FIG No 1

W P 34-81-02

78 12 M



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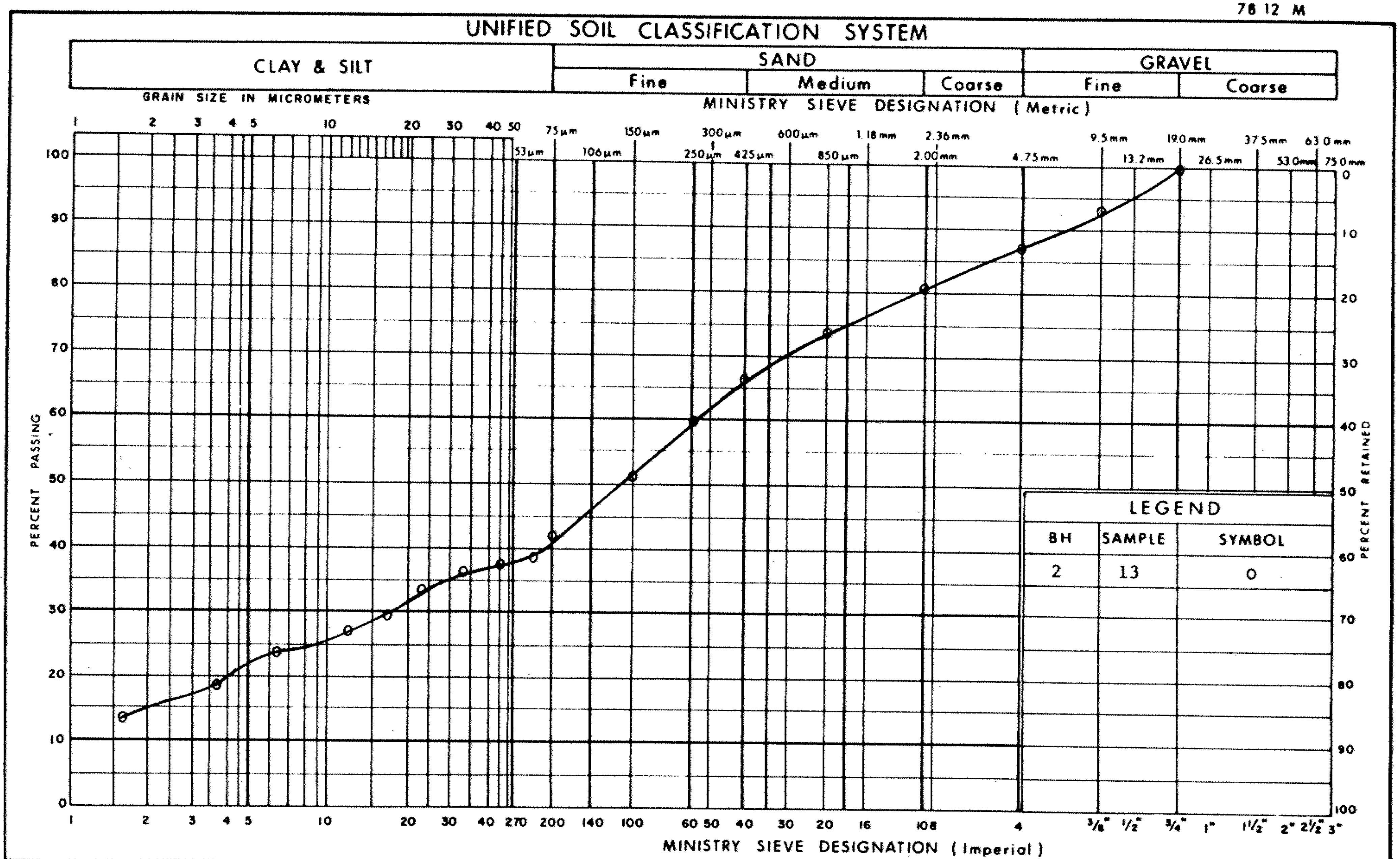
GRAIN SIZE DISTRIBUTION

Silty Clay

FIG No 2

W P 34-81-02



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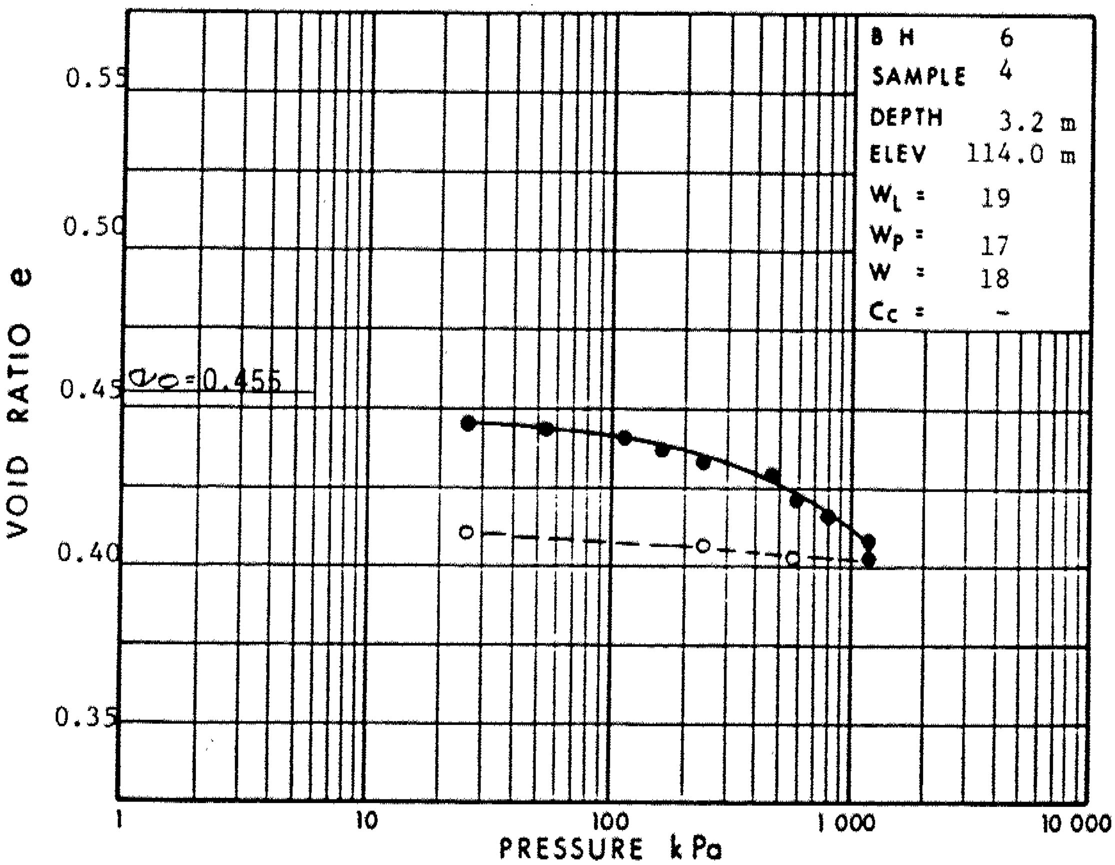
GRAIN SIZE DISTRIBUTION

Silty Sand, Some Gravel and Clay - TILL

FIG No 3

W P 34-81-02

VOID RATIO - PRESSURE CURVES



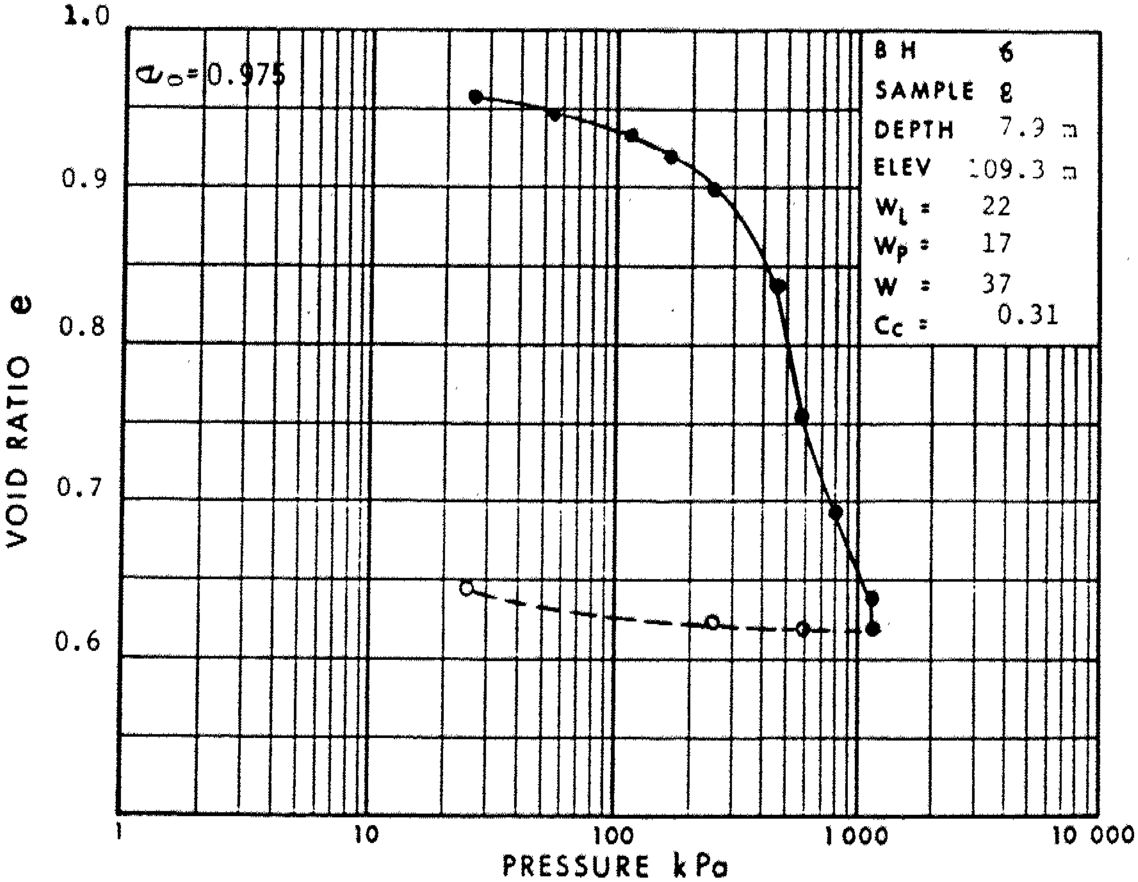
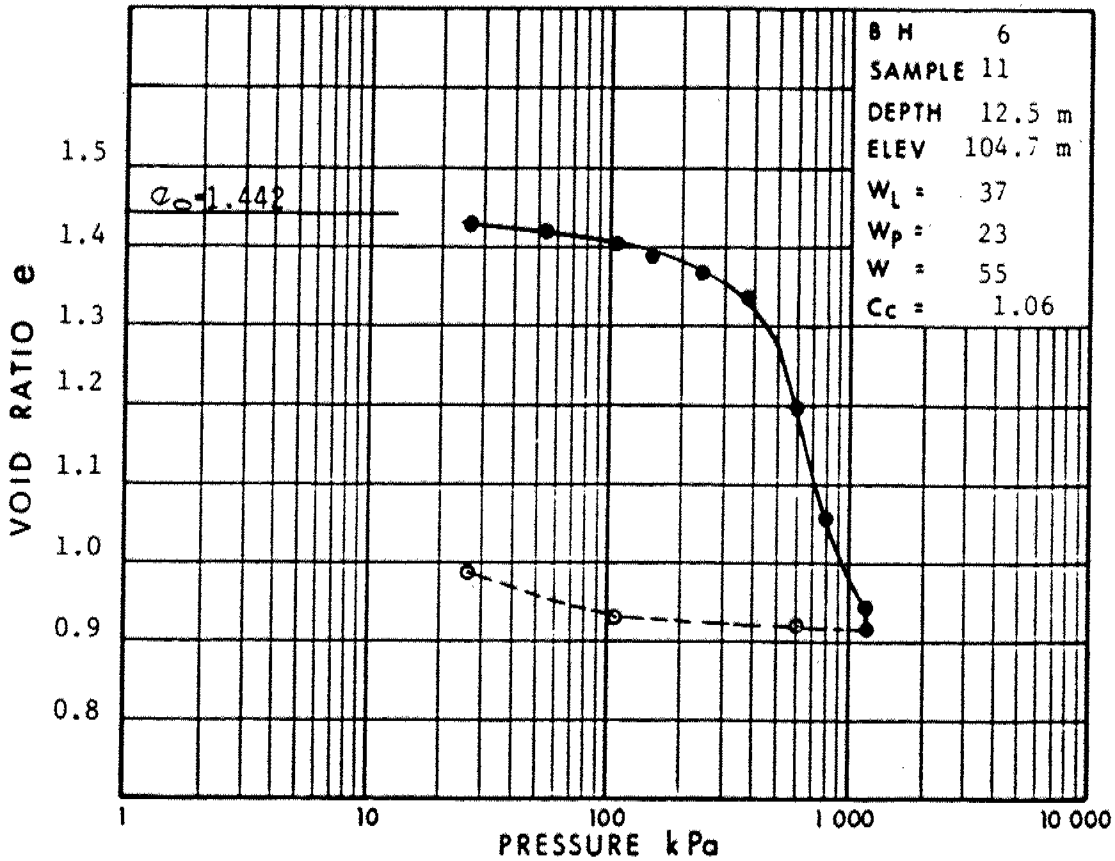


Fig No 4

VOID RATIO - PRESSURE CURVES



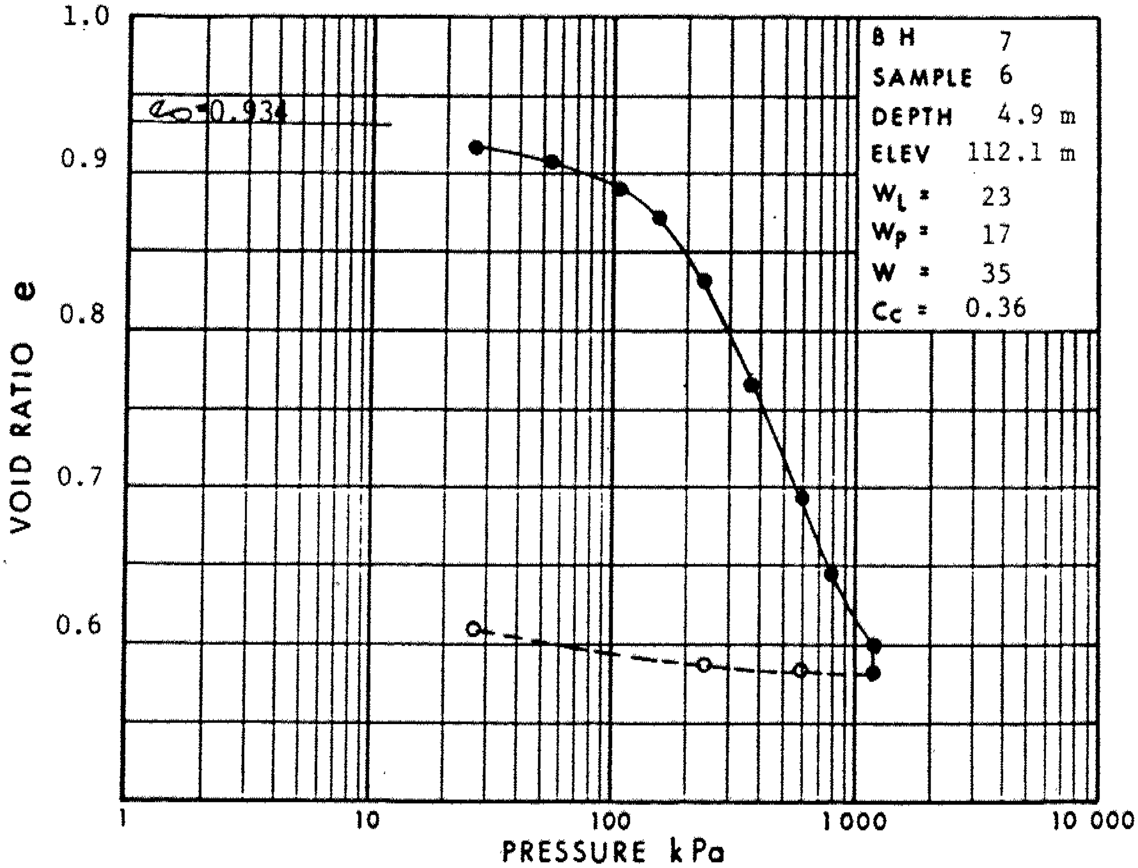


Fig No 5

FOUNDATION INVESTIGATION

PROPOSED
UNDERPASS STRUCTURE
RICHARDSON ROAD OVER HIGHWAY 417
OTTAWA, ONTARIO

W.P. 34-81-04 SITE NO. 3-570

DISTRICT 9, EASTERN REGION

1. INTRODUCTION

A field investigation for this project was conducted between December 12 and 20, 1989. Eleven boreholes were drilled to depths between 3.5 and 12.0 m below existing ground surface at the locations shown on Drawing No. 348104-A.* Five of these boreholes were advanced at the proposed foundation locations, while the remaining six boreholes were located along the approach embankments at the west and east ends of the structure. Details of the field work program are provided in Appendix "A" to this report.

The borehole locations were mutually agreed upon and were located in the field by staff of MTO who also provided the ground surface elevations. It is understood that the borehole elevations are referenced to geodetic datum.

2. SITE AND GEOLOGY

The site is located west of Ottawa (near Huntley), at the intersection of existing Highway 17 and Richardson Road (formerly Cowan Road).

* DWG NO 2 OF THE CONTRACT DWG'S

The property around the site is currently forested land. The ground surface in the area is very flat. Both roads are asphalt paved, with two lanes, gravel shoulders, side ditches, and right turning lanes along Highway 17.

A review of selected geologic references suggests that this site is located in an area characterized by shallow surficial deposits of glaciolacustrine sand and gravel. The sand and gravel is then underlain by glacial till and ultimately by limestone bedrock of the Trenton and Black River formations.

Ministry of the Environment Well Records for the area confirm the presence of sand and gravel overburden deposits, with limestone bedrock encountered at depths of about 6 to 8 m.

3. SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered at the site are summarized below, and are also presented on the accompanying Borehole Logs and Sections on Dwg. No. 348104-A*

Details of the laboratory tests and field tests are summarized on the Borehole Logs and on Figures 1 to 3 inclusive.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary at other locations. The boundaries between the various strata as shown on the logs and sections are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

The ground surface along the proposed bridge alignment is flat, and at about elevation 131 m Geodetic.

* DWG NO 2 OF THE CONTRACT DWG'S

In summary, the boreholes found 0.6 to 1.2 m of granular fill at the ground surface. The fill was underlain by the native compact to dense sand and gravel which extended to depths of up to 3.4 m.

The sand and gravel was underlain by a dense to very dense sandy silt till which extended to limestone bedrock, encountered at a depth of approximately 10 m.

3.1 Fill

The borings generally encountered a thin layer of fill at the ground surface, to depths of about 0.6 to 1.2 m. This fill material appears to be associated with the construction of the existing Richardson Road and Highway 17, and comprised relatively clean sand, to sand and gravel. The fill materials were generally frozen to depths of approximately 0.3 m. The results of limited penetration testing in the material suggest that it is generally in a compact to dense condition. The moisture contents of the fill ranged from 4 to 19 percent, with an average of 5 percent.

3.2 Sand and Gravel

Beneath the fill, native sand to sand and gravel was encountered to depths of 1.5 to 3.4 m, in all of the borings except Borehole 11. The sand and gravel material was absent in Borehole 11. This strata ranged in composition from sand with a trace to some gravel and silt, to sand and gravel with some silt and a trace of clay. Grain size distribution curves for the sand and gravel stratum are presented on Figure 1. It was generally in a compact to dense condition with "N" values ranging from about 8 to 62 blows per 300 mm. Standard Penetration Resistance Values were generally greater than 20. The higher blow counts noted in the sand and gravel materials are likely the result of coarse gravel materials encountered by the sampling spoon.

The moisture content of the sand and gravel ranged from 4 to 23 percent with an average of 11 percent. It should be noted that portions of the deposit were below the water table, and therefore the measured moisture contents may not be representative.

For design purposes, the following soil properties are estimated. These soil properties are not factored.

- effective angle of internal friction, (Phi)	- 32 degrees
- effective cohesion intercept, c'	- 0 kPa
- unit weight, (Gamma)	- 21 kN/cu.m

3.3 Silty Sand, Trace Clay and Gravel (Glacial Till)

Beneath the sand and gravel, a strata of glacial till was encountered to either the base of the borehole, or the underlying bedrock surface at depths of 10 to 10.2 m. The glacial till generally comprised silty sand with a trace to some clay and gravel. Grinding of the augers was noted at various depths throughout the till deposit, suggesting the presence of occasional cobbles and boulders. Grain size distribution curves for the glacial till are shown on the accompanying Figure 2. The till was generally in a dense to very dense state with Penetration Resistance Values ranging from about 32 to over 100 blows per 150 mm. Some of the higher penetration resistance values may have been the result of coarse gravel or cobble materials encountered by the sampling spoon. There was a tendency for a decrease of "N" values with depth, likely as the result of upward seepage and loosening of the material in the boring.

The moisture contents varied from 4 to 10 percent with an average of 7 percent.

Atterburg limits were determined on four select samples and the results presented on the Borehole Logs and as summarized below:

Borehole No.	Sample No.	Plastic Limit	Liquid Limit	Plasticity Index
1	3	12	20	8
4	7	11	13	2
7	10	10	16	6
10	7	12	17	5

These reflect an inorganic silt of low plasticity (unified soil classification CL, or CL-ML). The till is slightly cohesive in nature.

For design purposes, the following soil properties are estimated based on index properties of the deposit. These properties are not factored.

- effective angle of internal friction, (Phi) - 35 degrees
- effective cohesion intercept, c' - 0 kPa
- unit weight, (Gamma) - 22 kN/cu.m.

3.4 Bedrock

Boreholes 4 to 8 were carried to refusal on the underlying bedrock at depths of 10 to 10.2 m (corresponding to elevations of 120.9 to 121.4 m). Based on the borehole data, the bedrock surface appears to be relatively flat-lying. Bedrock cores were obtained from Boreholes 4, 6 and 7 for lengths of approximately 1.5 m. The total core recovery obtained was close to 100 percent, with RQD values of 65 to 90 percent. The bedrock was slightly weathered to fresh.

3.5 Groundwater

Seepage was encountered in all of the borings, near the base of the sand and gravel strata. The water levels were measured in the borings on December 22, 1989, about one week following their completion. The water levels were relatively uniform at 1.3 to 1.6 m below ground surface. Heavy seepage was noted in the borings through the sand and gravel strata and also through isolated areas within the glacial till, particularly at depth.

The long term static levels should be anticipated to fluctuate, with higher levels expected during wet seasons.

Water levels were measured in the piezometers installed in the boreholes on December 22, 1989. The water levels are summarized on the Borehole Logs and are as follows:

Borehole	Water Depth/Elev.
1	1.3/129.9m
2	1.5/129.6m
3	1.4/129.6m
4	1.6/129.4m
5	1.6/129.6m
6	1.4/129.9m
7	1.5/129.8m
8	1.5/130.0m
9	1.5/129.5m
10	1.6/129.1m
11	1.5/129.1m

APPENDIX A

Summary of Field Investigation Procedures

W.P. 34-81-04

Richardson Road

APPENDIX - A

FIELD PROCEDURE

The field investigation for this project was conducted between December 12 and 20, 1989, when 11 boreholes were advanced to depths of 3.5 to 12.1 m below existing grades, at the locations shown on Drawing NO. 348104-A *. The drilling was conducted using machinery supplied and operated by Longyear Canada Inc. mobilized from Toronto, Ontario. The drilling operations were directed and supervised by Mr. Renato Pasqualoni, B.A.Sc., P.Eng. of Terraprobe Limited.

Five boreholes (BH 4 to 8) were put down in the vicinity of the proposed piers and abutments for the proposed bridge. In addition, 6 boreholes (BH 1 to 3, 9 to 11) were advanced along the alignment of the proposed approach embankments.

The borings were put down using a crawler-mounted CME 55 power auger machine. Split-spoon samples of the overburden materials were obtained as detailed on the Borehole Logs and Sections. Dynamic Cone Tests were also carried out at the Boreholes locations. (All samples obtained in this investigation were sealed in jars and transported to our laboratory for detailed inspection and testing.)

Standpipe type piezometers were sealed into all the boreholes in order to permit observation of groundwater levels. The standpipes comprised 12 mm I.D. CPVC tubing, which was saw-slotted near the base, and fitted with a sand filter and bentonite seal, as noted on the Borehole Logs.

The locations of the borings were determined by measuring relative to the survey stakes placed and marked by Ministry of Transportation representatives. The ground surface elevations at the borehole locations were determined by our field engineer with reference to survey points determined by MTO representatives.

* DWG NO 2 OF THE CONTRACT DWG'S

The water levels in the standpipes were measured prior to demobilization from the area on December 22, 1989.

All of the soil samples obtained in this investigation were examined in detail by the project engineer, and classified according to visual and index properties.

Water content determination was carried out for all samples obtained. In addition, laboratory tests were carried out on selected samples, including grain-size distribution, and unit weights where applicable. The results of the testing are presented on the Borehole Logs and on Figures 1 to 3.

NOTE: The preceding report is a copy of the factual information from the Foundation Investigation Report prepared by Terraprobe Ltd. (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.

APPENDIX

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 34-81-04 LOCATION Sta 10 + 105 o/s 5.0m RT ℄/ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 14, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.2	Ground Surface						
0.0	Fill, sand and gravel trace silt. Compact Grey	1	CS	-	Seal	131	
130.6 / 0.6	Sand and Gravel, trace silt and clay. Compact Brown	2	SS	23	Water Level Dec.22/89	130	
129.7 / 1.5	Silty Sand, trace gravel, some clay. Dense to very Dense Grey (Glacial Till)	3	SS	39		129	2 53 35 10
		4	SS	55		128	
		5	SS	88			
127.7 / 3.5	End of Borehole						

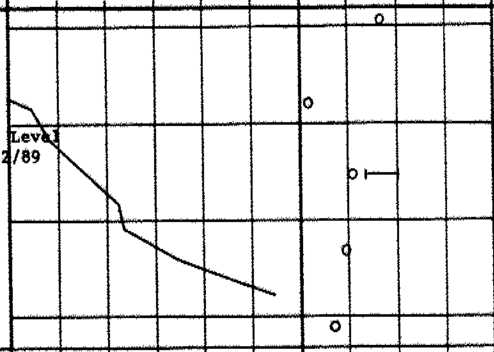
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ



$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W P 34-81-04 LOCATION Sta 10 + 080 o/s 4.0m L ¢ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger; Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 18, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.1	Ground Surface								
0.0	Fill, sand and gravel, trace silt. Compact Grey		1	CS	-	Seal	131		
130.5 / 0.6	Sand, trace gravel and silt. Compact to Dense Brown		2	SS	15		130		
			3	SS	37	Water Level Dec.22/89			
129.1 / 2.0	Silty Sand, trace gravel and some clay. Dense to very Dense Grey (Glacial Till) cobbles		4	SS	38		129		
			5	SS	63		128	22.4	
			6	SS	64		127		
			7	SS	45		126		
			8	SS	63		125		
124.5 / 6.6	End of Borehole								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 -ǒ- 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 34-81-04 LOCATION Sta 10 + 055 o/s 4m Rt. ¢ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 20, 1989 CHECKED BY PB

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
131.0	Ground Surface					
0.0	Fill, sand and gravel, trace silt. Compact Brown	1	CS	-	Seal	
130.3 / 0.7	Sand and Gravel, some silt, trace clay.	2	SS	50/50mm		130
	Compact to Dense Grey	3	SS	25	Water Level Dec.22/89	129
128.3		4	SS	50		
2.7	Silty Sand, trace gravel and some clay.	5	SS	98		128
	Very Dense Grey	6	SS	95		127
	(Glacial Till)	7	SS	51/150mm		126
	cobbles					125
124.4		8	SS	35		
6.6	End of Borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -○- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

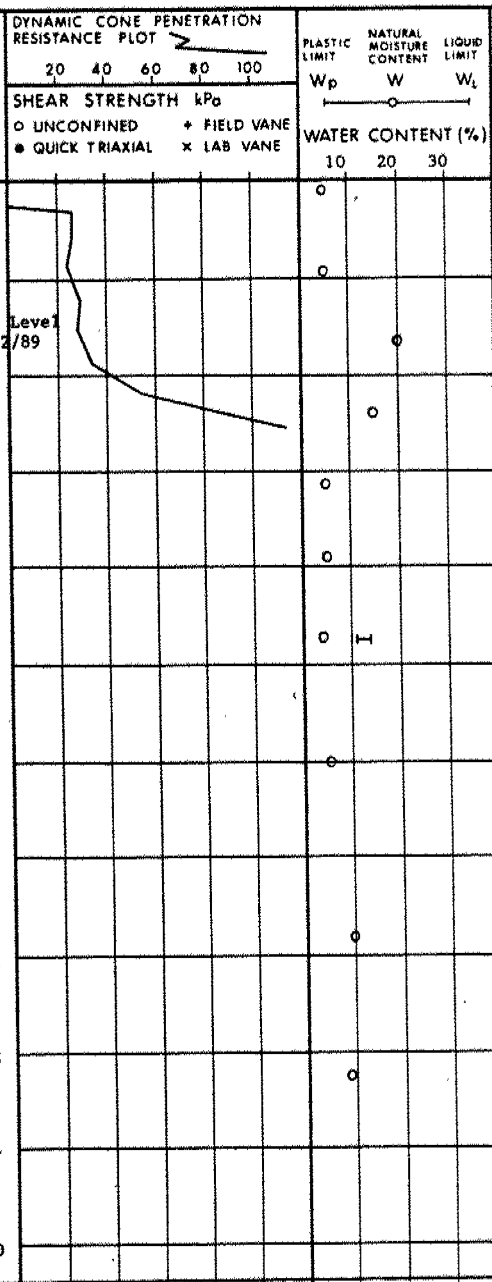
METRIC

W P 34-81-04 LOCATION Sta 10 + 030 o/s 4.5m Rt. ¢ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Core, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 15, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
131.0	Ground Surface										
0.0	Fill, sand and gravel, trace silt. Compact, Brown	1	CS	-	Seal						
129.8		2	SS	19		130					
1.2	Sand, some gravel, trace silt. Loose to very Dense, Brown	3	SS	8	Water Level Dec.22/89	129					
		4	SS	22		128					
127.6		5	SS	95							
3.4	Silty Sand, trace gravel and clay. Very Dense to Dense, Grey (Glacial Till)	6	SS	72/150mm		127					
		7	SS	87		126	22.9	8	52	32	8
		8	SS	95		125					
						124					
		9	SS	48		123					
		10	SS	40		122					
120.9	Auger Refusal					121					
10.1	Bedrock Limestone. Grey	11	RC NQ	100% RQD 90%		120					
119.6 / 11.4	End of Borehole										



+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 34-81-04 LOCATION Sta 10 + 030 o/s 4.5m Lt ℄ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 19, 1989 CHECKED BY PB

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.2	Ground Surface							
0.0	Fill, sand and gravel. Compact Brown	1	CS	-	Seal	131		
130.6 / 0.6	Sand and Gravel, some silt, trace clay. Compact Brown	2	SS	62	Water Level Dec.22/89	130		
		3	SS	27				
128.5		4	SS	25		129		
2.7	Silty Sand, trace gravel and some clay. Very Dense Grey (Glacial Till)	5	SS	70/150mm		128		
		6	SS	78		127		
		7	SS	60/150mm		126		
		8	SS	80		125	22.7	
		9	SS	49		124, 123		
		10	SS	34		122		
121.2 / 10.0	End of Borehole Auger Refusal on Probable Bedrock							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 6

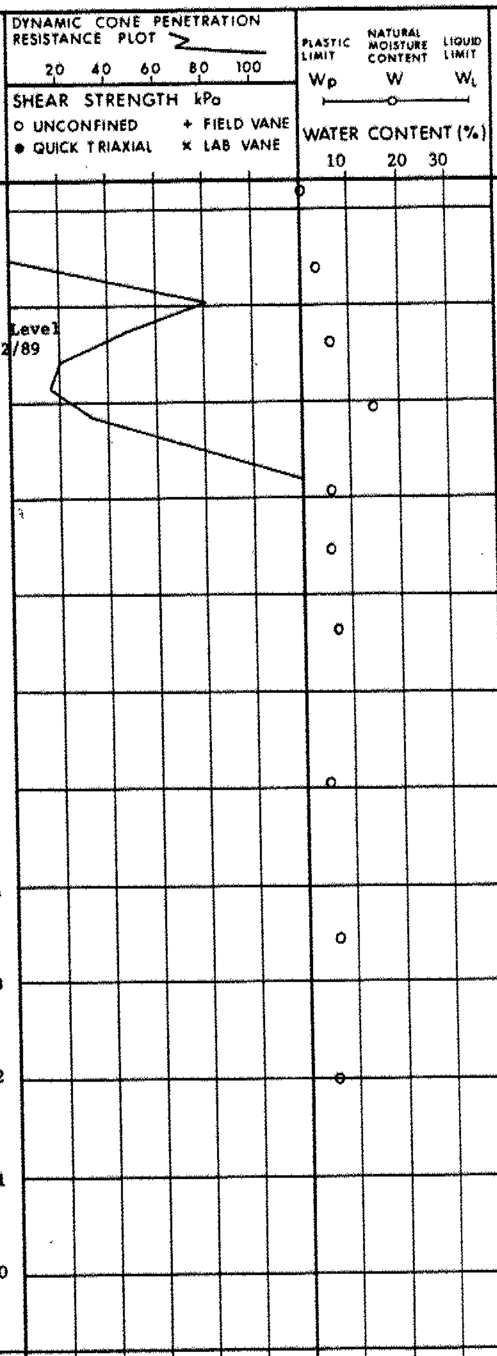
METRIC

W P 34-81-04 LOCATION Sta 10 + 000 o/s 0.0m ℄ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Core, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 15, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.3	Ground Surface							
0.0	Asphalt - 140mm Fill, sand and gravel.	1	CS	-	Seal	131		
130.1	Dense Grey	2	SS	41				
1.2	Sand and Gravel, trace silt and clay.				Water Level Dec.22/89	130		30 52 13 5
		3	SS	31				
128.6	Compact Brown	4	SS	26		129		
2.7	Silty Sand, trace gravel, some clay.	5	SS	88		128		
		6	SS	77/225mm				
	Very Dense Grey					127		
		7	SS	73			22.4	
	(Glacial Till)					126		
		8	SS	90		125		
						124		
		9	SS	32		123		
121.2		10	SS	41		122		
10.1	Bedrock, Limestone. Grey	11	RC NQ	100% RQD 71		121		
119.2		12	RC NQ	100% RQD 85		120		
12.1	End of Borehole							



+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 / 10)

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 7

METRIC

W P 34-81-04 LOCATION Sta 9 + 970 o/s 4.5m Rt ¢ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Core, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 13, 1989 CHECKED BY PB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.3	Ground Surface							
0.0	Asphalt - 125mm Fill, sand and gravel.	1	CS	-	Seal	131		
130.3 1.0	Compact Brown Sand and Gravel, trace silt and clay.	2	SS	23		130		
	Very Brown Dense	3	SS	62	Water Level Dec.21/89			
129.0 2.3	Silty Sand, trace gravel, some clay, occasional cobbles.	4	SS	88		129		
		5	SS	95		128		
	Very Grey Dense	6	SS	58				
	(Glacial Till)	7	SS	55/50mm		127		
						126		
		8	SS	93		125		
						124		
		9	SS	57		123		
		10	SS	81		122	22.1	8 50 28 14
121.1 10.2	Auger Refusal Bedrock Limestone. Grey	11	RC NQ	100% RQD 65%		121		
						120		
119.6 11.7	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE - PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 8

METRIC

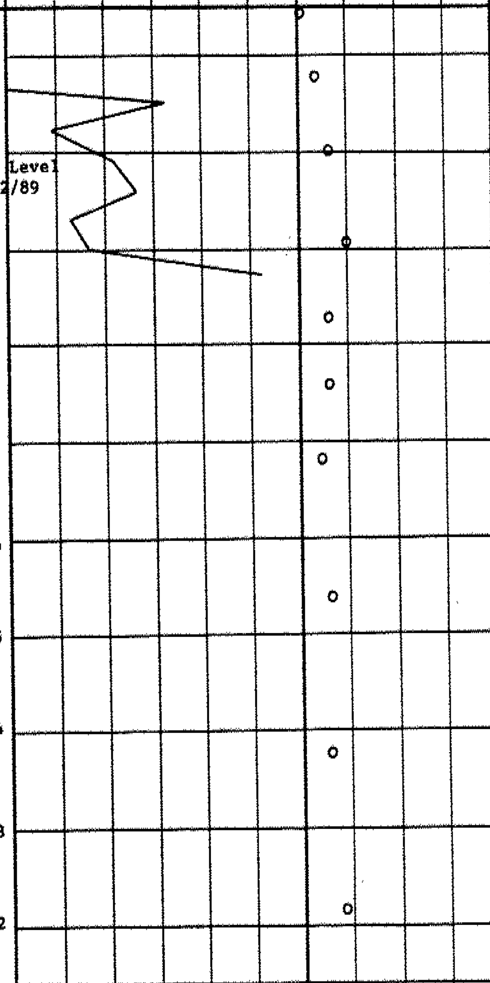
W P 34-81-04 LOCATION Sta 9 + 970 o/s 4.5m Lt. ₵ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 19, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.5	Ground Surface							
0.0	Asphalt - 125mm. Fill, sand and gravel. Compact, Brown	1	CS	-	Seal	131		
130.5		2	SS	83				
1.0	Sand and Gravel, some silt, trace clay.	3	SS	6/150mm	Water Level Dec.22/89	130		30 50 12 8
128.8	Very Dense to Compact, Brown	4	SS	27		129		
2.7	Silty Sand, trace gravel, trace to some clay.	5	SS	73		128	22.7	
	Very Dense, Grey	6	SS	53		127		
	(Glacial Till)	7	SS	72		126		
		8	SS	60/150mm		125	22.3	
		9	SS	55		124, 123		
	sand and gravel seam.	10	SS	38		122		
121.4 / 10.1	End of Borehole. Auger Refusal on probable Bedrock.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%)



+³, ×⁵ : Numbers refer to Sensitivity 15 ⟡ 5 (%) STRAIN AT FAILURE (20, 10)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 9

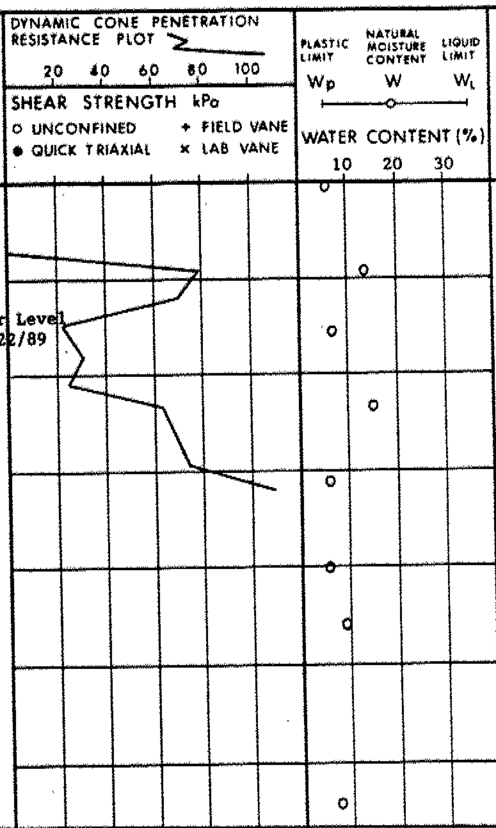
METRIC

W P 34-81-04 LOCATION Sta 9 + 945 o/s 6m Lt. ℄ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 12, 1989 CHECKED BY PB

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
131.0	Ground Surface					
0.0	Fill, sand, some gravel. Compact Brown	1	CS	-	Seal	
		2	SS	9		130
129.8 / 1.2	Sand and Gravel, some silt, trace clay. Compact Brown	3	SS	42	Water Level Dec.22/89	
		4	SS	20		129
128.3 / 2.7	Silty Sand, trace gravel, some clay. Very Dense Brown (Glacial Till)	5	SS	80		128
		6	SS	55		127
		7	SS	50/50mm		126
124.4 / 6.6	End of Borehole	8	SS	60		125



UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 10

METRIC

W P 34-81-04 LOCATION Sta 9 + 920 o/s 5M Rt. ₵ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KJ

DATUM Geodetic DATE December 12, 1989 CHECKED BY PB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
130.7	Ground Surface							
0.0	Fill, sand and gravel. Compact Brown	1	CS	-	Seal	130		
129.7 / 1.0	Sand and Gravel, some silt, trace clay. Compact Brown	2	SS	47	Water Level Dec.21/89			
		3	SS	17		129		
128.0 / 2.7	Silty Sand, trace gravel, some clay. Very Dense Brown (Glacial Till)	4	SS	32		128		
		5	SS	60				
		6	SS	73/150mm		127		
		7	SS	105		126	21.9	4 38 40 18
						125		
124.6 / 6.1	End of Borehole Auger and Spoon Refusal							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L – WATER CONTENT (%) 10 20 30

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 11

METRIC

W P 34-81-04 LOCATION Sta 9 + 895 o/s 5m Lt. ℄ Richardson Rd. ORIGINATED BY KJ

DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger, Cone Test COMPILED BY KJ

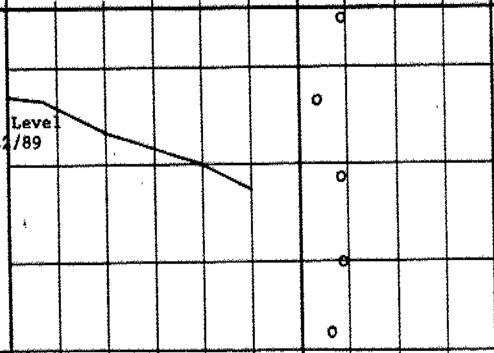
DATUM Geodetic DATE December 13, 1989 CHECKED BY PB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS / ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
130.6	Ground Surface							
0.0	Fill, sand and gravel.	1	CS	-	Seal 130			
129.4	Dense Brown	2	SS	57	Water Level Dec.21/89			
1.2	Silty Sand, trace gravel, some clay.				129			
		3	SS	64				
	Very Dense Brown (Glacial Till)							
		4	SS	88	128		21.4	
127.1		5	SS	113				
3.5	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

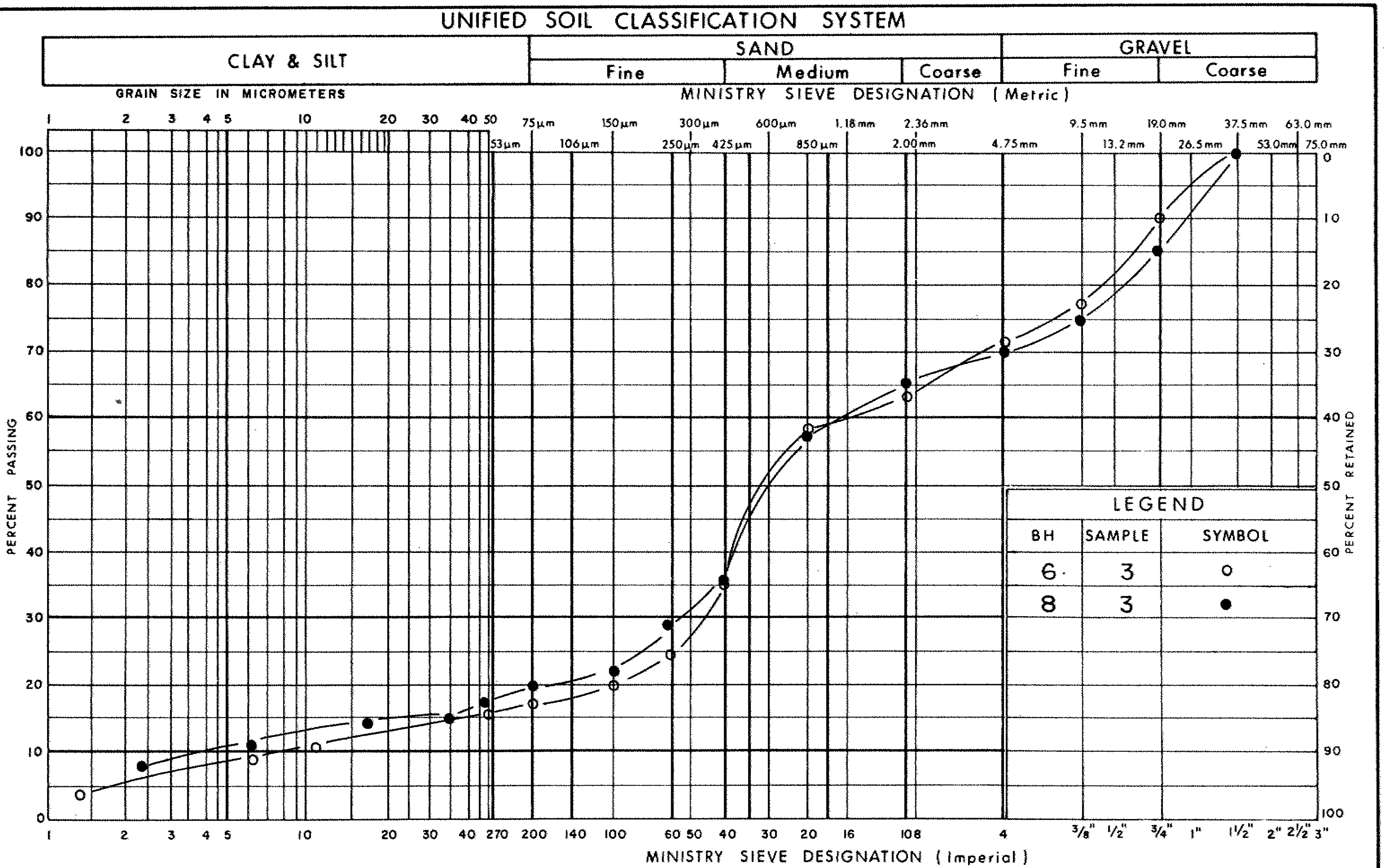
PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30



$+^3$, $\times^5$: Numbers refer to Sensitivity

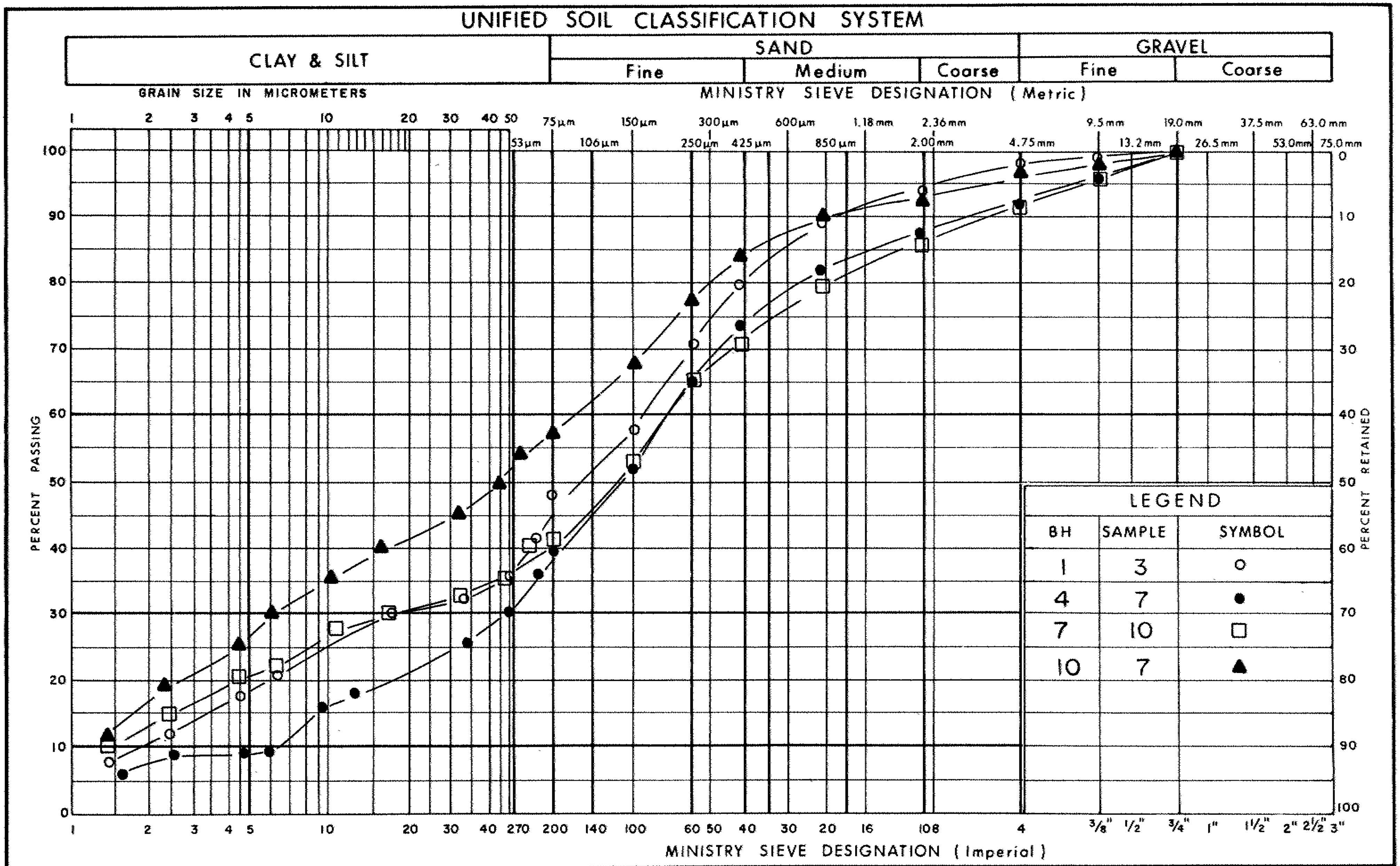
20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION	FIG No 1
	SAND AND GRAVEL SOME SILT, TRACE CLAY (SM) (TILL)	W P 34-81-04

57

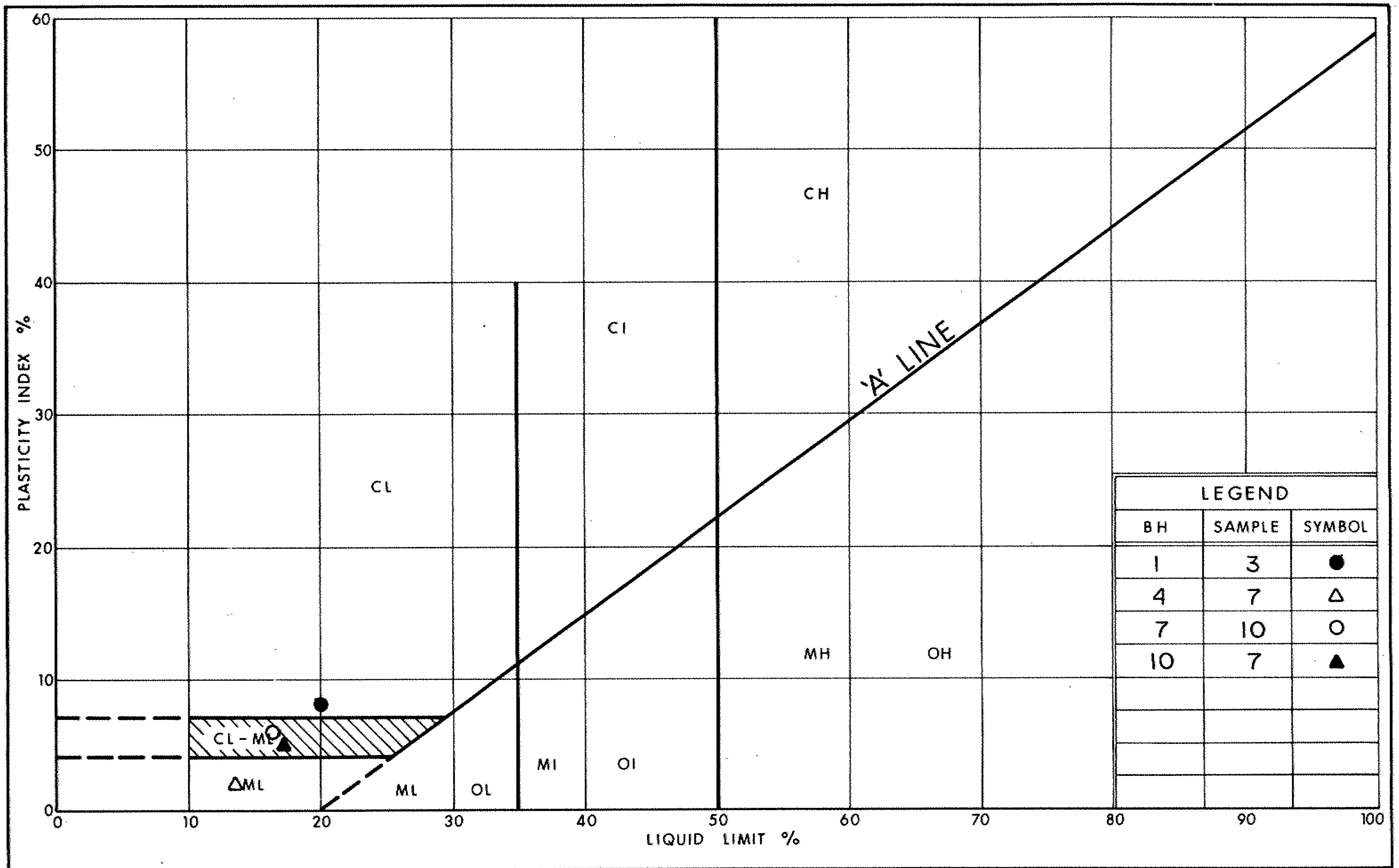


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY SAND TRACE GRAVEL, SOME CLAY (TILL) (SM)

FIG No 2
WP 34-81-04

Oct 75, FF-S-21



Ministry of Transportation
Ontario

PLASTICITY CHART

SILTY SAND TRACE GRAVEL, SOME CLAY (TILL)

FIG No 3

W P 34-81-04

T11600

REPORT TO

MINISTRY OF TRANSPORTATION OF ONTARIO
FOUNDATION DESIGN SECTION

SUBSURFACE GEOTECHNICAL INVESTIGATION PROPOSED HIGHWAY 17 AND 44, UNDERPASS OTTAWA ONTARIO

WP 34-81-02
CONT 91-52

GEOCRES # 31F-110

Distribution:

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Downsview, Ontario

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Mississauga, Ontario

March, 1990

Geocon

GEOTECHNICAL CONSULTANTS

GEOCON INC.
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March 1, 1990

Ministry of Transportation of Ontario
Foundation Design Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Attention: Mr. Murty Devata, P.Eng.
Chief Foundation Engineer

RE: SUBSURFACE INVESTIGATION
PROPOSED HIGHWAY 44 & 17 UNDERPASS
OTTAWA, ONTARIO

Dear Sirs:

We are pleased to forward our final report for the above noted study as per your request.

The report documents the soil and ground water conditions observed in boreholes drilled at the above noted site in the general areas of abutments, approach fills and central piers. Also presented in this report are recommendations for foundation types and considerations for the construction at the proposed bridge.

In absence of an "E" plan for this project we have not drafted a detailed stratigraphic section but instead we have enclosed a copy of a preliminary "working" section prepared by us. We will forward a properly drafted "E" plan and stratigraphic section in due course.

We trust this is sufficient for your present requirements and would be pleased to discuss any aspect of the study with you. Thank you for the opportunity to be of continuing service.

Yours very truly
GEOCON INC.

R.D. Powell, P.Eng.
General Manager

RDP:bg
T11600/15000

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1.0 INTRODUCTION

The Ministry of Transportation of Ontario, propose to upgrade the existing Highway 17, west of Ottawa from two lanes to a four lane expressway. In order to achieve this work, several regional roads in the area require grade separations over the proposed expressway. Geocon Inc. was requested to conduct a subsurface geotechnical investigation for one of these proposed grade separations, located at the intersection of Regional Road 44 and Highway 17 (Drawing No. T11600.1).

The purpose of this investigation was to determine the subsurface conditions present at the site and, based on that information provide geotechnical recommendations for use in the foundation design aspects of the project. The Terms of Reference for this investigation were outlined in our proposal letters of November 20, 1989 and January 15, 1990. This work was completed under Ministry of Transportation Work Order No. 4238-9089-240.

2.0 SITE GEOLOGY

The site is located adjacent to the Ottawa River on a sand plain along the near shore area of the historical Champlain Sea (Chapman and Putnam 1983). Expected soil conditions at site consist of a thin veneer of cohesionless material overlying significant depths of marine clay. These deposits are in turn underlain by glacial till which is overlying bedrock. These general soil conditions have been confirmed by previous subsurface investigations in the area (Ministry of Transportation of Ontario - Geo-Cres library).

The bedrock geological mapping at the proposed site, as presented in the Ontario Geological Survey (O.G.S.) Map P 2726, indicates that the underlying bedrock consists of limestone of the Verulam Formation of the Middle Ordovician Period. Typically, this formation consists of fine grained limestone bedrock interbedded with shale layers up to 100 mm thick. Outcrops of the formation occur approximately 2 km west of the proposed site.

3.0 SITE AND PROJECT DESCRIPTION

The site of the proposed overpass structure is located at the intersection of Highway 44 and 17, approximately 35 km west of Ottawa, Ontario (Drawing No. T11600.1). The site is located on a generally flat plain locally traversed by three low level embankments.

The largest of the embankments is that associated with the north-west trending Highway 17 (Drawing T11600.1) which is approximately 2.0 m above the existing ground level. In the east-west direction, two embankments intersect obliquely with the larger Highway 17 embankment. The smallest of these embankments, associated with the previous Highway 44 alignment, is generally less than 1.0 m above the existing ground level. The second, associated with the diversion of Highway 44 to a location some 60 m north of the previous location (Figure 1) is slightly higher as it grades up from surrounding ground level to meet Highway 17 at grade. A drainage ditch traverses the site in a roughly north easterly direction.

The proposed grade separation of the two highways will be achieved using a new overpass structure located on the old alignment of Highway 44. It is understood that the overpass structure will consist of two abutments approximately 82 m apart with an additional support located at centre span. Approach embankment fills of the order of 8 m to 9 m high immediately adjacent to the bridge abutments are proposed on both sides of the overpass.

4.0 INVESTIGATION PROCEDURE

The field work for this investigation was conducted in two phases, the first between December 4, 1989 and December 18, 1989 and the second between January 22 and January 25, 1990. During that time, a total of 9 boreholes was drilled at the locations shown on Figure 2. In addition, one dynamic cone penetration test was conducted from surface. Details of the drilling program are summarized in Table 1. The borehole logs and the results of dynamic cone penetration tests are presented in Appendix I.

The boreholes were advanced using 200 mm diameter hollow stem augers. The drill type used for this investigation was a Bombardier mounted CME 55, owned and operated by Johnson Drilling, Ottawa, Ontario. During advance of the boreholes, sampling of the subsurface materials was performed at regular intervals. Generally, in the upper 7.5 m of each borehole , sampling was performed at 0.75 m intervals and thereafter at 1.5 m intervals. Sampling was generally achieved using a split spoon sampler associated with the Standard Penetration Test. At selected locations in the cohesive units, undisturbed thin-walled Shelby tube samples were taken. All recovered samples were initially examined and logged in the field and thereafter transported to our Mississauga office for detailed visual and tactile examination. Sample types and locations are presented on the borehole logs (Appendix I).

In addition to sampling the subsurface materials, field insitu undrained vane tests were performed at regular intervals throughout the cohesive strata. Generally, the spacing of field vane determinations was 0.75 m in the upper 7.5 m and 1.5 m there-after. Also, as a general rule in the upper 6.0 m of the borehole, a split spoon sample was taken for material identification purposes after each field vane test was performed. Between 6.0 m and 15.0 m field vane tests and insitu sampling were spaced at approxi- mately 0.75 m. At depths in excess of 15.0 m, field vane tests and sampling were performed successively at 1.5 m intervals to depth. At Boreholes 4A and 8, drilled to penetrate the lower units, no sampling or insitu testing of the overburden cohesive strata was performed. Measured field insitu undrained vane strengths are presented on the borehole logs (Appendix I).

At Borehole 2, drilling and sampling of the underlying silty sand till layer was achieved by advancing B-size casing whilst washboring using a bentonite based drilling mud. This system was used to counteract the excess hydrostatic water pressures within the till layer which caused difficulty with material blowing up in the augers when this layer was penetrated using hollow stem augers.

The underlying bedrock was cored using a BX size core barrel at the locations of Borehole 2, 4A and 8 for total depths of 1.6 m, 6.2 m and 3.5 m, respectively.

In addition to monitoring the groundwater conditions during drilling, a total of four piezometers was installed in Boreholes 2, 3, 4A and 5. The installation details of these piezometers and the recorded water levels are presented on the borehole logs (Appendix I) and summarized in Table 1.

The field work was supervised at all times by a member of our engineering staff who supervised the drilling and sampling operations, ensured proper procedures for field vane tests were adhered to, logged the boreholes, cared for the samples obtained and supervised the installation of the piezometers.

The locations and ground surface elevations of the boreholes were confirmed in the field by the survey department, Ministry of Transportation of Ontario, Ottawa, Ontario prior to the commencement of drilling. It is understood that the elevations are related to geodetic datum.

5.0 LABORATORY TESTING

Laboratory testing on the recovered samples taken from within the overburden materials consisted of routine index testing on the disturbed split spoon samples and consolidation testing on selected Shelby tube samples.

Index testing consisted of moisture content determinations, grain size analysis, and Atterberg limits. The results of these tests are presented on the borehole logs (Appendix I). The results of 5 grain size analyses are presented on Figures 1 to 3 of Appendix II.

Four consolidation tests were carried out on Shelby tube samples taken from within the overburden materials. The locations of the tests are indicated on the borehole logs (Appendix I) with the test results presented on Figures 4 to 5 of Appendix II. An incremental load ratio of 2 was used for the consolidation test performed on Sample No. 4, Borehole 6. The remaining tests were conducted with an incremental load factor of 1.5.

One unconfined compression test was performed on a sample of the bedrock taken from Borehole 4A at a depth of 26.2 m (elevation 92.0 m). The test result is discussed in Section 6.0.

6.0 SUBSURFACE GROUND CONDITIONS

Based on the soil conditions encountered at the location of the boreholes, stratigraphic conditions at the proposed site generally consist of fill overlying a layer of sandy silt which in turn overlies silty clay. The silty clay was found to be underlain at depth by a silty sand till which in turn was underlain by limestone bedrock. A summary of the borehole information is presented on the stratigraphic section (Drawing No. T11600-2). More detailed information on the stratigraphy at the borehole locations is presented on the borehole logs (Appendix I). A description of these individual units is presented below.

Fill

A layer of fill, associated with the construction of the previous alignment of Highway 44 and the present Highway 17, was encountered in all the boreholes. The fill generally consists of a brown, medium sand. Standard Penetration "N" values within the fill ranged from 9 to 48 blows and the layer can generally be described as having a compact relative density.

At the locations of Boreholes 1, 2, 3 and 7, drilled from the surface of the previous alignment of Highway 44, the thickness of this layer is of the order of 1.2 m. At the locations of Boreholes 4, 5 and 6, drilled through the higher Highway 17 a maximum thickness of 2.1 m was recorded.

Silty Fine Sand - Sandy Silt

A layer of silty fine sand to sandy silt with trace clay, was encountered in all of the boreholes. At the location of Borehole 4 the thickness of this layer was determined at 4.1 m with an associated lower interface elevation of 111.9 m. However, at the remaining boreholes the thickness varied from 1.6 m to 2.4 m with an associated range in elevation of the underside of this layer from 113.2 m to 114.0 m. The reason for the observed increase in thickness of this layer at the location of Borehole 4 is not known.

Measured Standard Penetration Test "N" values within the layer varied from 3 to 28 indicating a very loose to compact relative density.

Tactile investigation of disturbed samples taken from within this layer indicate that the sand content decreases with depth while the silt and clay contents increase. Occasional small white shells were present within this layer. The results of two grain size analyses from samples taken from within this layer are presented on Figure 1 of Appendix II.

Moisture content determinations from within this layer with the exception of a value of 35 percent at Borehole 1, Sample 3 show values which range from 17 to 23 percent. One determination of Atterberg limits from a sample taken from within this layer gave plastic and liquid limit values of 17 and 19 percent respectively with an associated plasticity index of 2. The layer can generally be described as being non-plastic.

The results of one consolidation test performed from an undisturbed Shelby sample taken from within this layer (Borehole 6; Sample 4) is presented on Figure 4 of Appendix II with key parameters summarized in Table 2. The coefficient of recompression is estimated at 0.005. The test was not continued sufficiently far beyond the preconsolidation pressure to permit an accurate estimate of the coefficient of compression. The pre-consolidation pressure of this sample is estimated at to be at least 480 kPa with an associated over-consolidation ratio of at least 11.7.

<u>Silty Clay</u>

Present in all boreholes, the measured thickness of this layer was found to reduce across the site towards the west. Measured thickness varied from 8.0 m at Borehole 1 to 17.1 m at Borehole 6. The associated elevations of the underside of this layer were 105.9 m at Borehole 1 and 96.5 m at Borehole 6. The observed thinning of this layer toward the west is further illustrated on the stratigraphic section as presented in Drawing No. T11600-2.

Tactile inspection of split spoon samples taken from within the layer indicated that the layer generally consists of silty clay material interbedded with clayey silt layers (varves). The thickness of the clayey silt varves are of the order of 3 to 5 mm with spacings of the order of 25 to 30 mm. Within the bottom 3 to 4 m of the layer the thickness of the varves increases with an associated reduction in spacing.

The results of 2 grain size distribution tests on material from this layer are presented on Figure 2 of Appendix II. The gradations of these two samples include clay contents varying from 18 to 46 percent. This is believed to be indicative of the local gradation variances throughout this layer as a result of its varved nature.

Drawing No. T11600-3 presents a compilation plot of the key geotechnical parameters obtained within this layer. Specifically, it presents data on the measured field insitu undrained vane shear strengths, sensitivity, moisture content and consolidation data. These are discussed separately below.

- Insitu Undrained Vane Shear Strength

Individual measurements of the undrained shear strength with depth as determined in the field are presented on the borehole logs (Appendix I). The compilation plot of Drawing No. T11600-3 shows a generally observed trend of decreasing values with depth to a minimum value a short distance below the underside of the generally non-plastic sandy silt material. The minimum observed value is of the order of 40 kPa and occurs at an elevation of approximately 113.0 m or 1.5 m to 2.0 m below the underside of the upper sandy silt layer. Below the minimum value, the measured values show a general increase with depth, estimated to be of the order of 4 kPa per metre. The layer can be described as having a firm to stiff consistency.

- Sensitivity

Observed sensitivity values from within the silty clay layer, defined as the ratio of the peak undrained shear strength and the vane remoulded strength as determined after 10 revolutions of the vane, are presented on Drawing T11600-3. Individual values are presented on the borehole logs (Appendix I). In general, the sensitivity shows no discernible trend with depth, with values ranging from a minimum of 3 to a maximum of 31. However, the majority of the values fall in the range between 5 and 20 and the material can be described as being highly sensitive and is typical of Champlain Sea deposits of eastern Canada.

- Moisture Content

Observed moisture content from within the upper sandy silt and the silty clay are presented on Drawing No. T11600-3 with individual values being presented on the borehole logs (Appendix I). Generally, the upper sandy silt material has a constant moisture content of the order of 20 percent. From the underside of the upper layer, at approximate elevation 114.0 m, values show an increasing trend up to maximum of the order of 50 percent at elevation 106.0 m. Below this elevation, values are generally constant until the bottom of the layer where moisture content values tend to decrease.

The results of 5 Atterberg limits from within the layer show a plastic limit range of between 15 and 34 percent with liquid limit values varying from 20 to 37 percent. The associated plasticity index varies from 5 to 13, and based on these values, the layer can be described as being of low to medium plasticity. The higher plasticity index values are presumed to be associated with the more clay rich samples. Also, it is expected that the silty clay varves have significantly higher plasticity values than those obtained from the "bulk" samples which were tested. Insitu moisture content values throughout the layer are generally higher than the measured liquid limit values.

- Consolidation Test Data

Four consolidation tests were performed on undisturbed Shelby samples taken from Borehole 6 and 7 (Appendix II - Figures 4 and 5). A summary of the test data is presented in Table 2 with estimated preconsolidation pressures with depth presented graphically on Drawing No. T11600.3.

As indicated on Drawing No. T11600.3 the silty clay deposit is highly overconsolidated with Over Consolidation Ratios (OCR) of the order of 3.2 to 3.4. However, the observed preconsolidation pressure profile is linear with depth indicating a consistent trend within this deposit. Mesri (1975) relates the undrained shear strength of clay deposits to preconsolidation pressure. Therefore, the observed linear increase of preconsolidation pressure with depth is in keeping with an observed similar trend for the field insitu undrained vane shear strength.

Compression Index values for the tests vary from 0.305 to 1.06 with re-compression index values ranging from 0.013 to 0.040. As presented in Table 2, the compression and recompression indices of the deepest test are significantly higher than the other two tests performed within this layer. This indicates that the material at depth within this layer may have a more brittle structure than those closer to the surface. This conclusion is supported by the higher observed natural moisture content in this region of the layer.

Coefficient of Consolidation values (C_v), are significantly higher in the pressure range up to the preconsolidation pressure than thereafter. Typical C_v values up to the preconsolidation pressure are of the order of 25 to 50 m^2/yr. Within the virgin consolidation range, typical C_v values are of the order of 3 to 15 m^2/yr.

Till

As encountered at the boreholes, the underlying till layer becomes deeper from west to east across the site. This trend is illustrated on the stratigraphic section presented on Drawing No. T11600-2. The surface of this layer was at elevation 105.9 at Borehole 1 and at elevation 95.8 m at Borehole 8.

The thickness of this layer in Boreholes 2, 4A and 8 was 4.2 m, 4.6 m and 2.0 m, respectively. At Borehole 6, auger refusal within this layer suggests a minimum thickness of 3.1 m. Further, dynamic penetration tests performed at the bottom of Boreholes 1 and 3 infer minimum thicknesses for this layer of 4.5 m and 7.2 m, respectively, for these locations. Hence, it can be concluded that the thickness of this layer is quite variable although it does appear to increase in thickness towards the west.

Visual and tactile examination of recovered samples indicate the till consists primarily of a mixture of silty sand with some gravel. Boulders, up to at least 0.5 m diameter (Borehole 4) were confirmed to exist within the deposit. A grain size analysis, performed on a sample taken from within this layer is presented on Figure 3 of Appendix II.

Measured "N" values within the deposit are quite variable and may be unreliable due to the "blowing up" of sand within the hollow stem augers while drilling within the layer. Also, as previously discussed, the deposit contains boulders which may also influence the measured "N" values. However, based on the measured "N" values as well as dynamic cone penetration tests conducted at Boreholes 1 and 3 and adjacent to Borehole 6, the upper 3.0 m of this layer appears to be generally in a loose state of relative density. Beyond 3.0 m depth into the layer, the measured "N" values tend to show a general increase.

Bedrock

The surface of the bedrock, as confirmed at Boreholes 2, 4A and 8, tends to rise from east to west across the site. Specifically at Boreholes 2, 4A and 8 the surface of the bedrock was determined to be at elevations 97.3 m, 94.4 m and 93.8 m, respectively. At these boreholes, the total lengths of cored bedrock was 1.5 m, 6.1 m and 3.3 m, respectively.

The bedrock consists of unweathered fresh limestone with thin (<1.0 mm) very closely spaced shale partings. Occasionally thicker horizons of shale up to 30 mm thickness were found in the deposit. The thin shale partings within the rock are generally tight but have a low tensile strength and hence splitting on these features occurs quite readily, especially during drilling. This is believed to be a major reason for the observed low RQD values reported on the borehole logs (Appendix I). Further, the shale partings are primarily non linear features and in some instances within the dimensions of the the core, the observed surfaces were quite irregular.

The shale on the thin partings is estimated to be of very soft rock strength. However, where the thickness of these features increases beyond 1 to 2 mm their strength was indistinguishable from the host limestone rock.

One Unconfined Compression Test strength determination, on a sample taken from Borehole 4A, gave a value of 48 MPa. The tested sample was representative of the general observed rock formation with very closely spaced thin shale partings as described above, present along its entire length.

Groundwater

Four standpipe piezometers were installed at the proposed site to monitor the ambient groundwater regime. One of the piezometers (Borehole 5) was sealed within the upper silty sand while another (Borehole 2) was sealed into the silty clay layer. The remaining piezometers were sealed into the underlying silty sand till.

Groundwater water levels within the upper sandy silt and silty clay appear to be hydrostatic with a groundwater elevation of approximately 116.0 m or approximately at the level of the existing ground surface. The piezometric head within the underlying till is below this level as measured at 113.8 m in Borehole 3. Unfortunately, it was not possible to obtain a reading on the piezometer installed in Borehole 4A because of inclement weather after its installation. However, based on the available information to date, it appears that a situation of vertical drainage to the underlying silty sand till layer exists. This may explain in part, the observed linear increase in depth of moisture contents within the upper silty clay material (Drawing No. T11600.3).

7.0 ENGINEERING DISCUSSION AND RECOMMENDATIONS

7.1 General

The main geotechnical input to design of the proposed project is in the selection of the most suitable foundation system to support the superstructure loads, and the establishment of details for the highway approach embankments. These two issues are addressed herein in Sections 7.2 and 7.3. A number of general recommendations from a geotechnical engineering standpoint, are presented in Section 7.4.

The information presented in this section of the report is intended for use primarily by the design engineers for the project. Contractors bidding on all or part of the work must satisfy themselves in respect of the adequacy of the information and make their own interpretations of how it affects their construction equipment, methodology, scheduling, and the like, in accordance with contractual provisions specified by the Ministry of Transportation.

7.2 Bridge Foundation Alternatives

At the time of writing of this report, little information is available to us with regard to deformation tolerances for the proposed structure, anticipated loads and the like. However, based on the observed soil conditions at the proposed site and experience with similar projects, the following comments and results of preliminary analyses are presented for use in the design process leading to the selection of the most suitable foundation system for the proposed overpass structure, and the earthworks forming an integral part of it.

The dominant soil formation encountered at the proposed site is up to 17.1 m of sensitive silty clay (Sections 6.0). This formation has favourable strength properties but is highly compressible at pressures in excess of the preconsolidation pressure (Table 2). The silty clay is underlain by a relatively thin layer of silty sand till (<5 m) which in turn is underlain by bedrock.

Potential foundation systems for the bridge structure involve the utilization of the sensitive clay layer as a founding medium or alternatively, the use of pile type foundations to transfer the loads through this layer to the more competent material at depth. As a general comment, any foundation system which utilizes the sensitive clay layer as a founding medium may, depending on design details, result in significantly higher soil deformations than systems which transfer structural loads through the clay to the underlying till or bedrock.

Potentially suitable foundation alternatives which utilize the upper sensitive clay layer include either conventional spread footings or friction piles. However, for the probable loads and configuration of the structure, the use of conventional spread footings does not appear to be a viable alternative, although the measured undrained shear strength and compressibility characteristics of the clay layer are sufficiently favourable to warrant examination of the possibility of using friction piles.

Potentially suitable pile foundation alternatives which are based on transfer of structural loads through the clay include piles deriving their support in the silty sand till or driven to bedrock. The use of the silty sand till for founding end bearing driven piles is not recommended because of its variable and generally low relative density and the known presence of large boulders which affect the reliability of any pre-determined set criterion. The effects of variable relative density and the presence of cobbles and boulders would not preclude the use of expanded base concrete (Franki type) piles. However, the observed thickness of this layer at the proposed abutments and central pier location (typically limited to 3.0 to 4.5 m) together with the depths involved, may not be sufficient to merit the use of this pile type over piles end bearing on bedrock. Hence, expanded base concrete piles are not considered further herein, although they are not eliminated as a potential contender.

Finally, in the process of selection of the most suitable foundation system, the use of drilled concrete caissons socketed into bedrock, may be considered.

For purposes of this report, the following foundation systems have been selected for discussion in respect to the main geotechnical factors which will be applicable. As the design of the project progresses, and the structural loads and deformation tolerances of the superstructure are defined, a more detailed analysis of the proposed foundation alternatives can be performed in accordance with the Ultimate and Serviceability Limit State criteria as detailed in the Ontario Bridge Design Code (1983).

1) Driven Piles End Bearing on Bedrock
2) Drilled Concrete Caissons Socketed into Bedrock
3) Friction Piles in the Silty Clay

1) Driven Piles End Bearing on Bedrock

A variety of driven piles such as closed-end steel tube piles, steel H-Piles or precast concrete piles could be used for this alternative. Provided this pile is driven with a hammer generating at least 30 kNm per blow to a final set criterion of at least 10 blows per centimetre, the design of the pile can be calculated based on the structural capacity of the pile up to a maximum allowable end bearing pressure at the ultimate limit state of 7.5 MPa. The allowable end bearing pressure for piles of this type, has been reduced to limit potential settlement of the pile tip which may occur as a result of compression of the very closely spaced shale partings, present in the bedrock. However, this allowable end bearing pressure may be increased, based on the results of field load tests or local engineering experience with similar pile types founded in this bedrock formation. Specifically, providing other structural criteria are adhered to, a steel 310 x 110 H-pile (or similar) can be designed for an ultimate load of 720 kN.

Provided the "set" criteria for the piles are achieved at, within, or close to the bedrock surface, settlements associated with this foundation type could reasonably be expected to be in the order of the elastic compression of the pile for the range of working loads which will probably be involved.

Installation of piles of this variety will have to contend with the presence of boulders within the silty sand till. Therefore, driving depths should be closely monitored to detect piles which may encounter refusal at an unacceptable distance above bedrock. Provisions, such as pre-augering or pre-boring in advance of pile driving operations to clear the path to the bedrock surface, and the use of special pile tips, and careful driving procedures, warrant consideration to minimize possible damage to piles during installation.

Design of this pile foundation system should incorporate the loads generated by negative skin friction forces where applicable. These will be of relevance particularly at the abutment locations as a result of consolidation of the silty clay layer under the imposed load of the highway approach embankments. The possible effects of bending induced forces in the piles by embankment-induced loads and deformations, should also be taken into account in design, as discussed later.

2) Drilled Concrete Caissons Socketed into Bedrock

Drilled concrete caissons suitably socketed into the bedrock offer a positive method of transferring the bridge loading into the bedrock and may represent a cost effective foundation solution.

The caissons should (on the basis of bearing capacity considerations), be socketed a minimum of 1.0 m into the underlying bedrock to ensure the caisson base is located in rock of acceptable quality. An allowable end bearing pressure at the ultimate limit state of 5 MPa can be assumed at this level for design purposes. If a minimum socket depth of 2.0 m is used an allowable bearing pressure at the ultimate limit state of 10 MPa can similarly be assumed for design purposes. From end bearing considerations only, these allowable bearing pressures represent maximum ultimate limit state loads of 1800 kN and 3600 kN at socket depths of 1.0 m and 2.0 m respectively, for a 0.6 m diameter caisson. However, the structural capacity of the caisson in respect of bending, buckling, etc. may preclude the use of this magnitude of loading.

Socketing operations will most likely result in significant disturbance on the walls of the rock excavation as a result of splitting along the shale partings which are common in the upper 5.0 m of the bedrock. Hence, it is recommended that the frictional resistance along the shaft of the caisson located within the bedrock not be relied upon in the calculation of the ultimate capacity of the caisson. Expected settlements for this pile type would reasonably be expected to be in the order of the elastic compression of the caisson shaft.

As with option (1), the caissons should be designed for the additional loads arising from negative skin friction forces and bending, where applicable.

Effective installation of this type of foundation unit conventionally requires provision for careful inspection and monitoring, and use of various practical techniques, which are beyond the scope of this report to enumerate in detail. Only a few of the latter are therefore discussed herein by way of illustrating this point. Firstly, it will be necessary to install caissons through the till layer which is known to be of variable density and contain boulders up to at least 0.5 m in size. Secondly, the till is sandy and thus permeable. This in conjunction with the high hydrostatic conditions within the till and the generally fractured nature of the upper approximately 0.5 m of the bedrock will have to be contended with in sealing the caisson casing at the bedrock contact, and dewatering of the socket. This may make it impossible (in a practical sense) to effectively dewater the base of the caisson socket for inspection purposes and placing of concrete in the dry.

For the specified design bearing pressures, it is important that all loose and disturbed material be removed from the base of the caisson excavation prior to placing the concrete. This important item is of particular significance if the caisson socket cannot be adequately dewatered in advance of placement of concrete.

3) Friction Piles in the Silty Clay

The relatively deep deposit of the silty clay stratum, and its engineering characteristics may warrant consideration of this pile type. Its main advantage is that it will permit the use of shorter pile lengths than for the other foundation alternatives. Piles of this variety can be of various material types including timber piles.

The factored shaft resistance at the ultimate limit state of a single pile terminating within the silty clay layer (ignoring tip resistance) can be estimated for preliminary design purposes, using the relationship (Ontario Bridge Design Code, 1983b),

$$Q_{sf} = C_f msrA_s$$

where
Q_{sf} = Total Factored Shaft Resistance
A_s = Surface Area of Pile within the silty clay layer
C_f = Undrained Shear Strength (factored)
m = Adhesion Coefficient
s = Shape Factor
r = Regeneration Coefficient

For the generally stiff clays present at the proposed site, a value of 0.7 can be assumed for the regeneration coefficient. For timber and concrete piles, an adhesion coefficient value of 1.0 can be assumed with a value of 0.7 if steel piles are used. A shape factor of 1.0 can be assumed for cylindrical or prismatic piles with a value of 1.2 if the piles are tapered. The factored undrained shear strength can be taken as 0.5 times the characteristic undrained strength.

Based on the above values, the estimated Ultimate Limit State load for a 0.3 m diameter timber pile installed 10 m into the silty clay layer (to at least elevation 103.5 m) is estimated at 235 kN. However, the allowable loads for the pile groups which will likely be required for this project, should this system be adopted, will have to be calculated taking the group effect into consideration. Also, allowable loads may be controlled by consideration of tolerable settlements for the structure at the serviceability limit state. It is recommended that at least two pile load tests be conducted on site in accordance with accepted good practice, should this foundation system be adopted.

Expected settlements for a single timber friction pile 0.3 m in diameter and installed 10 m into the silty clay layer, estimated using the method proposed by Bengtsson and Sallfors (1983), should be of the order of 10 mm at the total factored shaft resistance load. However, this figure is subject to review based on the results of the pile load tests and other factors such as group effects, negative skin friction, settlements due to dissipation of excess pore pressures generated by pile driving, and the like. Also, as the design proceeds and more information becomes available, settlements can be calculated for the serviceability limit state condition. An unfactored characteristic shear strength can be used in the calculation of settlements at the serviceability condition.

In the design of a foundation system based on use of friction piles, consideration will have to be given to the way negative skin friction develops along the piles or pile groups particularly at the abutment locations. The estimation of skin friction loads, pile group settlements, and soil-structure interaction, are not amenable to precise theoretical methods.

All of the above pile foundation alternatives must be designed to accommodate rotation induced forces which may occur at the abutment locations as a result of the settlement of the approach embankments adjacent to the abutment.

Three technically feasible foundation alternatives for the proposed overpass structure have been presented in the preceding paragraphs. The three have differing degrees of geotechnical input to design, and differing performance characteristics and installation details. A final decision on the most suitable foundation system must be based on concurrent consideration of factors beyond the scope of this report, such as structural design requirements for the bridge deformation tolerances, the overall interaction between the foundation system, bridge superstructure and approach embankments, and comparative economics.

7.3 Highway Approach Embankments

7.3.1 Stability

From preliminary cross-sections supplied to us it is understood that the proposed crest elevation of the approach embankment fills, adjacent to the bridge abutments, is approximately elevation 124.5 m. The average elevation of the natural ground below the surficial fill is of the order of 116.0 m (Table 1) translating to a maximum fill height in the order of 8.5 m. Based on this height, and an embankment side slope of 2.5 horizontal to 1 vertical, a total stress analysis on the stability of the embankment was performed using the PC-SLOPE modified Bishop program. The minimum calculated Factor of Safety against rotational and translational shear failure were 1.48 and 1.65, respectively. The location of the critical surfaces and other key computer print-out data is presented in Appendix III. The embankment side slope of 2.5 horizontal to 1 vertical used in the stability analyses was required to obtain acceptable factors of safety. If preferred, the designers of the project may use a combination of berms with side slopes of 2 horizontal to 1 vertical as long as the overall slope, as measured from the crest of the slope through the centre line of the berm(s), is not greater than the design side slope used in the analyses.

The idealized soil and strength profile used in the stability analyses is presented on Figure 1 pf Appendix III. Also presented on Figure 1 is a compilation plot of all measured field insitu undrained vane shear strengths from the boreholes. The design strength profile with depth is seen to envelope all of the measured values on the conservative side.

Also indicated on Figure 1 are the estimated shear strength values at the location of the consolidation tests based on the relationship proposed by Mesri (1975) that the undrained shear strength (C_u) = 0.22 P_c, where P_c is the preconsolidation pressure. Several authors (Trak et al, 1980; Chapuis, 1982) have concluded that this relationship quite accurately predicts the mobilized shear strength in clay deposits underneath embankments in eastern Canada. Again, the strength profile with depth used in the stability analyses is on the conservative side of this value.

7.3.2 Embankment Settlement

The estimated total elastic and consolidation settlements, (at the centreline of the embankments adjacent to the bridge abutments, based on an 8.5 m fill height) is of the order of 100 mm. This value is based on consolidation test data as presented in Table 2. The majority of this settlement is associated with loading in the pressure range below the preconsolidation pressure and will therefore occur quite quickly. The magnitude of expected settlements at the centreline of the proposed embankment at distances away from the abutment will be roughly proportional to the height of fill at that location relative to the maximum of 8.5 m assumed in the calculation.

In the design of the abutment foundations, due regard must be given to the additional forces that may be induced on the foundation as a result of the vertical rotational movement generated by the settlement of the approach embankments.

7.4 General

The following general comments are made to illustrate some of the many geotechnical engineering and practical construction factors involved.

7.4.1 Pile Installation

- Friction Piles

  During installation of this pile type careful records of the pile driving must be maintained for all piles. Specifically, the installed tip elevation of each pile must be in accordance with the design elevation. Also, once installed the pile top elevation must be closely monitored and the pile re-driven should heave be determined to have occurred as a result of the installation of adjacent piles.

  The installation procedure of this pile type within the silty clay layer will generate significant excess pore water pressures immediately adjacent to the pile. It is important that these pore water pressures be allowed to dissipate before loading of the pile occurs. Therefore, it is recommended that a minimum period of 14 days, and more preferably 30 days, be allowed to pass prior to pile loading.

- Piles End Bearing on Bedrock

  Careful installation records should be maintained for each pile of this type. In addition to maintaining records of the pile "set" prior to the designated design "zero set" criterion, the tip elevations of adjacent piles must be recorded in an attempt to detect pile refusal depths which may have occurred on boulders within the silty sand till. As mentioned previously, special provisions will probably be required to facilitate pile installation to bedrock. Once installed, the pile top elevation should be closely monitored and the piles re-driven should heave of the pile be determined to have occurred as a result of the installation of adjacent piles.

7.4.2 Caisson Installation

If this solution is adopted, the base of each caisson socket should be inspected to ensure that the recommended minimum depths into bedrock are achieved and that the bedrock at founding depth is of acceptable quality. Checking of bedrock quality below base of socket, should also be made by probing or core drilling, in accordance with accepted good practice. Also, any loose debris in the base of the socket should be cleared out prior to pouring concrete.

7.4.3 Depth of Frost Penetration

The estimated depth of frost penetration at the proposed site location is 1.8 m (Canadian Foundation Engineering Manual).

Therefore, the base elevation for foundation pile caps should be located at least this distance below the final adjacent proposed grade. If this minimum distance cannot be guaranteed, an equivalent thickness of insulation such as Styrofoam SM should be provided.

7.4.4 Lateral Earth Pressure

Providing free draining material is placed within 3.0 m from the abutments, lateral earth pressures acting against the back of the abutment may be estimated for preliminary design purposes using the relationship,

$$p = K\gamma H$$

where
- p = Pressure acting at Depth H
- γ = Unit weight of embankment fill Material
- H = Depth below the final grade of the embankment
- K = Earth Pressure Coefficient

Providing large vibratory compactors are not used within 3.0 m from the back of the abutment, an at rest soil condition can be assumed to exist behind the abutment and a K value of 0.5 can be used. Alternatively, if movement of the abutment will be permitted, an active soil condition can be assumed to exist and a value of 0.33 used for K in the analysis.

Lateral earth pressures against the abutments should also be reviewed during design, with due cognisance of effects of embankment settlement and resulting rotation of the abutments in a vertical plane.

7.4.5 Temporary Excavations

Temporary excavations at the site will most likely be within the upper sandy silt material. Excavations in this material, to a minimum base elevation of 114.0 m should be stable in the short term if excavated no steeper than 1 horizontal to 1 vertical. However, the governing requirements of support for temporary excavation may be as stipulated in health and safety regulations for the area.

Proposed excavations below a base elevation of 114.0 m will require analysis in light of the underlying weak silty clay. This analysis is beyond the scope of work for this report.

The water elevation at the site is at an approximate elevation 116.0 m. Excavations below this level can expect some seepage to occur. However, the expected magnitude of water flow through the fine grained sandy silt material should be low and accommodated by procedures using a filtered sump and pump arrangement. In excavations below elevation 115.0 m problems may be experienced with boiling sand conditions due to the cohesionless nature of the sandy silt material, unless suitable advance dewatering is carried out.

The bases of excavations should be kept dry and not allowed to deteriorate prior to the placing of concrete. This can best be achieved if the last 0.15 m of material is excavated a short time (hours) before placing of the concrete. If this cannot be achieved, a 50 mm thick mudmat of weak concrete should be placed in the base of all excavations upon the completion of all excavation activities. Also, during low temperatures, adequate precautions should be taken to prevent freezing of the ground below the excavated base.

7.4.6 Embankment Construction

Prior to the placement of any fill material for the proposed approach embankments, all organic material located within the base of the proposed fill should be removed. In addition, any soft or organic zones detected during site stripping operations should be subexcavated and backfilled with suitable material.

The scope of work of this project did not include the evaluation of potential construction materials or potential borrow sources. However, as a general rule the embankment material should be of acceptable quality and compacted to a minimum of 95 percent of its Standard Proctor Optimum dry density as determined in the laboratory for it to afford adequate subgrade support to the highway.

The portions of the embankments with final elevations less than 118.0 m should be constructed entirely of non frost susceptible material. At other locations, the upper 2.0 m of the embankments should be constructed using non frost susceptible material. Material meeting the gradation levels of MTO Granular "B" or better will suffice.

Design of the pavement should incorporate the effects of potential frost heave if any frost susceptible material is located within 2.0 m of the final grade.

7.4.7 Site Supervision

Various assumptions and recommendations pertinent to the effective implementation of the geotechnical design aspects of this project have been presented in the preceding sections of this report. During construction, it is important that these items of work be performed under suitably qualified geotechnical engineering supervision.

8.0 CLOSURE

The fieldwork for this report was performed under the supervision of Mr. I. Corbett, P.Eng. who also wrote this report. This report was reviewed by Mr. R.D. Powell, P.Eng. and Mr. M.A.J. Matich, P.Eng.

The contents of this report are subject to the General Conditions and Limitations to be found following the text of this report.

Yours very truly
GEOCON INC.

for/ I. Corbett, P.Eng.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
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R. D. POWELL
PROVINCE OF ONTARIO

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GEOCON 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 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.

TABLE 1 - BOREHOLE SUMMARY

Borehole No.	Location				Stratigraphic Lower Elevations (Layer Thickness in Brackets)					Groundwater		
	General	Chainage (km+m)	Offset (m)	Ground Elevation (m)	Fill (m)	Silty Sand (m)	Silty Clay (m)	Till (m)	Bedrock Surface Elevation	Piezometer Installed	Piezometer Tip Elevation (m)	Groundwater Elev. (m)
1	West Approach Fill	10+090	0.0	116.84	115.47 (1.37)	113.91 (1.56)	105.87 (8.04)	104.65* (1.22)	Not Confirmed	No		
2	West Abutment	10+046	8.7 Right	116.62	115.40 (1.22)	113.72 (1.68)	101.53 (12.19)	97.31 (4.22)	97.31 [1.52]	Yes	107.48	115.78
3	West Abutment	10+037	5.2 Left	116.81	115.59 (1.22)	113.76 (1.83)	103.40 (10.36)	102.64* (0.76)	Not Confirmed	Yes	103.40	113.81
4	Central Pier	10+000	0.0	118.17	116.04 (2.13)	111.92 (4.12)	98.97 (12.95)	96.83* (2.14)	Not Confirmed	No		
4A	Central Pier	10+000	1.0 Right	118.17	116.04 (2.13)	111.92 (4.12)	98.97 (12.95)	94.40 (4.57)	94.40 [6.14]	Yes	95.32	Not Measured
5	East Abutment	9.963	5.0 Right	117.46	115.33 (2.13)	113.19 (2.14)	96.74 (16.45)	95.67 (1.07)	Not Confirmed	yes	113.65	116.06
6	East Abutment	9+954	9.0 Left	117.23	115.71 (1.52)	113.57 (2.14)	96.51 (17.06)	93.46* (3.05)	Not Confirmed	No		
7	East Approach Fill	9+910	0.0	116.94	115.72 (1.22)	113.28 (2.44)	103.99* (9.29)	-	Not Confirmed	No		
8	East Abutment	9+957	5.0 Right	117.16	-	-	95.83 ?	93.79 (2.04)	93.79 [3.30]	No		

Notes

1) Asterisk indicates that layer was not fully penetrated
Elevation given is lowest elevation confirmed by drilling

2) Elevations given are assumed to be Geodetic

3) Square brackets indicate depth of bedrock drilling

4) Consult borehole logs for more detailed information

TABLE 2

Consolidation Test Data

Sample Location					Test Data					
Test #	Borehole No.	Sample No.	Depth (m)	Elevation (m)	σ'_v (kPa)	P_c (kPa)	O.C.R.	C_c	C_r	e_o
1	6	4	3.27	113.96	41	>480	>11.7	-	0.005	0.455
2	7	6	4.87	112.07	56	182	3.25	0.360	0.015	0.934
3	6	8	7.92	109.31	77	259	3.36	0.305	0.013	0.975
4	6	11	12.50	104.73	112	383	3.42	1.060	0.040	1.442

Definitions

σ'_v - Existing Vertical Effective Stress
P_c - P_c Estimated Preconsolidation Pressure
O.C.R. - Over Consolidation Ratio
C_c - Compression Index
C_r - Reload Compression Index
e_o - Initial Void Ratio

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APPENDIX I

Borehole Logs

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:

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

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

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

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 $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

use co-ordinates values

Ministry of Transportation Ontario

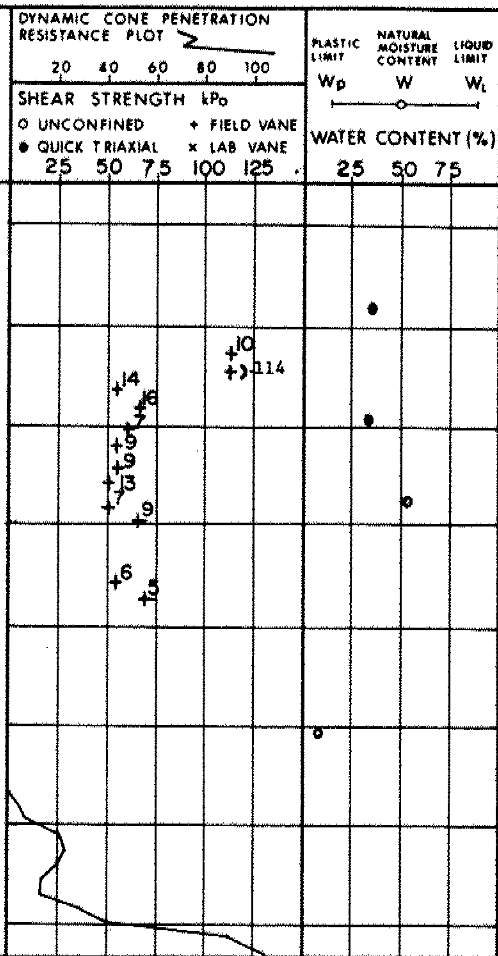
RECORD OF BOREHOLE No 1 — METRIC

W P 34-81-02 | LOCATION CH 10 + 090 (Hwy. 44) Centre Line | ORIGINATED BY R.K.

DIST 9 HWY 44 | BOREHOLE TYPE Hollow Stem Auger & Penetration Test | COMPILED BY I.C.

DATUM Geodetic | DATE December 14, 1989 | CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE
116.84	Ground Level				
0.00	Compact, brown sand.				116
115.47	Fill	1	SS	18	
1.37	Compact to loose, grey silty sand to sandy silt.	2	SS	10	
113.91	Tr. Clay.	3	SS	3	114
2.93	Stiff, grey silty Clay with 3 mm thick clayey silt varves.	4	SS	PM*	
	Thickness and frequency of varves increasing with depth.	5	SS	PM*	112
		6	SS	PM*	
		7	SS	PM*	110
					108
		8	SS	PM	
105.87		9	SS	4	106
10.97	Grey, silty sand. Some gravel. Occ boulder. Till				
104.65					
12.19	End of Borehole				104
					102
101.40					
15.44	End of penetration test				



GROUND WATER CONDITIONS | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Note
After completion of Penetration Test the rods were pulled back 0.9 m. A total of 16 blows were required to re-advance the rods.

PM* - Sample was taken from disturbed ground.

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 34-81-02 LOCATION CH 10 + 045.9 - 8.7 RT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 15.09 m. COMPILED BY I.C.

DATUM Geodetic DATE December 11, 1989 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.62	Ground Level						
0.00 / 115.40	Loose, brown sand. Some silt. Fill	1	SS	9		116	
1.22	Compact to loose silty fine sand. Tr clay. Silt and clay. Content increase with depth.	2	SS	12			0, 40,53,7
113.72		3	SS	4		114	
2.90		4	SS	PM*			
	Stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	5	ST	-		112	
		6	SS	PM*			
		7	ST	-		110	
		8	SS	PM	Seal		
		9	ST	-		108	
		10	SS	PM		106	
		11	SS	PM		104	
101.53						102	
15.09	Loose to compact, grey silty sand. Tr clay, some gravel. Occ boulder. Till	12	SS	3		100	
		13	SS	26			12, 44, 20,15
97.31		14	SS	18			
19.31	Fresh, grey, medium grained limestone bedrock with dark grey, closely spaced, dark grey partings (below 10mm) of shale 50mm fractured zone at - 20.2 m.	15	BQ				
95.79		16	BQ				
20.83	End of Borehole						

Shear strength plot (kPa; + field vane, numbers refer to sensitivity): +114 at approx. 113 m; 9, 9, 9, 12 at approx. 111–112 m; 9, 16 at approx. 109.5 m; 24, 31 at approx. 108.5 m; 6, 7 at approx. 107 m; 20, 13 at approx. 105.5 m; 9, 10 at approx. 102.5 m.

Unit weight: 98 (sample 14), 96.

Rec %	RQD %	Water Return
90	65	100
95	80	100

Notes

Water level in standpipe Piezometer at elevation 115.78 m on 22/12/89.

PM* - Sample taken from disturbed ground.

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

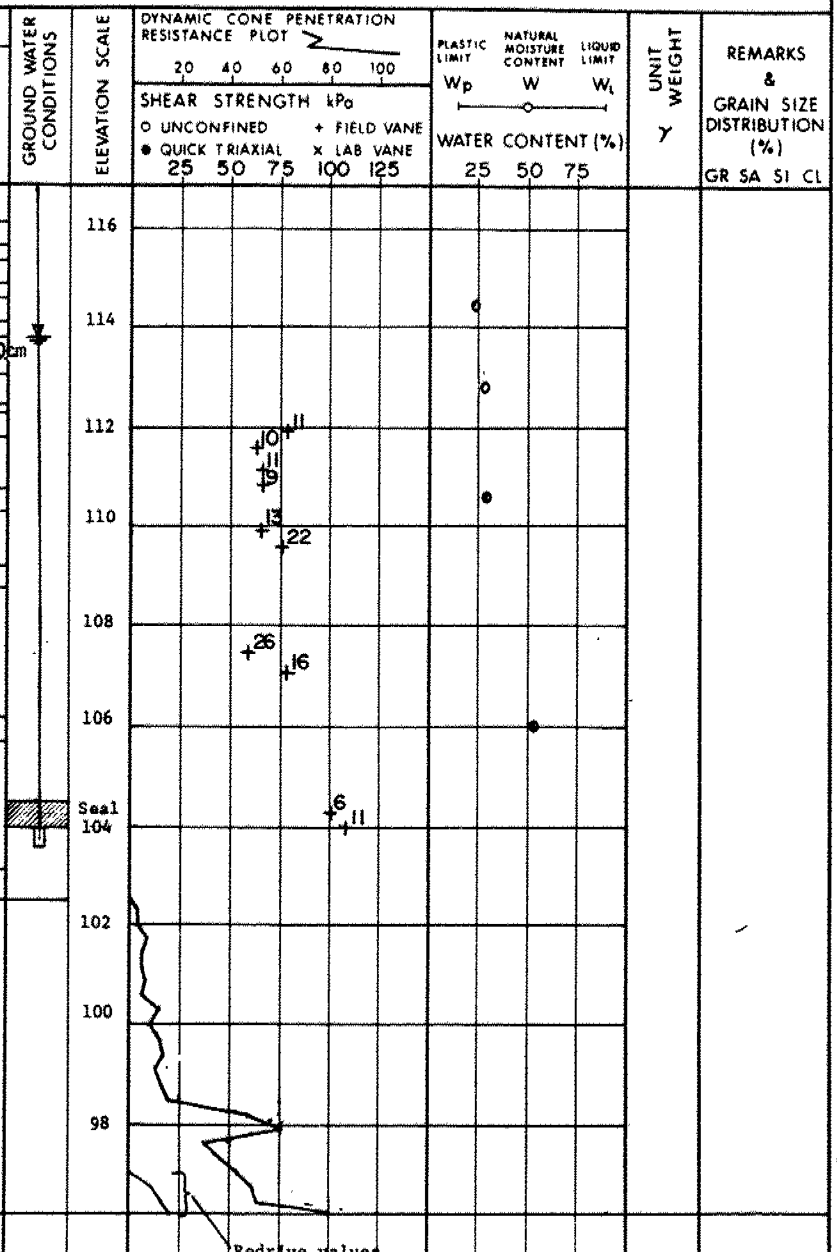
METRIC

W P 34-81-02 LOCATION CH 10 + 037.2 - 5.2 LT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger & Penetration Test COMPILED BY I.C.

DATUM Geodetic DATE December 13-14, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES
116.81	Ground Level			
0.00 / 115.59	Compact, brown sand Fill	1	SS	13
1.22 / 113.76	Compact to loose sandy Silt. Tr Clay. Silt and clay contents increase with depth.	2	SS	11
		3	SS	5
3.05 / 103.40	Stiff grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	4	SS	1/50cm
		5	ST	-
		6	SS	PM
		7	SS	PM
		8	SS	PM
		9	SS	PM
13.41 / 102.64	Loose, grey silty sand Some gravel.Till.	10	SS	5
14.17	End of Borehole			
96.16 / 20.65	End of penetration test			



Notes

Water level in stand-pipe at elevation 113.81 m on 24/01/90

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 34-81-02 LOCATION CH 10 + 000 (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 19.61 m COMPILED BY I.C.

DATUM Geodetic DATE December 8-11, 1989 CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES
118.17	Ground Level			
0.00	Dense to compact, brown sand. Tr gravel Fill	1	SS	48
116.04		2	SS	24
2.13	Compact to loose, grey sandy silt. Tr Clay. Silt and clay contents increase with depth.	3	SS	24
		4	SS	3
		5	SS	28
		6	SS	28
		7	SS	6
111.92 / 6.25	Very stiff becoming stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30mm spacings.	8	ST	-
		9	SS	PM
		10	SS	PM
		11	SS	PM
		12	ST	-
		13	SS	PM
		14	SS	PM
98.97 / 19.20	Grey silty sand. Some gravel. Occ. boulder. Till Boulder 19.61 m-20.11 m	15	BQ	-
96.83 / 21.34	End of Borehole Auger Refusal at 19.61m Started coring (BQ size) at this depth			

GROUND WATER CONDITIONS | ELEVATION SCALE (118, 116, 114, 112, 110, 108, 106, 104, 102, 100, 98)

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (25 50 75 100 125)

Field vane sensitivity values plotted: 8, 114, 14, 13, 15, 15, 20, 14, 16, 6, 4, 7, 16, 16

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (25 50 75)

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

OFFICE REPORT ON SOIL EXPLORATION

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

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4A

METRIC

W P 34-81-02 LOCATION CH 10 + 000 - 1.0 Rt (Hwy 44) ORIGINATED BY MK

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) 23.77 m COMPILED BY IC

DATUM Geodetic DATE January 24 & 25, 1990 CHECKED BY

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	Rec %	RQD %	Water Return %	UNIT WEIGHT	REMARKS
118.17	Ground Level											
0.00	Overburden material not sampled (For stratigraphy see Borehole No. 4)						118					
98.97 / 19.20	Probably, grey silty sand, some gravel. Occ. boulder. Till					Seal						
94.40 / 23.77	Fresh, sound, grey limestone bedrock with very closely spaced thin, (<1 mm) tight shale partings. Closely spaced shale bands (10-20 mm). Shale partings generally have irregular surface.		1	BQ		R1		95	61	100	94	
			2	BQ		R2		95	39	100	92	UCS @ 91.88-91.96 = 48 MPa
	Core breaks readily on shale partings		3	BQ		R3	90	92	34	100	90	
			4	BQ				0	0	100		
88.25 / 29.92			5	BQ		R4		87	24	100		

Note: Water level in piezometer not measured.

Elevation scale: 118, 116, 114, 112, 110, 108, 106, 104, 102, 100, Seal, 98, 96, 94, 92, 90

End of Borehole

$+^3, \times^5$: Numbers refer to Sensitivity

20 / 15 -◇- 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 34-81-02 LOCATION CH 9 + 962.8 - 5.0 RT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger COMPILED BY I.C.

DATUM Geodetic DATE December 4, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES
117.46	Ground Level			
0.00	Compact to loose brown sand. Occ organics. Fill.	1	SS	13
		2	SS	9
115.33 / 2.13	Compact, grey silt. Tr. sand and clay. Occ. shells	3	SS	17
		4	SS	16
		5	SS	10
113.19 / 4.27	Stiff to firm, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	6	SS	2
		7	SS	PM*
		8	SS	PM
		9	SS	PM
		10	SS	PM
		11	SS	PM
		12	SS	PM
		13	SS	PM
96.74 / 20.72	Loose, grey silty sand. Some gravel. Till	14	SS	7
95.67 / 21.79	End of Borehole			

Ground water conditions: Seal at elevation 114.

Elevation scale: 116, 114, 112, 110, 108, 106, 104, 102, 100, 98

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 25 50 75 100 125

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Note

Piezometer installed a short distance away from Borehole 5.

Water level in stand-pipe at elevation 116.06 m on 22/12/89.

PM* - Sample taken from disturbed ground.

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 6

METRIC

W P 34-81-02 LOCATION CH 9 + 954.1 ~ 9.0 LT (Hwy. 44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger COMPILED BY I.C.

DATUM Geodetic DATE December 5, 6, 1989 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.23	Ground Level					
0.00	Loose, brown sand. Tr Silt Fill	1	SS	10		
115.71 / 1.52	Compact to loose, grey sandy silt. Tr. clay. Occ. shells	2	SS	18		
		3	SS	7		
		4	ST	-	21.1	0, 54, 45, 1
113.57 / 3.66	Stiff, grey silty clay with 3 mm thick clayey silt varves at 25-30mm spacings.	5	SS	1/50 cm		
		6	SS			
		7	SS	PM		
		8	ST	-	18.4	
		9	SS	PM		
		10	SS	PM		
		11	ST	-	16.8	0, 2, 52, 46
		12	SS	PM		
		13	ST	-		
		14	SS	PM		
96.51 / 20.72	Compact, grey silty sand and gravel. Tr clay. Occ boulder. Till.	15	SS	12		
93.46 / 23.77	End of Borehole Auger refusal					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96, 94); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75.

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

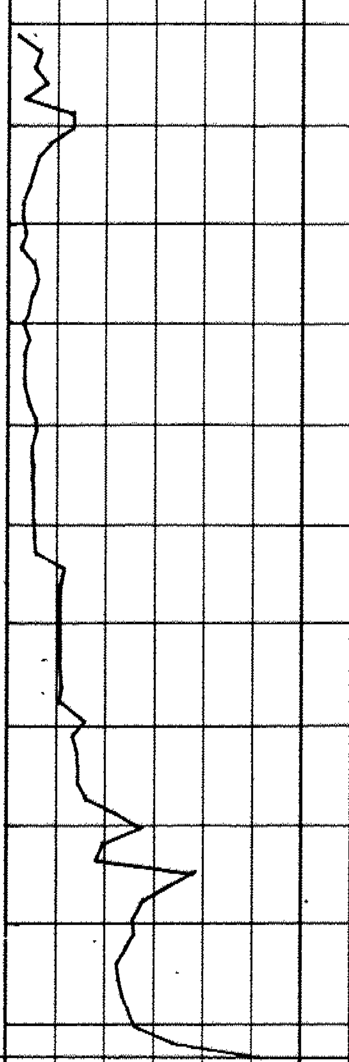
RECORD OF BOREHOLE No 6A

METRIC

W P 34-81-02
LOCATION CH 9 + 954.1 - 2.0 LT (Hwy. 44)
ORIGINATED BY RK

DIST 9 HWY 44
BOREHOLE TYPE Hollow Stem Auger, Penetration Test @ 1.5 m
COMPILED BY IC

DATUM Geodetic
DATE December 18 , 1989
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
117.23	Ground Level										
0.00	Overburden materials not sampled. For stratigraphy see Borehole 6						116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96				
95.03 / 22.20	End of Penetration Test							104/50 mm			

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 7

METRIC

W P 34-81-02 LOCATION CH 9 + 910 (Hwy.44) ORIGINATED BY R.K.

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger COMPILED BY I.C.

DATUM Geodetic DATE December 7, 1989 CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.94	Ground Level						
0.00	Loose, brown sand Tr silt. Fill	1	SS	9	116		
115.72 / 1.22	Compact, grey sandy silt Tr clay. Occ shells. Silt and clay contents increase with depth.	2	SS	21			
		3	SS	12	114		
		4	SS	12			
113.28 / 3.66	Stiff to firm, grey silty clay with 3 mm thick clayey silt varves at 25-30 mm spacings.	5	SS	PM*			
		6	ST	-	112	18.5	0, 10, 72, 18
		7	SS	PM*	110		
		8	ST	-	108		
		9	SS	PM			
		10	SS	PM	106		
103.99 / 12.95	End of Borehole PM* - Sample taken from disturbed ground.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 25 50 75 100 125

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75

Field vane sensitivity values plotted: 9, 4, 5, 3, 14, 8, 9, 7, 8, 9

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -○- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 8

METRIC

W P 34-81-02 LOCATION CH 9 + 957 - 5.0 Rt (Hwy 44) ORIGINATED BY MK

DIST 9 HWY 44 BOREHOLE TYPE Hollow Stem Auger, Rotary Coring (BQ) @ 23.37 m COMPILED BY IC

DATUM Geodetic DATE January 22, 23 and 24, 1990 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	Recovery %	RQD %	Water Return %	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
117.16 / 0.00	Ground Level											
	Overburden not sampled to 21.33 m. For stratigraphy see Borehole 5.					W.L.	116, 114, 112, 110, 108, 106, 104, 102, 100, 98, 96					
95.83 / 21.33	Compact to dense, grey silty sand. Some gravel, occ. boulders. Till		1	SS	38							
			2	SS	29						94	
93.79 / 23.37	Fresh, sound dark grey limestone bedrock with very closely spaced thin (<1 mm) shale partings. Core splits readily on shale partings. Parted surfaces generally very irregular		3	BQ				99	54	100		
											92	
			4	BQ				100	59	100		
90.49 / 26.67	End of Borehole											

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

$+^3$, $\times^5$: Numbers refer to Sensitivity

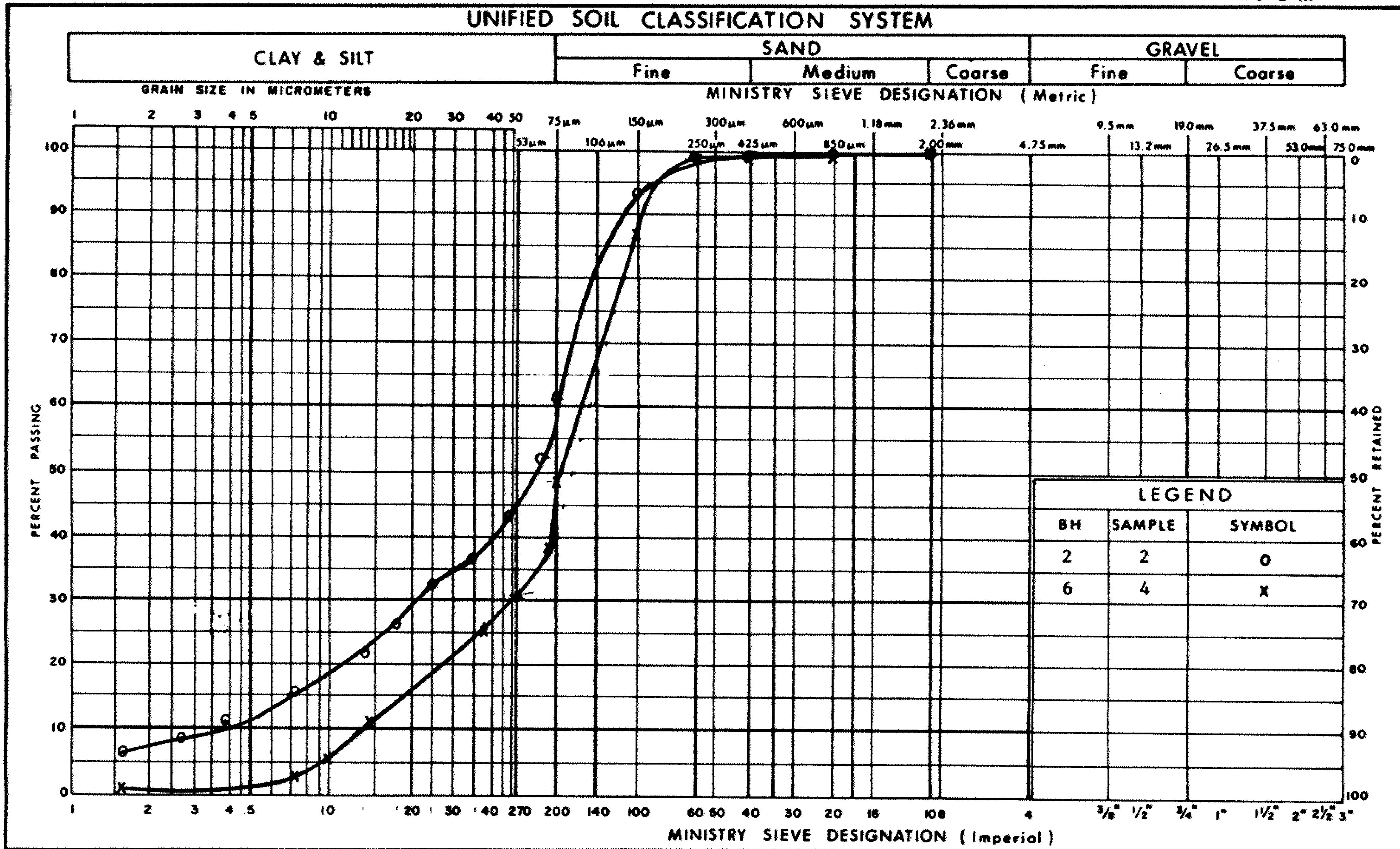
15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX II

Laboratory Test Data

78 12 M



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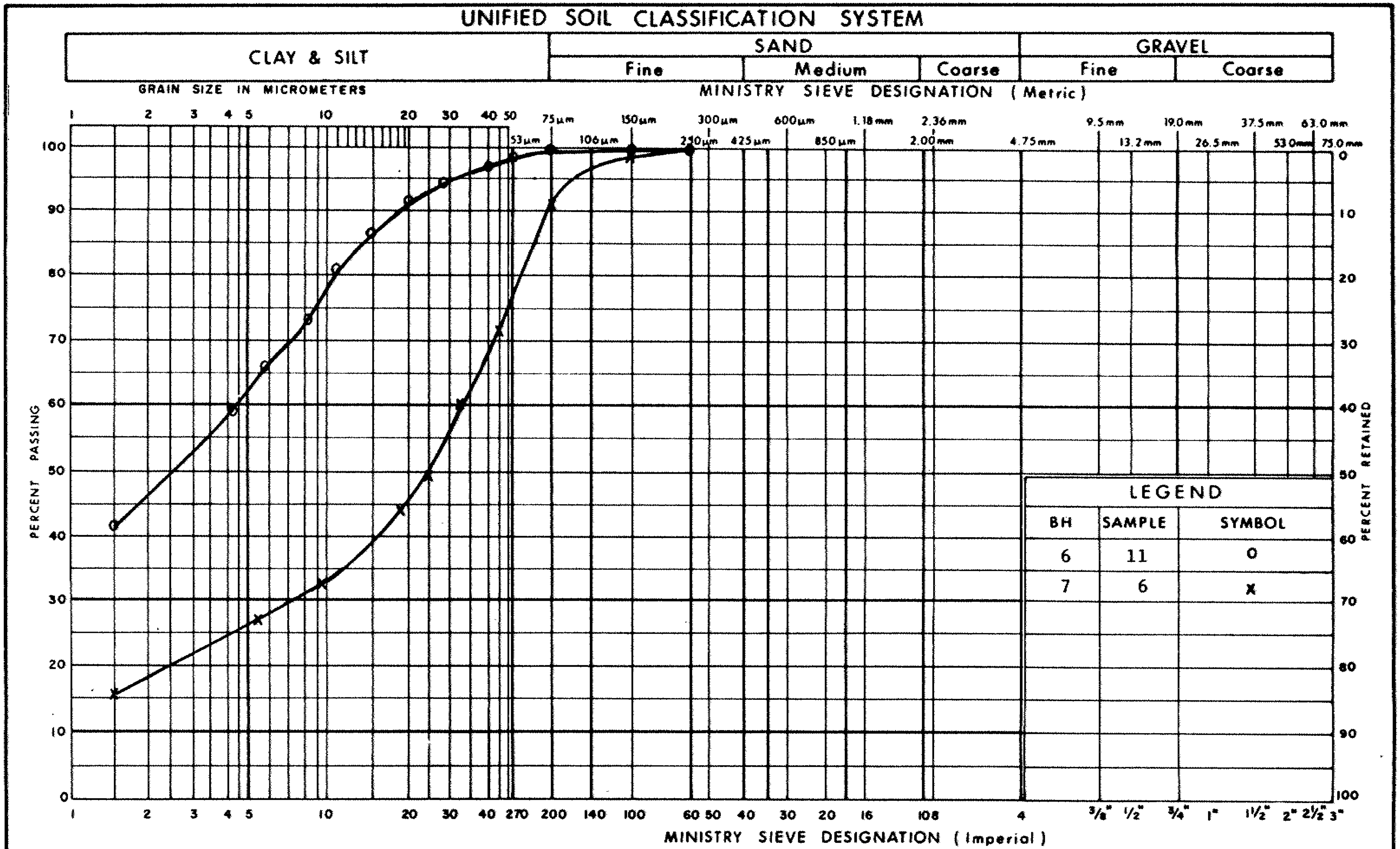
GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Silt, Trace Clay

FIG No 1

W P 34-81-02

78 12 M



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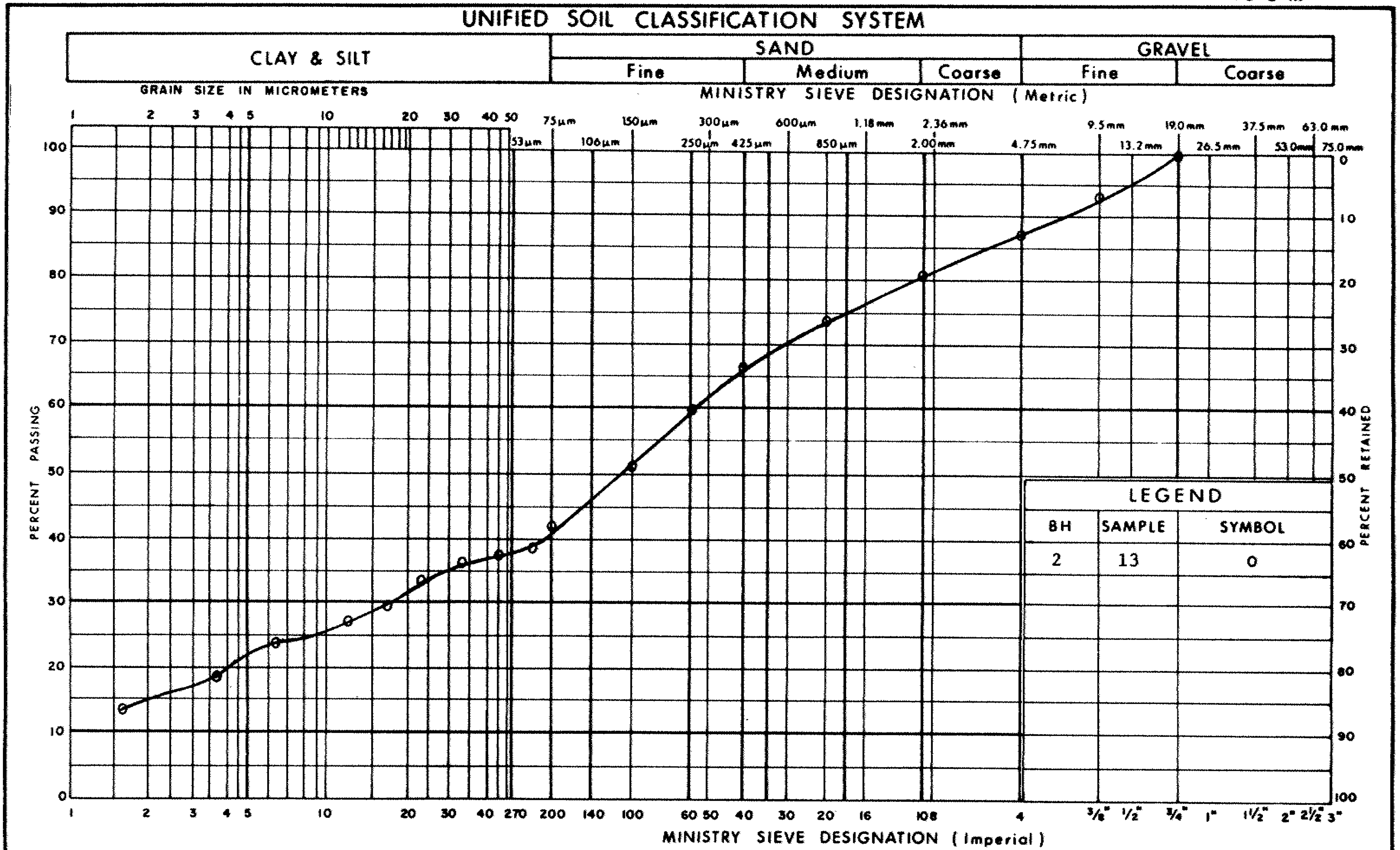
GRAIN SIZE DISTRIBUTION

Silty Clay

FIG No 2

W P 34-81-02

78 12 M



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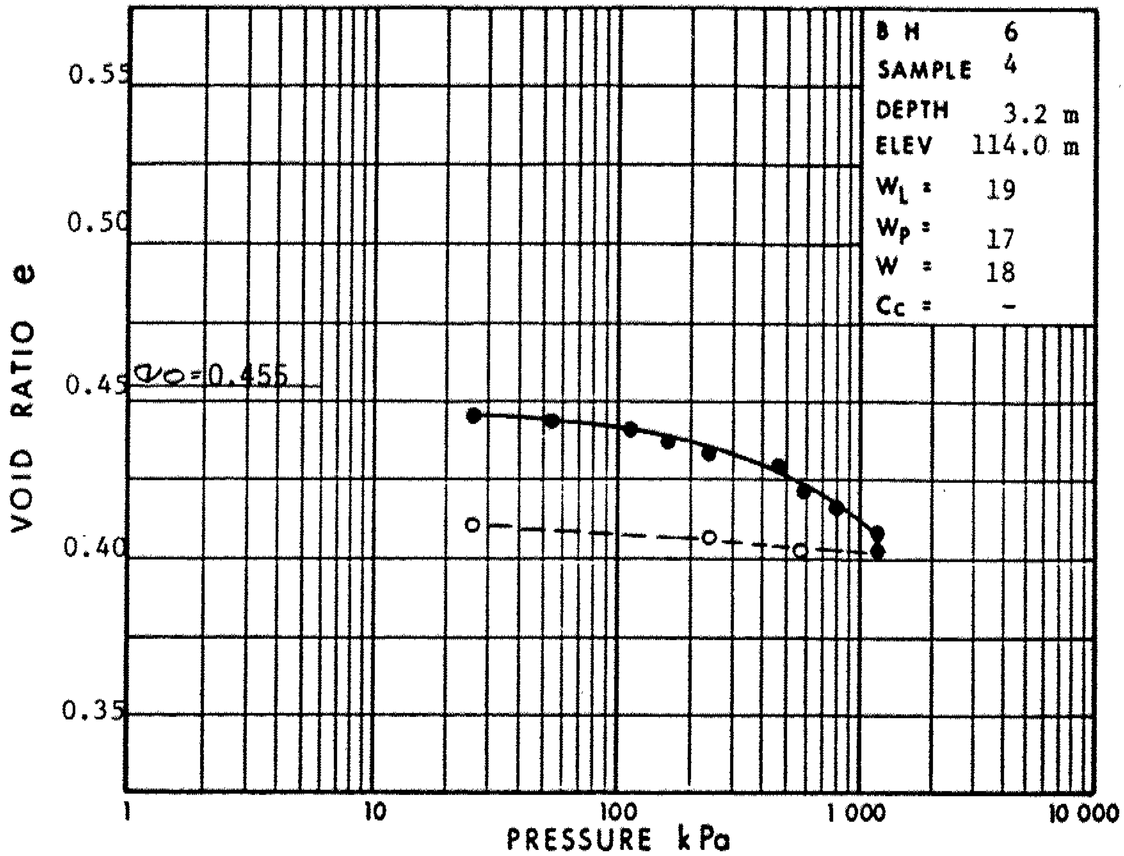
GRAIN SIZE DISTRIBUTION

Silty Sand, Some Gravel and Clay - TILL

FIG No 3

W P 34-81-02

VOID RATIO - PRESSURE CURVES



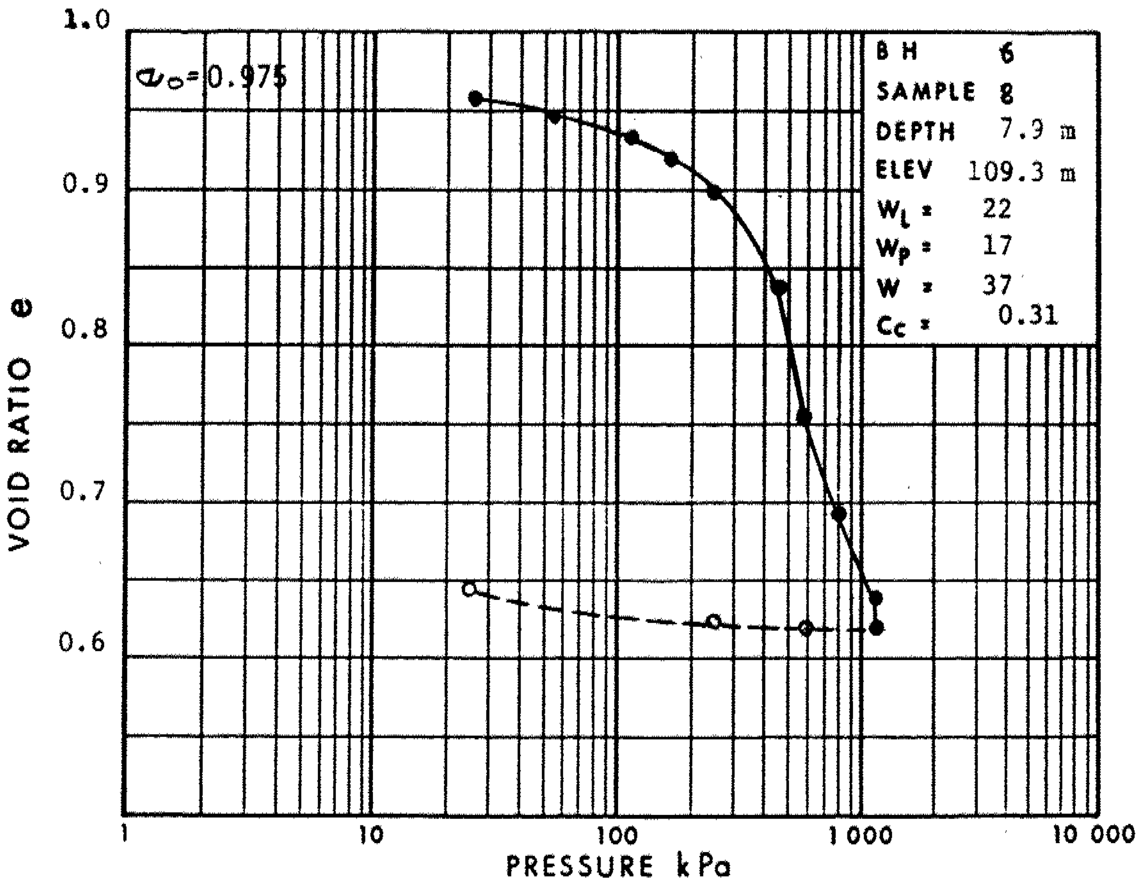
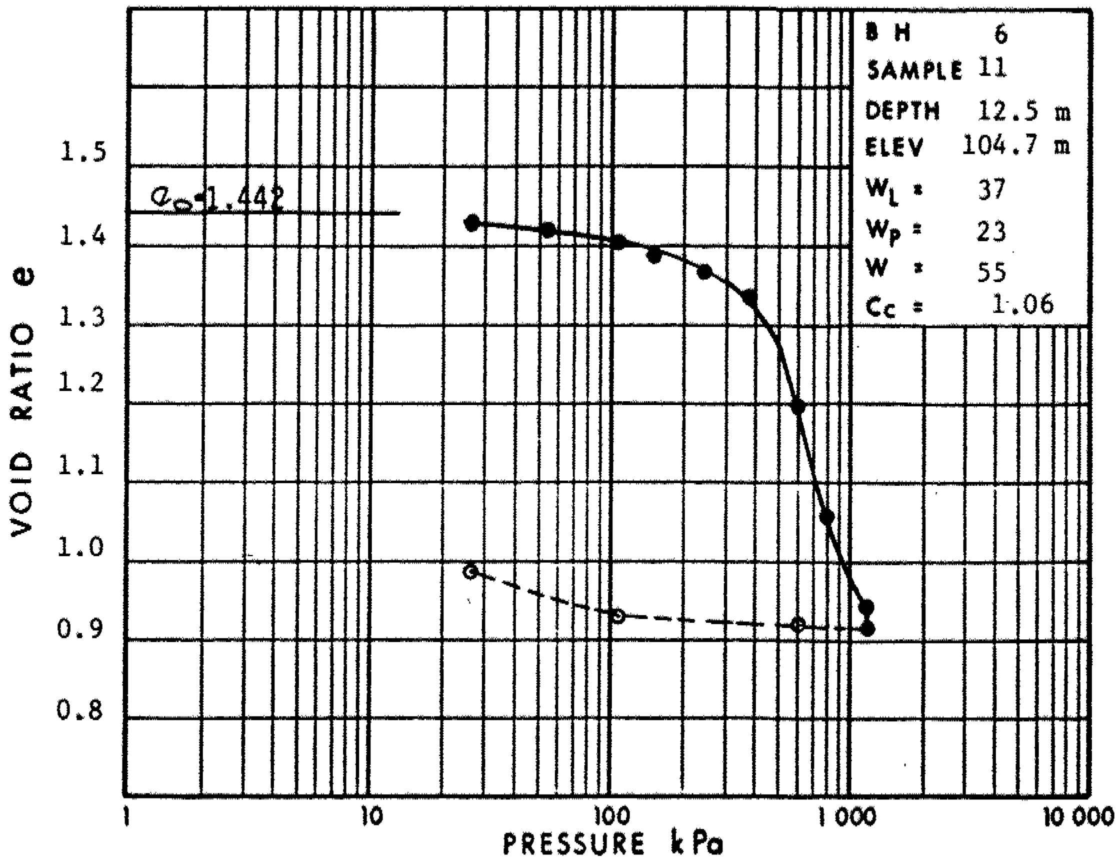
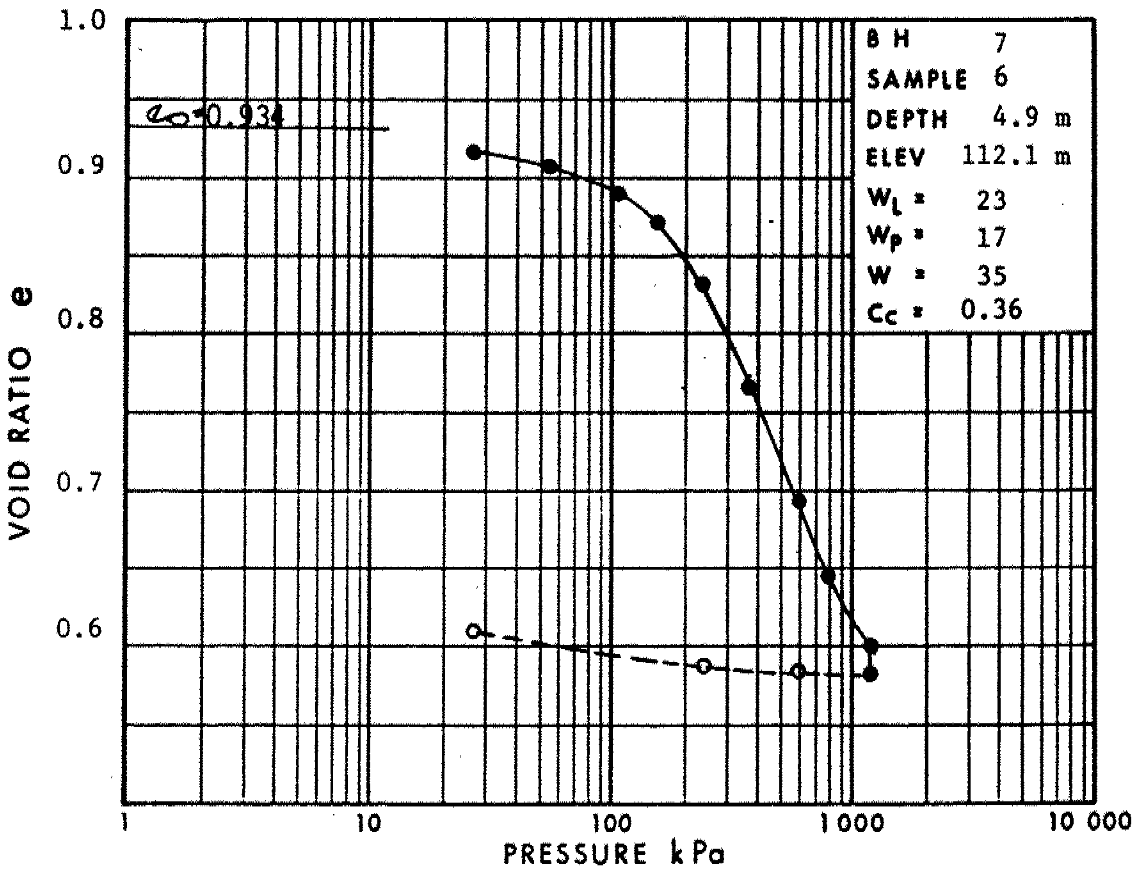


Fig No 4

WP 34-81-02

VOID RATIO - PRESSURE CURVES



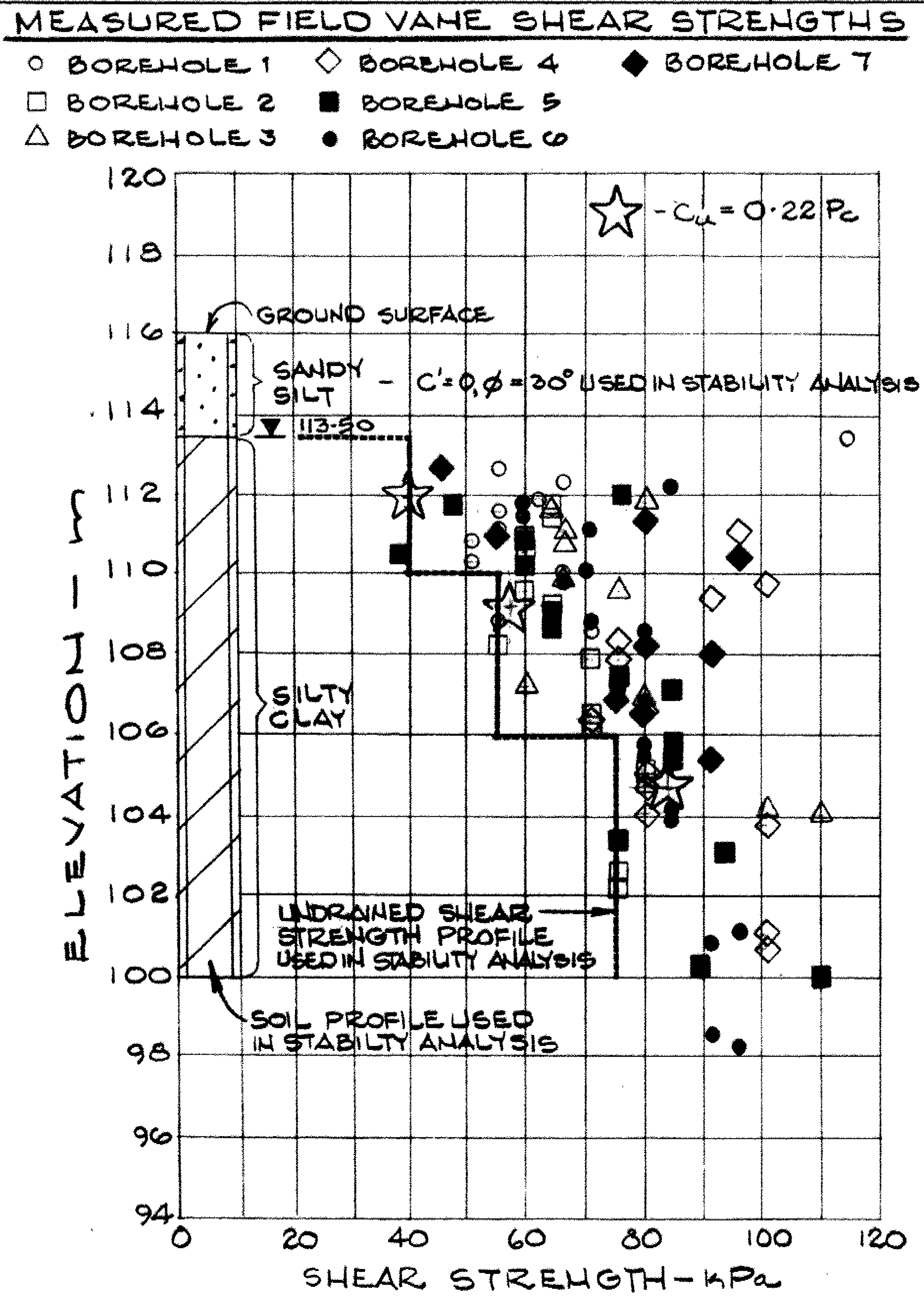


<u>APPENDIX III</u>

Embankment Stability Analysis Data

EMBANKMENT STABILITY ANALYSIS DESIGN SHEAR STRENGTH DATA

APPENDIX III
FIGURE
PROJECT T11600



NOTE
Pc = PRECONSOLIDATION PRESSURE

GEOCON

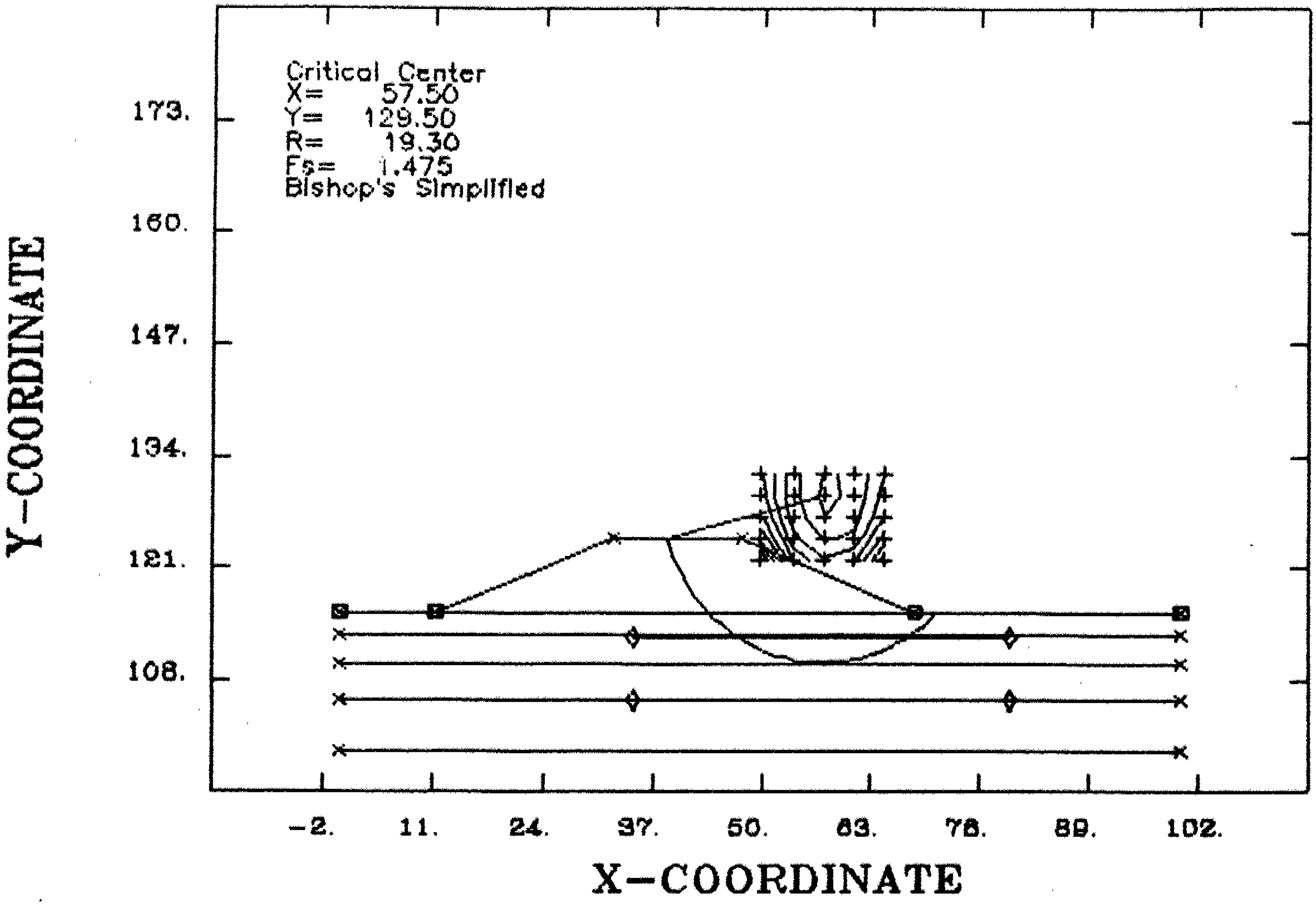
CROSS-SECTION OF GEOMETRY

HIGHWAY 44 AND 17 OVERPASS

NO. 2 Feb. 7, 1990

Slope stability analysis

SERIAL NO. 87128 is licensed to: GEOCON INC.



UNIT WEIGHT	COHESION	PHI	DESCRIPTION
20.00	.00	30.00	Granular fill
20.00	.00	30.00	granular
18.00	40.00	.00	clay
18.00	50.00	.00	clay
18.00	76.00	.00	clay
-1.00	.00	.00	hard bottom

File name : A:HIGHW2.SET

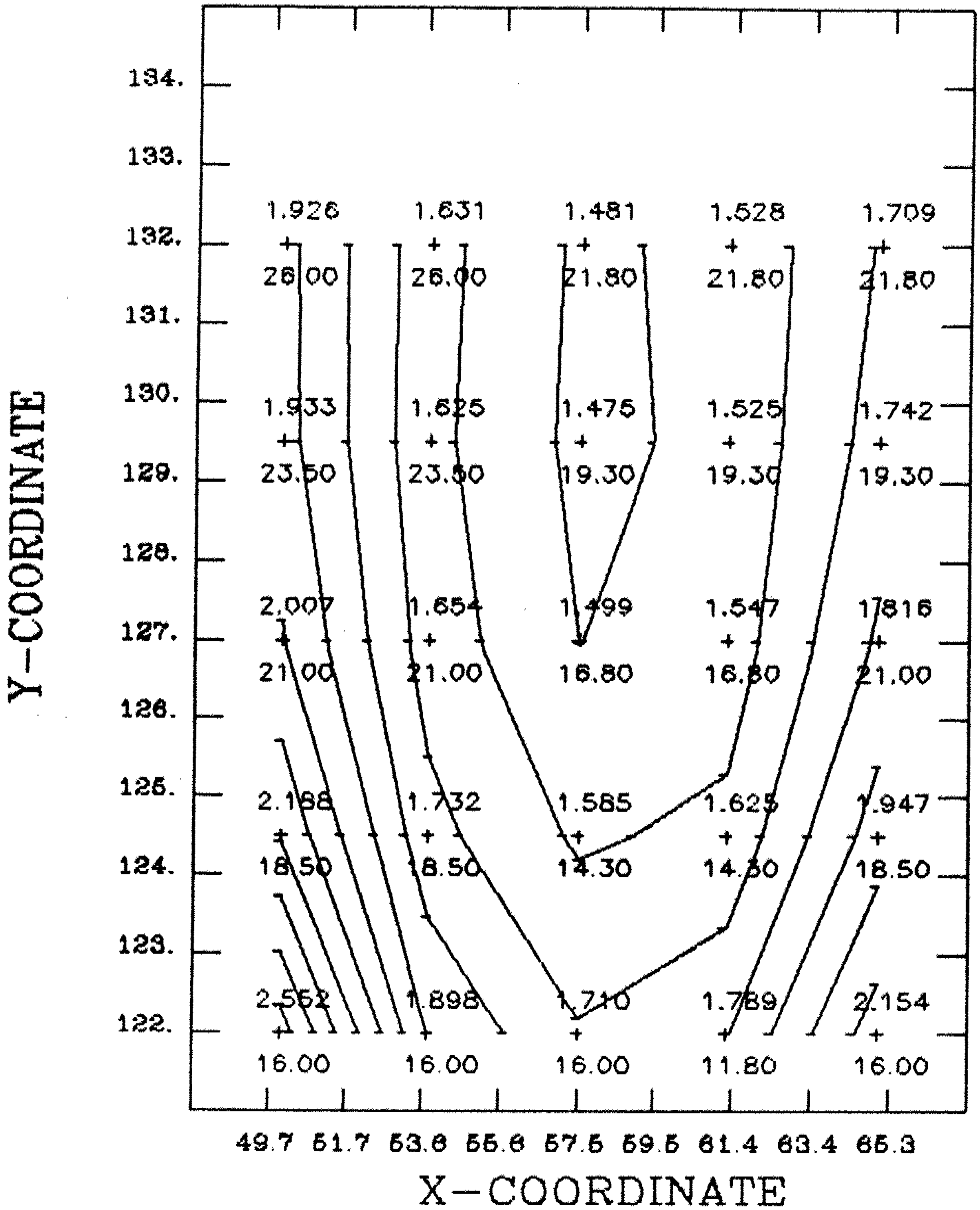
GRID OF FACTORS OF SAFETY

HIGHWAY 44 AND 17 OVERPASS

NO. 2 Feb. 7, 1990

Slope stability analysis

SERIAL NO. 87128 is licensed to: GEOCON INC.



BISHOP'S SIMPLIFIED METHOD

No. Above (+) is Factor of Safety

No. Below (+) is Slip Surface Radius

File name : A:HIGHW2.FAC

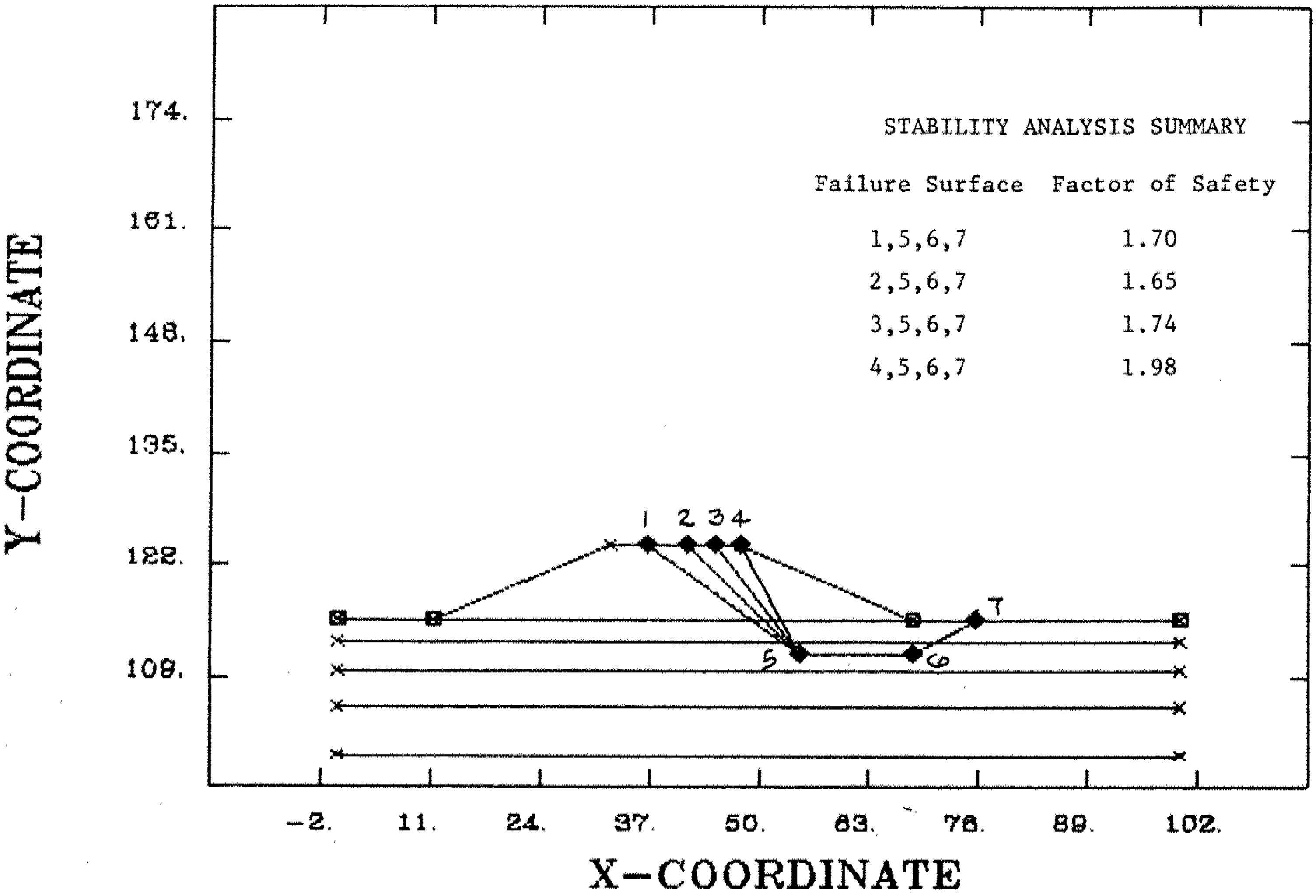
CROSS-SECTION OF GEOMETRY

HIGHWAY 44 AND 17 OVERPASS

NO. 4 Feb. 7, 1990

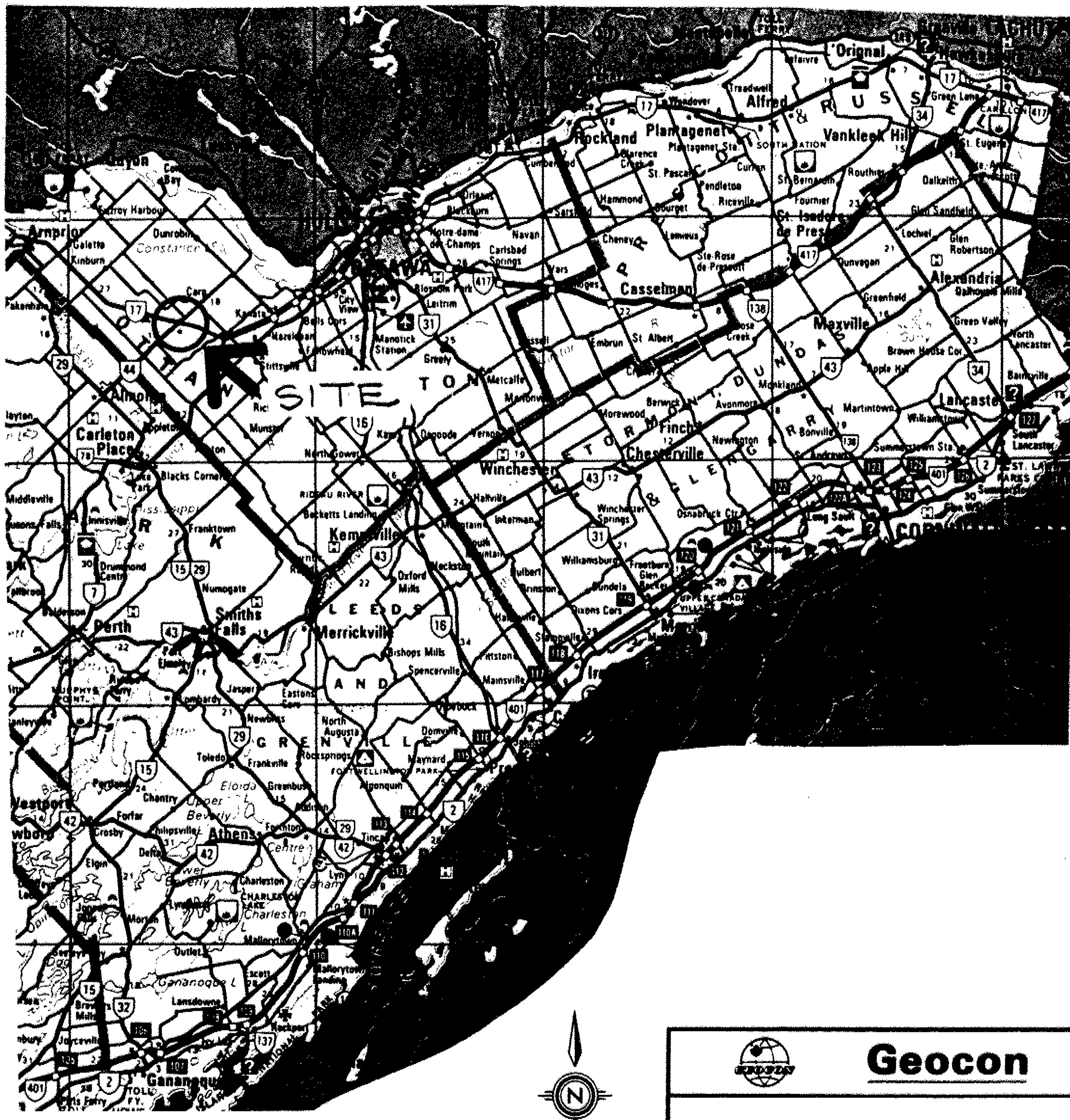
SLOPE STABILITY ANALYSIS - COMPOSITE SLIP SURFACE

SERIAL NO. 87126 is licensed to: GEOCON INC.



UNIT WEIGHT	COHESION	PHI	DESCRIPTION
20.00	.00	30.00	Granular fill
20.00	.00	30.00	granular
18.00	40.00	.00	clay
18.00	50.00	.00	clay
18.00	76.00	.00	clay
-1.00	.00	.00	hard bottom

File name : A:HIGHW4.SET



Geocon

SITE LOCATION PLAN

DATE	SCALE	
FEB. 8, 1990	H.T.S.	
DRAWN BY:	CHECKED BY:	APPROVED BY:
ADAPTED	I.C.	R.D.P.
PROJECT NO.	DRAWING NO.	REV.
15000	T11600-1	

OVERSIZE DRAWING

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

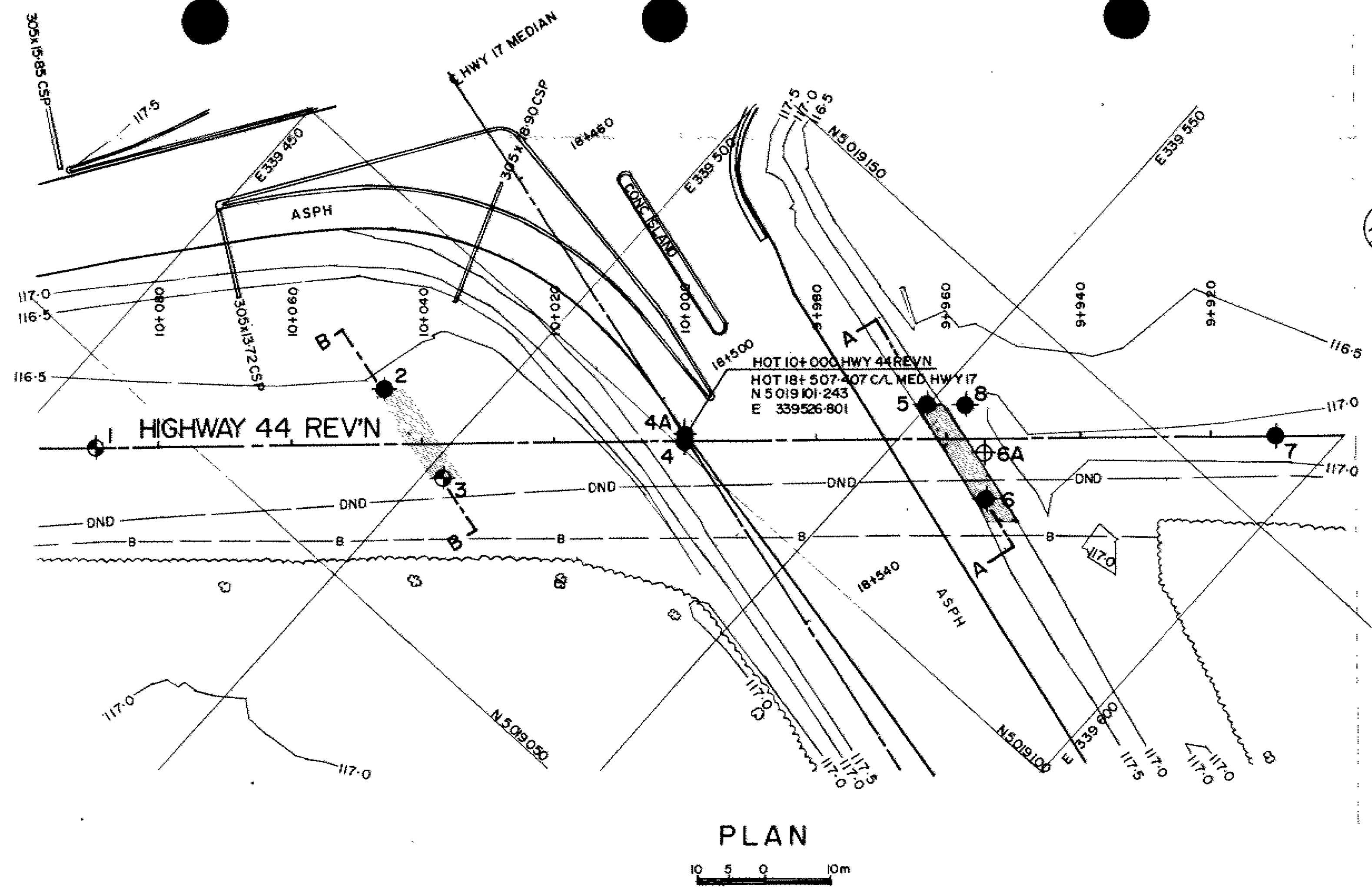
CONT No
WP No 34-81-02

HWYS 17 & 44 UNDERPASS

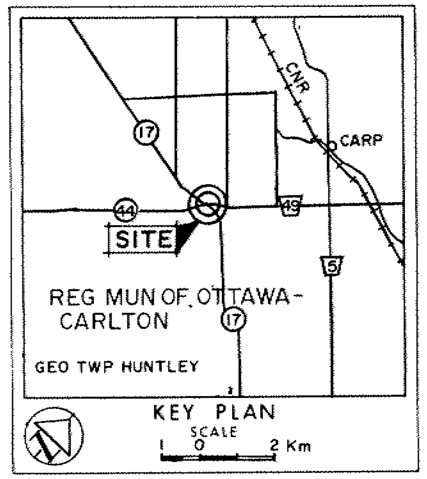
SHEET

BORE HOLE LOCATIONS & SOIL STRATA

GEOCON INC.



PLAN

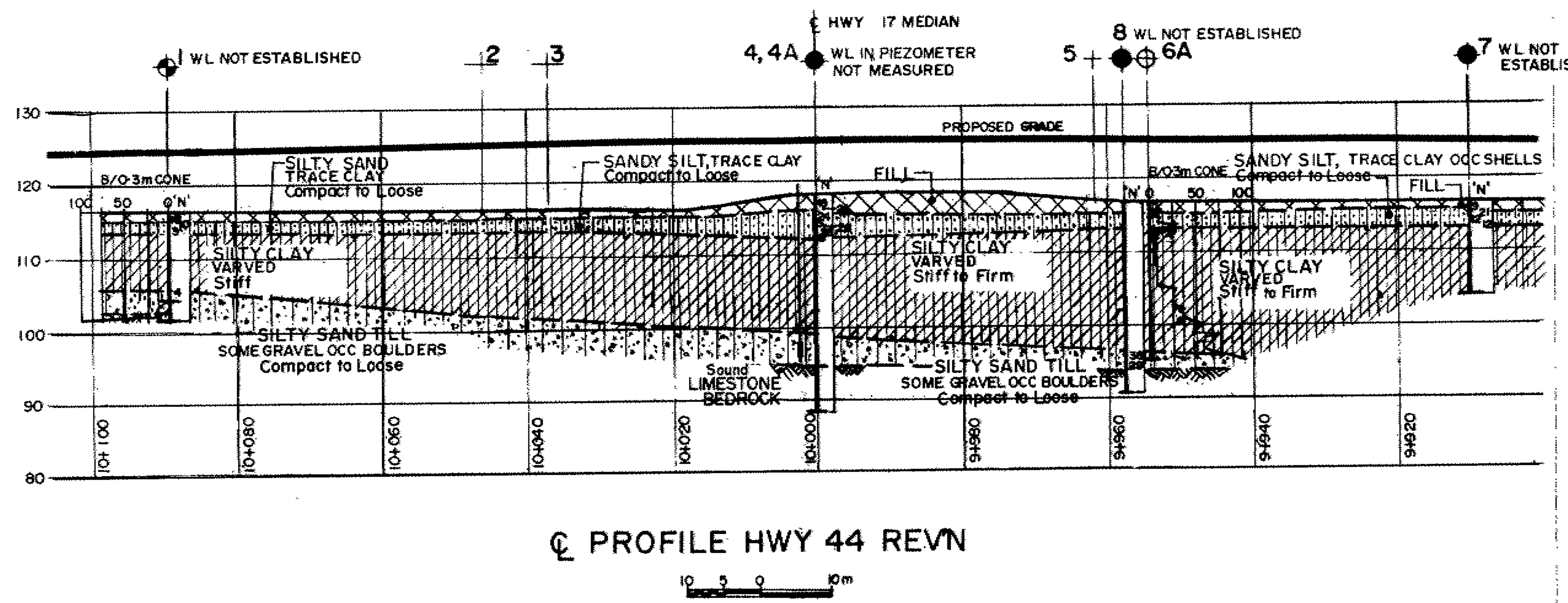


BOREHOLE No.	CHAINAGE	OFFSET
1	10+090·0	℄
2	10+045·9	8·7 RT
3	10+037·2	5·2 LT
4	10+000·0	℄
4A	10+000·0	1·0 RT
5	9+962·8	5·0 RT
6	9+954·1	9·0 LT
6A	9+954·1	2·0 LT
7	9+910·0	℄
8	9+957·0	5·0RT

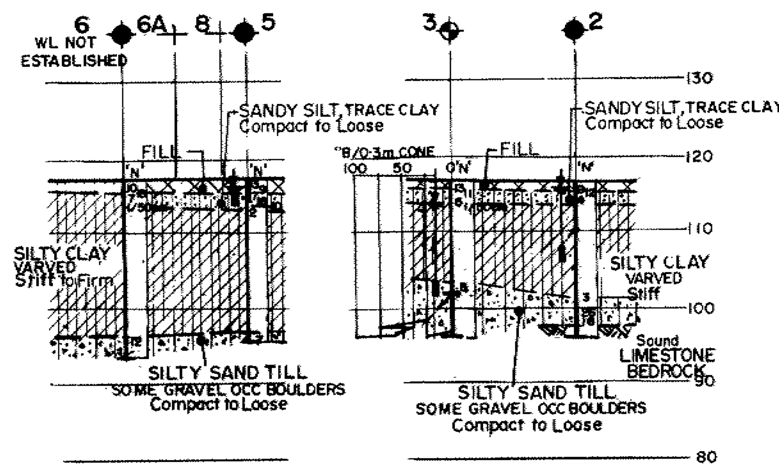
LEGEND

- 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)
- W L at time of investigation 1989 12
- PIEZOMETER

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	116·84	5019039·86	339 460·97
2	116·62	5019076·30	339 487·29
3	116·81	5019072·06	339 503·14
4	118·17	5019 101·24	339 526·80
4A	118·17	5019 101·97	339 526·11
5	117·46	5019 130·27	339 550·59
6	117·23	5019 125·96	339 566·50
6 A	117·23	5019 131·08	339 561·73
7	116·94	5019 162·62	339 592·62
8	117·16	5019 134·22	339 554·83



℄ PROFILE HWY 44 REVN



SECTIONS

A-A

B-B

≡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

REV. DATE	BY	DESCRIPTION

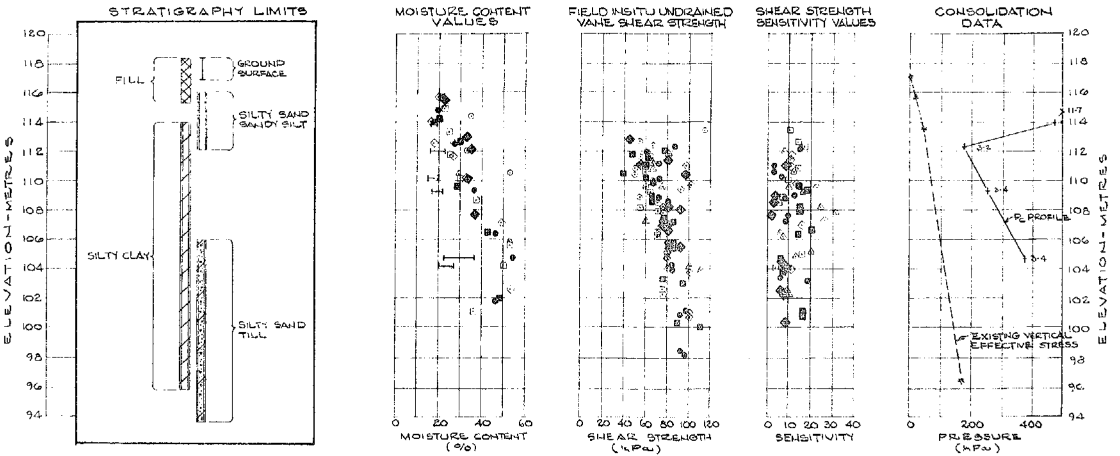
Geocres No 31F-110

HWY No 17 & 44 UNDERPASS			DIST 9
SUBM'D IC	CHECKED	DATE 1990 11 14	SITE 03-357
DRAWN MZ	CHECKED IC.	APPROVED RDP	DWG 348102-A

REF No E-65-17-1 1990 01

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

OVERSIZE DRAWING



LEGEND

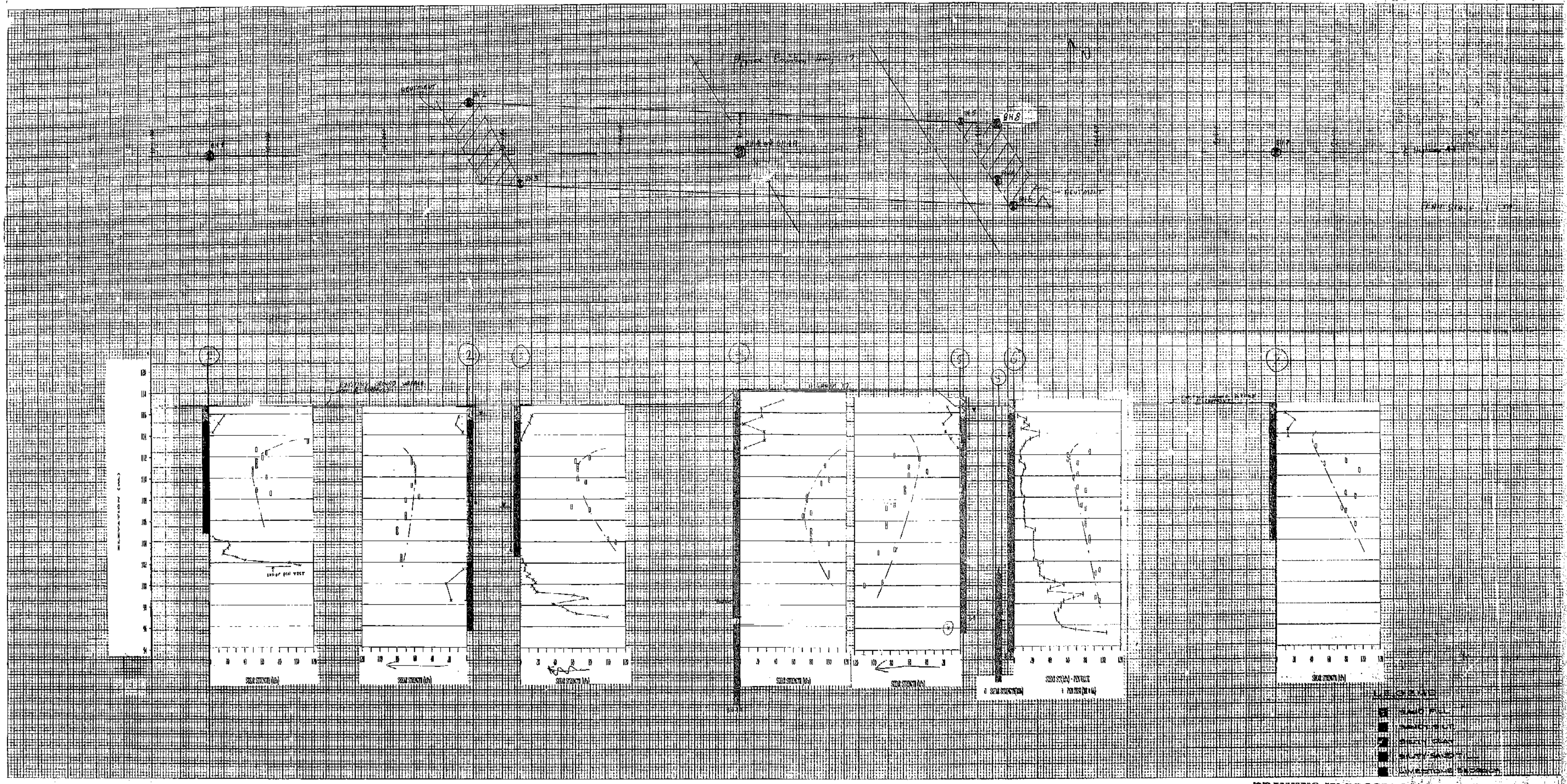
○ BOREHOLE 1
□ BOREHOLE 2
△ BOREHOLE 3
◇ BOREHOLE 4
■ BOREHOLE 5
● BOREHOLE 6
◆ BOREHOLE 7
├─┤ ATTERBERG LIMITS
Pc PRECONSOLIDATION PRESSURE
o3.2 OVER CONSOLIDATION RATIO

REVISIONS
REFERENCE

Geocon

GEOTECHNICAL DATA
SILTY CLAY LAYER

DATE FEB. 2, 1990	SCALE AS SHOWN	
DRAWN BY M.C.Z.	CHECKED BY R.D.P.	APPROVED BY R.D.P.
PROJECT NO. 115000	DRAWING NO. T11600-2	REV.



DRAWING IN PROGRESS

GEOCON INC.