

FOUNDATION INVESTIGATION REPORT

**W.P. 374-89-03, SITE 16-307
RAMP N-W OVER CEDAR GROVE ROAD
HWY. 401-416 INTERCHANGE
DISTRICT 9, OTTAWA
GEOCRES # 31B-75
MINISTRY OF TRANSPORTATION OF ONTARIO**

**SUBMITTED TO
DELCAN CORPORATION**

BY

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Report
on
Foundation Investigation
for
W.P. 374-89-03, Site 16-307
Ramp N-W Over Cedar Grove Road
Hwy. 401-416 Interchange
District 9, Ottawa

Jacques, Whitford Limited

March, 1992

Project No. 10211

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Drawing No. 3748903-A - Bore Hole Locations & Soil Strata

FOUNDATION INVESTIGATION REPORT

for

WP 374-89-03, Site 16-307
Ramp N-W Over Cedar Grove Road
Hwy. 401-416 Interchange
District 9, Ottawa

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out at the above noted site in the Township of Edwardsburg, Ontario. The investigation was carried out in accordance with our proposal dated December 11, 1990. Authorization to carry out the work was provided by Mr. H.R. Luck, P.Eng., of Delcan Corporation.

This report contains factual information obtained from this investigation pertaining to the subsurface conditions.

2.0 SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Cedar Grove Road about 100 metres north of Highway 401 and about 1.2 kilometres west of the existing intersection of Highways 16 and 401. The topography of the site is generally sloping downward from the northeast to the southwest at about 2% grade or less. The surrounding ground consists of pasture and grassed areas.

The existing Cedar Grove Road is a two-lane paved road with gravel shoulders. Cedar Grove Road at this location is in general about 0.8 m above the surrounding ground.

Drainage of the site is provided by highway ditches connected to Johnstown Creek which flows into the St. Lawrence River.

Physiographically, the site lies in the area known as the Glengarry Till Plain. The surface consists of morainic ridges and drumlins together with intervening clay flats and swamps. Bedrock underlying the overburden consists of Ordovician dolomitic limestone of the Oxford Formation. Overburden thickness in this area is less than 10 m.



3.0 PROCEDURE

3.1 Field Investigation

Prior to the onset of the drilling investigation, the necessary utility check clearances were obtained by our site personnel.

The field work for this investigation was carried out between May 3 and 6, 1991. A total of four (4) boreholes, (numbered 91-1 to 91-4) were put down at the site. Boreholes 91-1 and 91-4 were put down at the approach fill locations. Boreholes 91-2 and 91-3 were put down at the abutment locations. The test locations are indicated on Drawing No. 3748903-A provided in Appendix 2.

All boreholes were put down using a track-mounted power auger drill suitably equipped for soil and bedrock sampling. Hollow stem augers and N-sized rock coring techniques were employed during the course of the investigation to advance the boreholes.

The boreholes were put down to depths ranging from 7.9 m to 10.7 m. Boreholes 91-2 and 91-3 were terminated after coring in NQ-size 2.8 m and 1.8 m into bedrock respectively. Boreholes 91-1 and 91-4 were terminated at depths of 8.2 m and 7.9 m respectively, upon hollow stem auger refusal.

The overburden soils encountered were sampled by means of a split tube sampler during the performance of Standard Penetration Tests (SPT) (ASTM D1586). Where cohesive soils were encountered, field vane tests were carried out and thin-walled (Shelby) tube samples were collected at selected locations. Sampling and in situ testing were conducted on a near continuous basis (intervals of 0.76 m).

All soil samples recovered were stored in moisture-proof bags and were returned to our Ottawa laboratory together with rock cores for detailed classification and testing.



Standpipe piezometers 25 mm in diameter were installed in Boreholes 91-1, 91-2 and 91-4 between depths of 4.8 m and 8.2 m. A monitoring well 58 mm in diameter was installed in Borehole 91-3 at a depth of 8.4 m.

The standpipes and the monitoring well were backfilled with sand within the perforated lengths. A bentonite seal was then placed in the boreholes prior to backfilling with soil cuttings to near the ground surface. A bentonite surface seal was then provided and the ground surface was mounded to prevent water infiltration. A groundwater sample was collected from the monitor well and was subjected to chemical testing.

3.2 Survey

The borehole locations and ground surface elevations were surveyed by Delcan Corporation personnel after completion of the field work. The elevations are referenced to Geodetic datum. The borehole coordinates and elevation data is summarized on Drawing 3748903-A in Appendix 2.

3.3 Laboratory Testing

To identify the behaviour and properties of the soil samples collected during the field investigation, the following laboratory tests were carried out:

- Detailed visual classification,
- Natural moisture content,
- Sieve and hydrometer analyses,
- Atterberg Limits determination,
- Consolidation Test.

Samples remaining after testing will be stored in our laboratory for a period of six months after issuance of the final report. They will then be discarded unless we are directed otherwise.

4.0 RESULTS OF THE INVESTIGATION

4.1 Subsurface Conditions

The subsurface conditions observed in the boreholes are presented in detail on the Record of Boreholes provided in Appendix 1. An Explanation of Terms Used in Report is also provided in Appendix 1. The laboratory test results are summarized in the Record of Boreholes and also on Figures 1 to 5 in Appendix 1.

The ground surface elevations at the borehole locations varied from El. 85.7 m to 87.6 m at the time of the investigation. The subsurface materials at the boreholes consist of topsoil, overlying sand (where present), overlying silty clay, overlying a heterogeneous mixture of sandy silt, some clay and gravel, occasional boulders (glacial till), underlain by a dolomitic limestone bedrock. The bedrock surface was encountered between El. 78.0 m and El. 78.4 m at the two (2) boreholes which were cored. The groundwater level was observed between El. 85.7 m to 85.9 m (at ground surface to 1.8 m below ground surface).

A brief discussion of the observed subsurface conditions is provided below. Specific details of the subsurface materials should be obtained from the Record of Boreholes.

4.1.1 Topsoil

A surficial layer of topsoil was encountered in all boreholes. The thickness of the topsoil ranges from 150 mm to 200 mm.

4.1.2 Sand

Sand, trace silt was encountered underlying the topsoil in Boreholes 91-1, 91-2 and 91-4. The thickness of the sand ranges from 100 mm to 200 mm. Based on visual identification, the sand is classified as a cohesionless material.

4.1.3 Silty Clay

Silty Clay was encountered underlying the sand in Boreholes 91-1, 91-2 and 91-4 and underlying the topsoil in Borehole 91-3. The thickness of the silty clay ranges from 3.1 m to 6.1 m. The silty clay consists of an upper oxidized brown to grey, mottled layer underlain by a grey layer.

Field vanes tests conducted in the silty clay layer indicated that the undrained shear strength ranges from 139 kPa to over 235 kPa, with a sensitivity of 2 to 5. These undrained shear strengths indicate that the silty clay has a consistency of very stiff to hard. The results of laboratory testing are provided on the Record of Boreholes, on Figures 1 and 2 in Appendix 1, and are summarized below:

Property	Range	# Tests	Average
Moisture Content (%)	22 - 39	15	31
Liquid Limit (%)	50-51	2	50
Plastic Limit (%)	17-21	2	19
Plasticity Index (%)	30-33	2	31
Grain Size			
% Sand	1	2	1
% Silt	33-40	2	36
% Clay	59-66	2	63

A consolidation test was carried out on a representative silty clay sample from Borehole 91-3. The test results are presented in Figure 3 in Appendix 1, and are summarized below:

Initial Void Ratio (e_0)	0.86
Preconsolidation Pressure (P_c), kPa	380
Compression Index (C_c)	0.211
Recompression Index (C_r)	0.025

Based on visual identification and laboratory tests, the silty clay is classified as a cohesive material with intermediate to high plasticity.

4.1.4 Heterogeneous Mixture of Sandy Silt, some Clay and Gravel, occasional Boulders (Glacial Till)

A heterogeneous mixture of sandy silt, some clay and gravel, occasional boulders (glacial till) was encountered underlying the silty clay in all boreholes. The glacial till surface was encountered at elevations ranging from El. 79.6 m and El. 84.2 m (depths of 3.4 m to 6.2 m). The thickness of this stratum ranges from 1.8 m to over 4.8 m.

The SPT conducted in the glacial till layer yielded N values ranging from 7 to 69, indicating a denseness of loose to very dense, and generally in the compact to dense range. The results of laboratory testing are provided on the Record of Boreholes, on Figures 4 and 5 in Appendix 1, and are summarized below:

Property	Range	# Tests	Average
Moisture Content (%)	7 - 12	9	10
Liquid Limit (%)	13-17	2	15
Plastic Limit (%)	11-14	2	13
Plasticity Index (%)	2-3	2	2
Grain Size			
% Gravel	7-11	2	9
% Sand	35-40	2	38
% Silt and Clay	49-58	2	53

The above grain-size distributions represent only the minus 38 mm fraction (split spoon samples) of the glacial till. Cobbles and boulders are also present in this material. If the coarser portion is to be included, the actual percentage of fines would be less than that indicated above. Based on the above tests and visual identification, this till is classified as a cohesionless material.

4.1.5 Bedrock

Bedrock was encountered and proven by coring in NQ-size in Boreholes 91-2 and 91-3. The bedrock surface at Boreholes 91-2 and 91-3 were encountered at El. 78.4 m and El. 78.0 m (depths of 7.9 m and 8.4 m), respectively. The bedrock is a grey, unweathered, dolomitic limestone with close to moderately close spaced horizontal fractures. The bedrock is of fair to excellent quality (RQD ranging from 53% to 97%). Core recoveries varied between 98% and 100%.

In Boreholes 91-1 and 91-4, hollow stem auger refusal was encountered at El. 79.4 m and El. 77.8 m (depths of 8.2 m and 7.9 m), respectively. At borehole locations where bedrock coring was not carried out, it could not be determined whether auger refusal was encountered in glacial till or on bedrock.

4.2 Groundwater

Groundwater levels were recorded during drilling, in standpipe piezometers and monitor wells after drilling. The groundwater levels recorded ranged between El. 85.7 m to El. 85.9 m (from ground surface to a depth of 1.8 m).

Groundwater levels are subject to seasonal fluctuations and can vary from the values given in this report.



5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Proposed Development

The site is located at the existing Cedar Grove Road about 100 metres north of Highway 401 and about 1.2 km west of the existing intersection of Highways 16 and 401 in the Township of Edwardsburg, Ontario (refer to the Key Plan provided on Drawing No. 3748903-A in Appendix 2).

The proposed bridge structure is part of the Highway 416 development. The Cedar Grove Road Overpass is to span over Cedar Grove Road and is part of the proposed Ramp N-W (Highway 416 SBL). The Cedar Grove Road structure is to consist of the following components:

- A two-lane single-span fly-over structure spanning over Cedar Grove Road.
- The structure will be supported by two (2) abutments with associated approach fills.
- The approach fills are proposed to have forward slopes of 1.5 horizontal to 1 vertical. Side slopes at some locations may also be 1.5H:1V due to proximity to existing features.
- Fill heights at abutment locations are to range from approximately 7.6 m to 8.2 m.

5.2 Geotechnical Assessment

The subsurface soils profile at the site consists of silty clay overlying glacial till, underlain by dolomitic limestone bedrock.

The design dictates that the approach fills of up to 8.2 m in height will be constructed using forward slopes of 1.5H:1V, and side slopes at some locations at the same slope angle. For slopes of 1.5H:1V, granular fill material with internal slope reinforcement is recommended. For side slopes where the design does not dictate 1.5H:1V slopes be used, a 2H:1V slope would be appropriate for granular borrow.

Settlement of the embankment and the underlying soil is expected to be in the order of 60 mm. It is recommended that the approach fills be allowed to settle for a minimum period of two (2) months of pre-loading period prior to construction of the structure foundations.

At the end of the pre-loading period, the abutments may be supported on spread footings perched within compacted Granular 'A' fill, or on driven piles end bearing on bedrock. The presence of the silty clay limits the bearing capacities for the spread footings system. If the design can accommodate the rather lower bearing capacities, the spread footings system would be the preferred alternative due to cost.

This report contains our detailed recommendations in the following areas:

- 1) Approach Fill Recommendations
- 2) Structure Foundations
- 3) Abutment Backfill
- 4) Construction Considerations

5.3 Approach Fill Recommendations

5.3.1 Subgrade Preparation

All organic materials should be stripped and removed prior to fill placement within the entire fill area. Based on the approach fill boreholes (Boreholes 91-1 and 91-4), the anticipated depth of stripping is approximately 200 mm.

The exposed surface should be proof rolled and soft areas removed prior to fill placement. The fills should be placed and compacted in accordance with OPSS 212 and 501.

5.3.2 Stability

Fill placement of up to 8.2 metres is proposed for the approach fills within the investigated area with forward slopes and side slopes at some locations of 1.5H:1V. A slope stability analysis utilizing total stresses was performed for the fills in both the longitudinal and transverse directions. No deep-seated failures are anticipated through the foundation soils. However, the results indicate that for the proposed forward and side slopes of 1.5H:1V, granular material with internal reinforcements such as Geogrids¹ will be required. The Geogrid supplier should confirm

¹Geogrid is a registered trademark of the Tensar Corporation

that pile installation can be carried out through the reinforced embankment fill. Alternatively for the forward slopes and within piling area underneath the abutments, the fill may consist of compacted OPSS Granular 'A' material.

For side slopes where the design does not dictate 1.5H:1V slopes be used, a 2H:1V slope would be appropriate for granular borrow such as OPSS Select Subgrade Material.

To protect against surficial instability, normal slope vegetation should be established in accordance with MTO standards as soon as possible after construction. Where slope angle of 1.5H:1V is used, surficial erosion control using commercially available products such as straw blanket is recommended to ensure vegetation growth. Other landscaping alternatives can also be considered.

5.3.3 Settlement

Settlement of the approach fills under their own weight and from consolidation of the native soil beneath can be anticipated. Based on a 8.2 m fill height, settlement is estimated to be in the order of 60 mm. This settlement is time-dependent and consequently, it is recommended that the approach fills be allowed to settle for a minimum pre-loading period of two (2) months prior to construction of the structure foundations. A settlement monitoring program is recommended.

An existing oil pipeline is located within the proposed north approach fill area. If the pipeline is not to be relocated, it will undergo settlement due to the fill placement. Details of the pipeline are not available at this time. Assuming that the pipeline is buried 1.5 m below the ground surface, preliminary calculations indicate that it will settle 40-50 mm under the centreline of the fill, and 10-15 mm under the toe of the fill. Details of the pipeline should be obtained and its settlement tolerance should be investigated.

5.4 Structure Foundations

5.4.1 Perched Abutments on Compacted Granular 'A' Fill

The abutments may be founded on spread footings perched within compacted Granular 'A' fills in accordance with the details shown on Figure 6 in Appendix 1. As outlined in Section 5.3.2, it is recommended that the approach fill be allowed to settle for a minimum of 2 months prior to construction of the abutments.

At the end of the pre-loading period, it will be necessary to partially excavate the fill to construct the abutment. The load bearing portion of the fill must be constructed in accordance with the details shown on Figure 6. Subgrade preparation should be as outlined in Section 5.3.1.

Spread footings placed on granular pads constructed as recommended above may be designed based on the following bearing pressures:

	<u>Footing Elevation (m)*</u>	<u>Factored Bearing Capacity at U.L.S.</u>	<u>Bearing Capacity at S.L.S. Type II</u>
South Abutment	88.7	375 kPa	175 kPa
North Abutment	89.0	375 kPa	175 kPa

* Approximate elevations of the underside of the abutment footings as indicated by Delcan Corporation.

The presence of the silty clay layer is reflected in the lower bearing capacities recommended. The above bearing capacities have been calculated based on a footing width (B) of 5 m. The S.L.S. Type II bearing pressure has been calculated assuming that a total settlement of 25 mm is satisfactory. This settlement is additional to the 60 mm settlement expected during the pre-loading period due to the approach fill placement.

A minimum earth cover of 1.8 m over the footings should be provided for frost protection purposes and has been assumed in the calculations.

Sliding resistance between concrete footings and Granular 'A' should be calculated in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. using an unfactored friction coefficient of 0.7.

Only minor dewatering (if any) will be required for this alternative.

5.4.2 End Bearing Driven Piles

As an alternative to the perched abutment on compacted fill system, the abutments may be supported on end-bearing steel H-piles equipped with reinforced tips (to facilitate pile penetration through the glacial till deposit) and driven to bedrock.

Downdrag forces due to negative skin friction forces will need to be considered at the abutment locations. Downdrag forces are induced as a result of consolidation of the insitu silty clay caused by the embankment fill load. It is therefore recommended that the approach fills be allowed to settle for a minimum period of 2 months prior to construction as outlined in Section 5.3.3. Similar to the spread footing alternative, the fill will need to be partially excavated to construct the abutments.

Considering the downdrag forces, the following design parameters are recommended for steel H-piles:

Table 1

<u>Foundation Location</u>	<u>Reference Borehole</u>	<u>Estimated Tip Elevation (m)</u>
South Abutment	91-3	78.0
North Abutment	91-2	78.4

Table 2

<u>Pile Type</u>	<u>Factored Capacity at U.L.S. (kN)</u>	<u>Capacity at S.L.S. Type II (kN)</u>
HP 310 x 79	1000	790
HP 310 x 110	1450	1050

Steel H-piles should be driven to refusal with a pile hammer delivering an energy of 3.5 J/mm² to 4.5 J/mm² of steel cross-sectional area. With this energy, refusal may be taken as:

- i) 20 blows for the last 25 mm of penetration; and
- ii) a total of 50 blows for not more than 100 mm of penetration.

In cases where piles do not penetrate the glacial till stratum, the pile capacity should be controlled in the field using current MTO pile driving standards. Additional piles may be required. Attempts should be made in all cases to drive the piles to the bedrock surface.

Pile caps should be provided with 1.8 m of earth cover for frost protection.

The pile caps may be perched within the embankment fill provided that particle sizes within the piling areas do not exceed 75 mm. No dewatering will be required in this case.

Resistance to lateral load for battered piles should be calculated in accordance with Section 6-8.3.8 of the O.H.B.D.C.

5.5 Abutment Backfill

The abutments should be backfilled with free draining materials such as OPSS Granular 'A' or Granular 'B' to prevent hydrostatic pressure build-up.

Computation of earth pressures should be in accordance with Section 6-6.1.2.1 of the O.H.B.D.C. For abutments that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied structures, the at-rest pressure should be used for design, unless the stem can deflect enough (approximately 0.05 percent of the wall height) to establish the active pressure.

For a horizontal backfill, the following parameters are recommended for design:

	Granular 'A'	Granular 'B'
Bulk unit weight, γ (kN/m ³)	22.8	21.2
Effective friction angle, ϕ'	35°	30°
At Ultimate Limit States		
Coefficient of active earth pressure (K_a)	0.34	0.41
Coefficient of earth pressure at rest (K_o)	0.51	0.58
At Serviceability Limit States		
Coefficient of active earth pressure (K_a)	0.27	0.33
Coefficient of earth pressure at rest (K_o)	0.43	0.50

Compaction of the granular backfill near the walls should be carried out using hand-operated equipment to prevent overstressing the abutment walls. Weep holes should be provided in the abutment walls to drain any accumulated water within the backfill.

5.6 Construction Considerations

5.6.1 Dewatering

Only minor dewatering (if any) will be required for the spread footings on compacted fill alternative, and for the driven piles alternative if the pile caps are to be perched within the approach fill. If pile caps are to be placed within the surficial cohesive deposit, no dewatering problems are anticipated due to the impervious nature of the materials. Minor seepage or surface water may be removed by means of a sump pump.

5.6.2 Temporary Excavations

Temporary excavations (if required) less than 1.2 m in depth may be carried out with vertical side slopes. Temporary excavations more than 1.2 m in depth should be undertaken using slopes no steeper than 1 horizontal to 1 vertical. Where seepage is noted, flatter side slopes may be required or alternatively a shoring system may be utilized.

5.7 Groundwater Chemistry

A groundwater sample was submitted to Areco Canada Inc. in Ottawa for pH, sulphate and chloride testing. The test results showed that the sample had a pH of 8.1, a sulphate content of 18 ppm, and a chloride content of 57 ppm.

The above test results indicate that the potential degree of sulphate attack on concrete and the potential degree of attack on exposed steel are both negligible.

6.0 MISCELLANEOUS

The field work for this investigation was carried out under the supervision of Y. Larochelle, Engineer In Training, utilizing equipment owned and operated by Marathon Drilling Limited.

The project was written by Y. Larochelle and C. Kwok, Project Engineer, and approved by G. Kack, Project Manager.

Respectfully submitted,

JACQUES, WHITFORD LIMITED



A handwritten signature in black ink, appearing to read "C. C. Kwok".

Charles C.K. Kwok, M.Sc., P.Eng.
Project Engineer



A handwritten signature in black ink, appearing to read "Gordon J. Kack".

Gordon J. Kack, M.E.Sc., P.Eng.
Project Manager

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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

RECORD OF BOREHOLE No 91-1

METRIC

W P 374-89-03 LOCATION Co-ords: N 4 956 796.69; E 384 739.82 ORIGINATED BY Y.L.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY C.K.K.
 DATUM Geodetic DATE May 6, 1991 CHECKED BY G.J.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										SHEAR STRENGTH kPa			WATER CONTENT (%)		
																		○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					
87.6	Ground Surface																						
	Topsoil																						
0.2	Sand, trace silt		1	SS	3		Seal							○									
0.3	Very Loose Brown																						
	Silty Clay																						
	Hard Brown		2	SS	7									○									
			3	TW	PH									○									
84.2																							

+³, x⁵: Numbers refer to Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 91-2

METRIC

W P 374-89-03 LOCATION Co-ords: N 4 956 742.36; E 384 721.67 ORIGINATED BY Y.L.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, N-Casing, Rock Coring COMPILED BY C.K.K.
 DATUM Geodetic DATE May 3, 1991 CHECKED BY G.J.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH kPa					
86.3	Ground Surface							○ UNCONFINED + FIELD VANE						GR SA SI CL
0.2	Topsoil						Seal	● QUICK TRIAXIAL x LAB VANE						
0.4	Silty Clay Very Stiff to Hard		1	SS	3		86							
			2	TW	PH		85							
			3	SS	12									
							84							
							Seal							
			4	TW	PH		83							
							82							
81.6			5	TW	PH		81							
			6	SS	15		80							
			7	SS	35		79							
78.4							78							
7.9	Bedrock Dolomitic Limestone Fair to Excellent		8	NQ RC	REC 100%		77							
			9	NQ RC	REC 100%		76							
75.6														
10.7	End of Borehole													

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 91-3

METRIC

W P 374-89-03 LOCATION Co-ords: N 4 956 702.54; E 384 718.54 ORIGINATED BY Y.L.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, N-Casing, Rock Coring COMPILED BY C.K.K.
 DATUM Geodetic DATE May 3, 1991 CHECKED BY G.J.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										SHEAR STRENGTH kPa			WATER CONTENT (%)		
																		○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					
86.4	Ground Surface																						
86.2	Topsoil						Seal																
0.2	Silty Clay Very Stiff to Hard		1	SS	4		86 June 25, 1991																
			2	SS	9																		
							85																
							Native Material																
			3	TW	PH		84																
			4	TW	PH		83																
							82																
							Seal																
							Sand Backfill																
							81																
							80																
6.2	Het. Mixture of Sandy Silt, some clay and gravel, occ. boulders (Glacial Till) Very Dense Grey		5	SS	61		Monitoring Well																
							79																
			6	SS	51		78																
8.4	Bedrock Dolomitic Limestone Good to Excellent		7	NQ RC	REC 98%		Seal										RQD = 86%						
			8	NQ RC	REC 100%		77										RQD = 93%						
76.2																							
10.2	End of Borehole																						

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

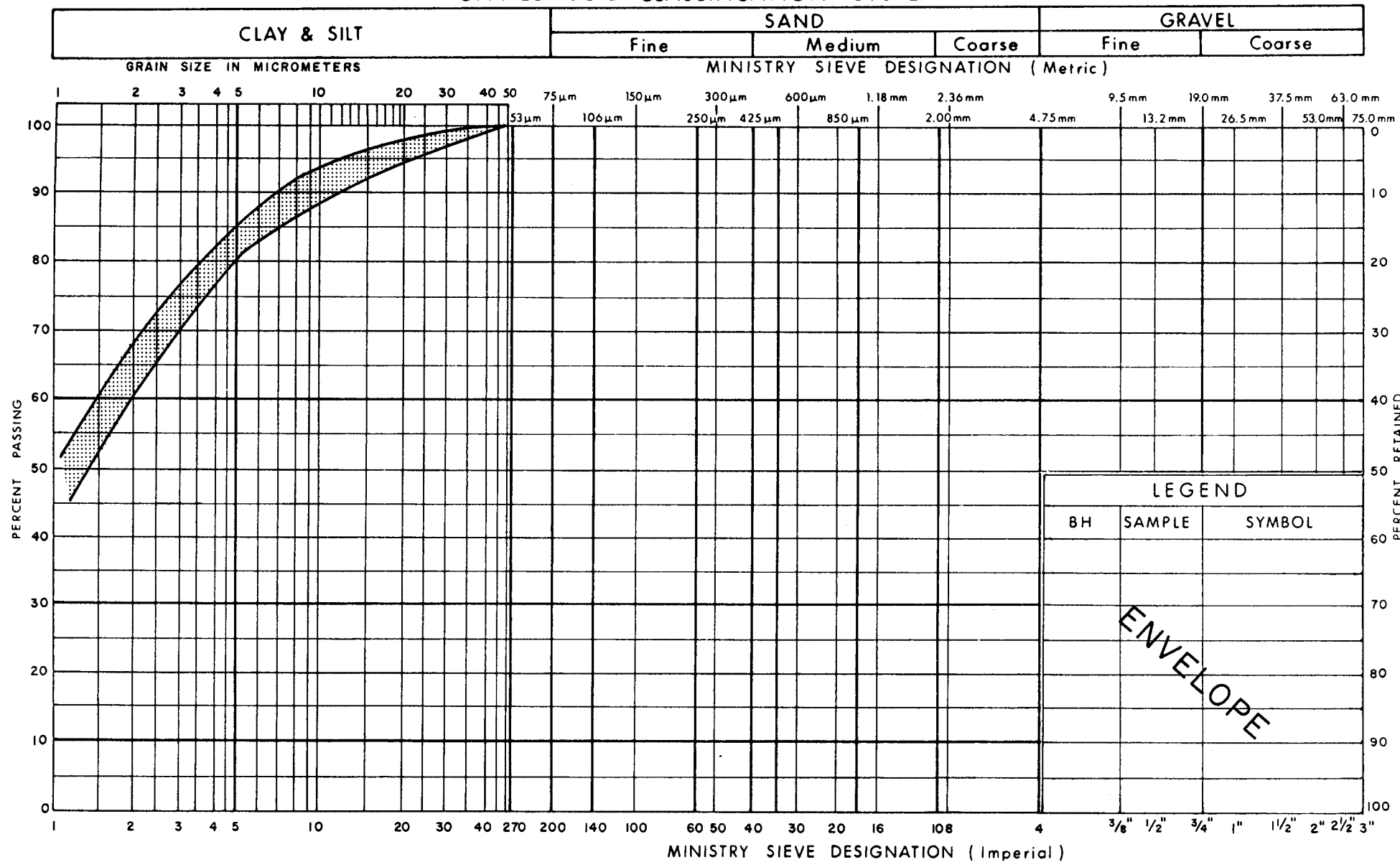
RECORD OF BOREHOLE No 91-4

METRIC

W P 374-89-03 LOCATION Co-ords: N 4 956 667.88; E 384 704.00 ORIGINATED BY Y.L.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY C.K.K.
 DATUM Geodetic DATE May 6, 1991 CHECKED BY G.J.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
85.7	Ground Surface													
85.5	Topsoil						June 25, 1991							
0.2	Sand, trace silt		1	SS	3	Native Material Seal								
0.3	Silty Clay Very Stiff to Hard		2	SS	3	Native Material								

UNIFIED SOIL CLASSIFICATION SYSTEM



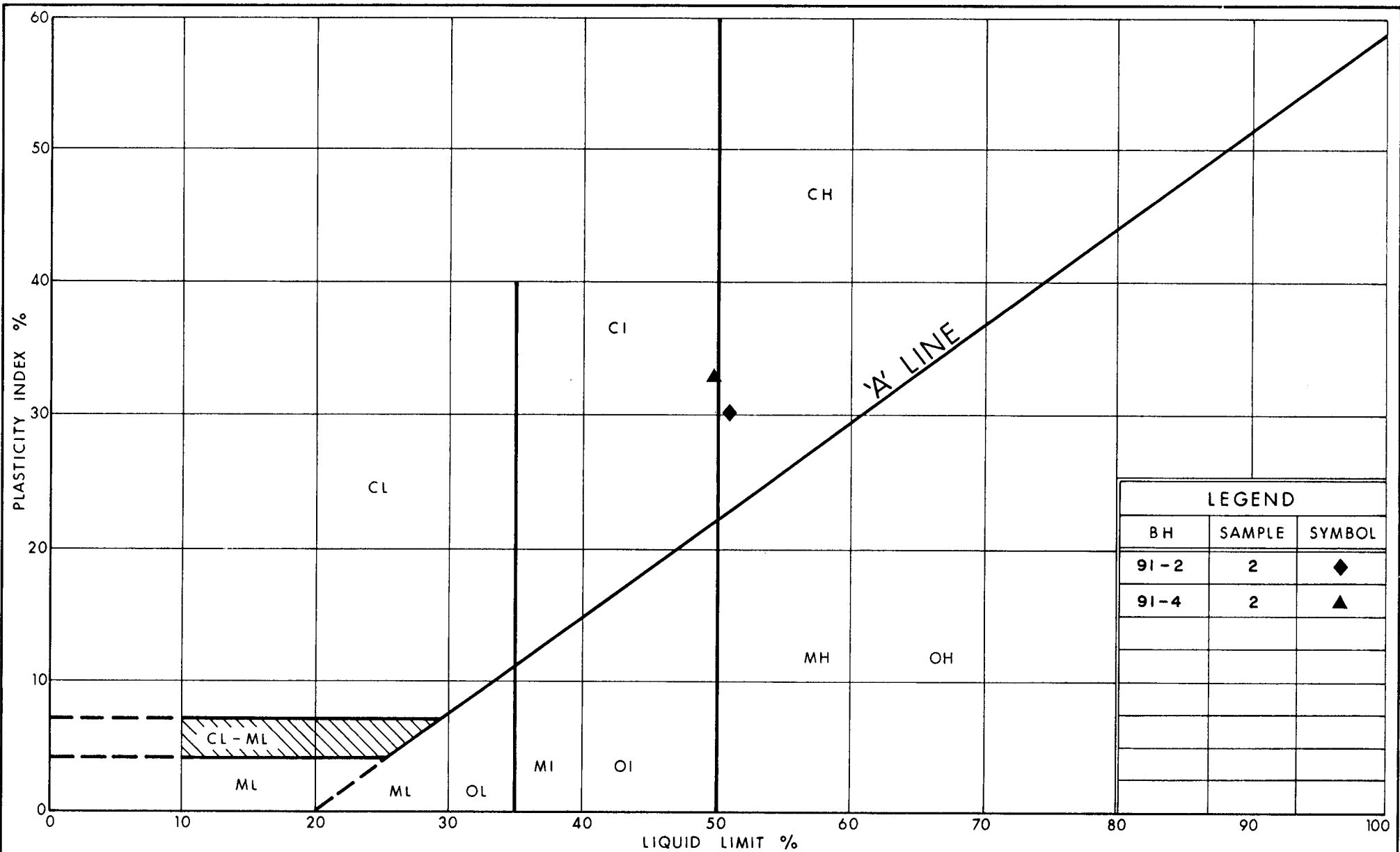
Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIG No 1

W P 374-89-03



Ministry of
Transportation
Ontario

PLASTICITY CHART SILTY CLAY

FIG No 2

W P 374-89-03

VOID RATIO - PRESSURE CURVES

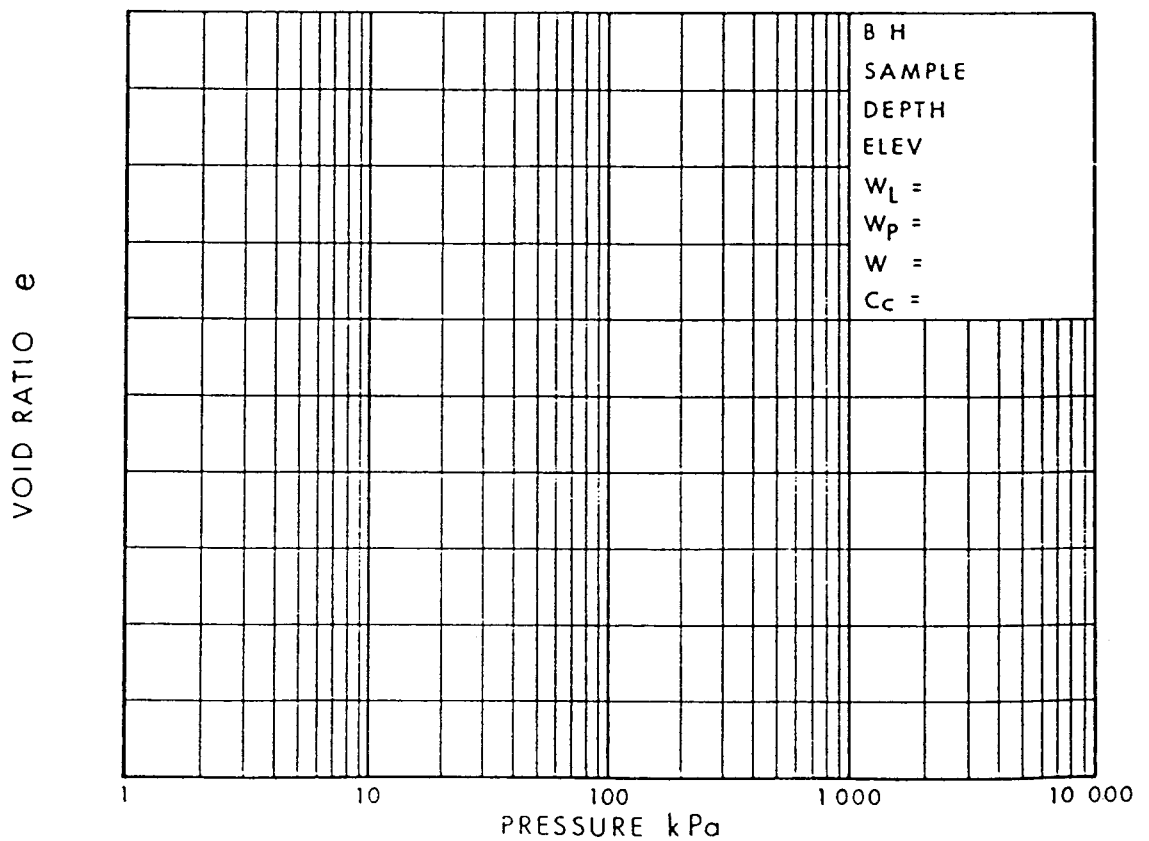
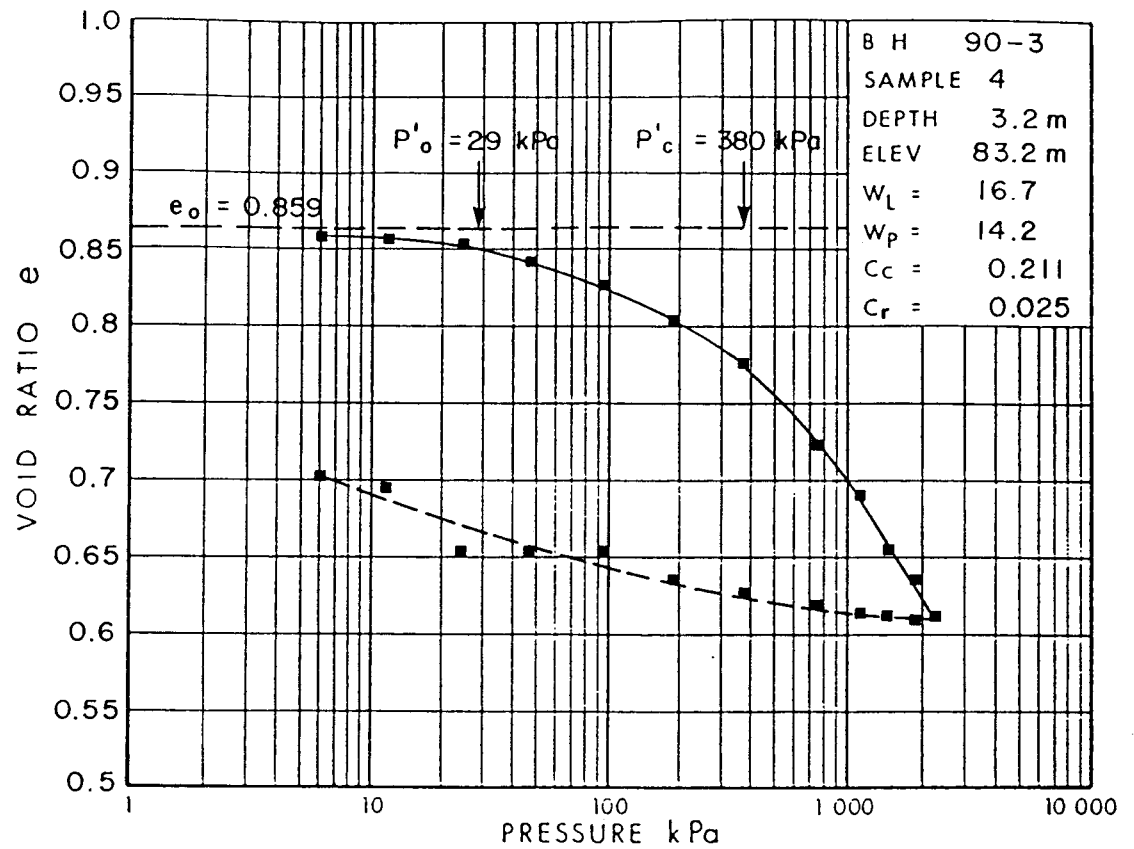
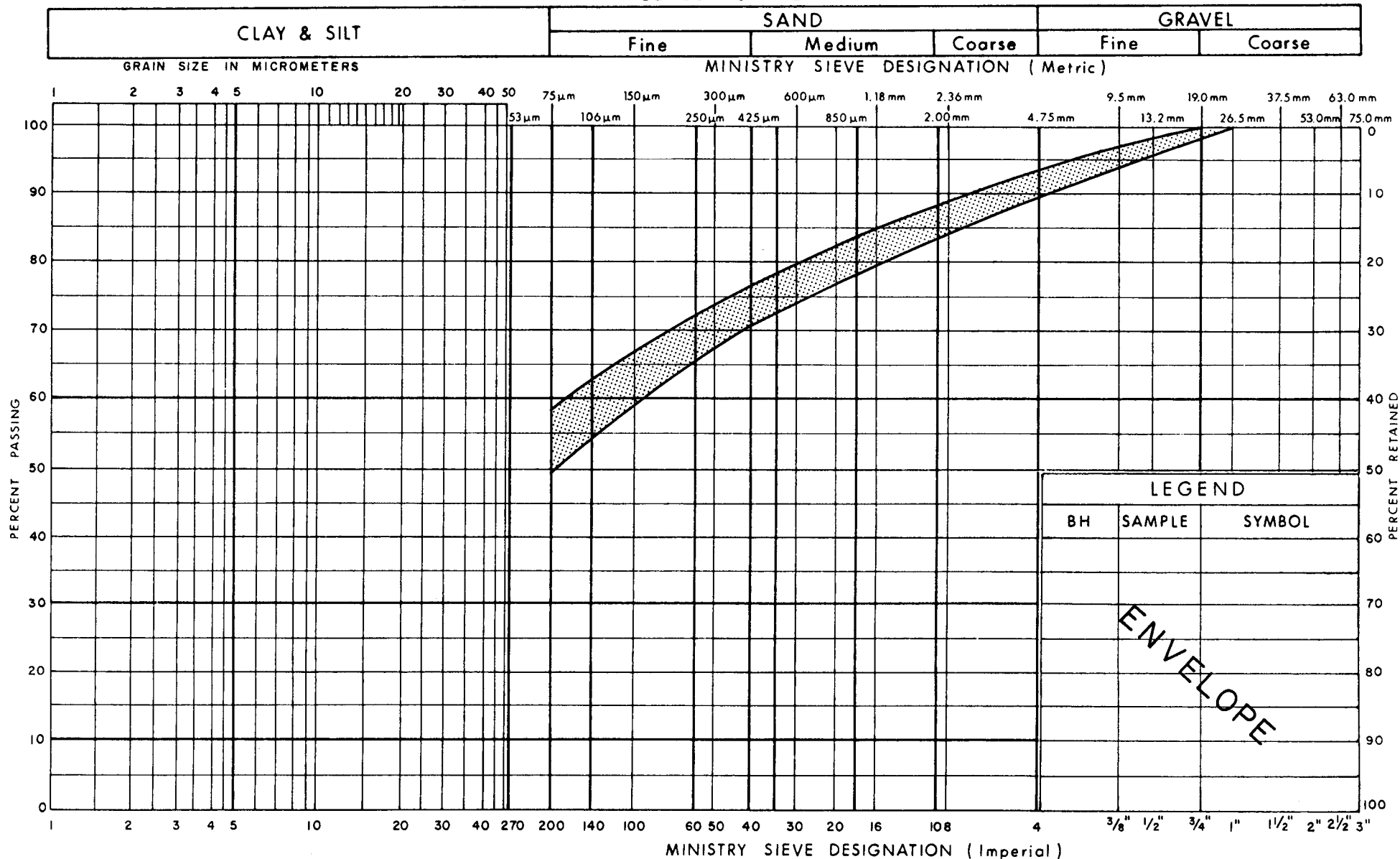


Fig 3

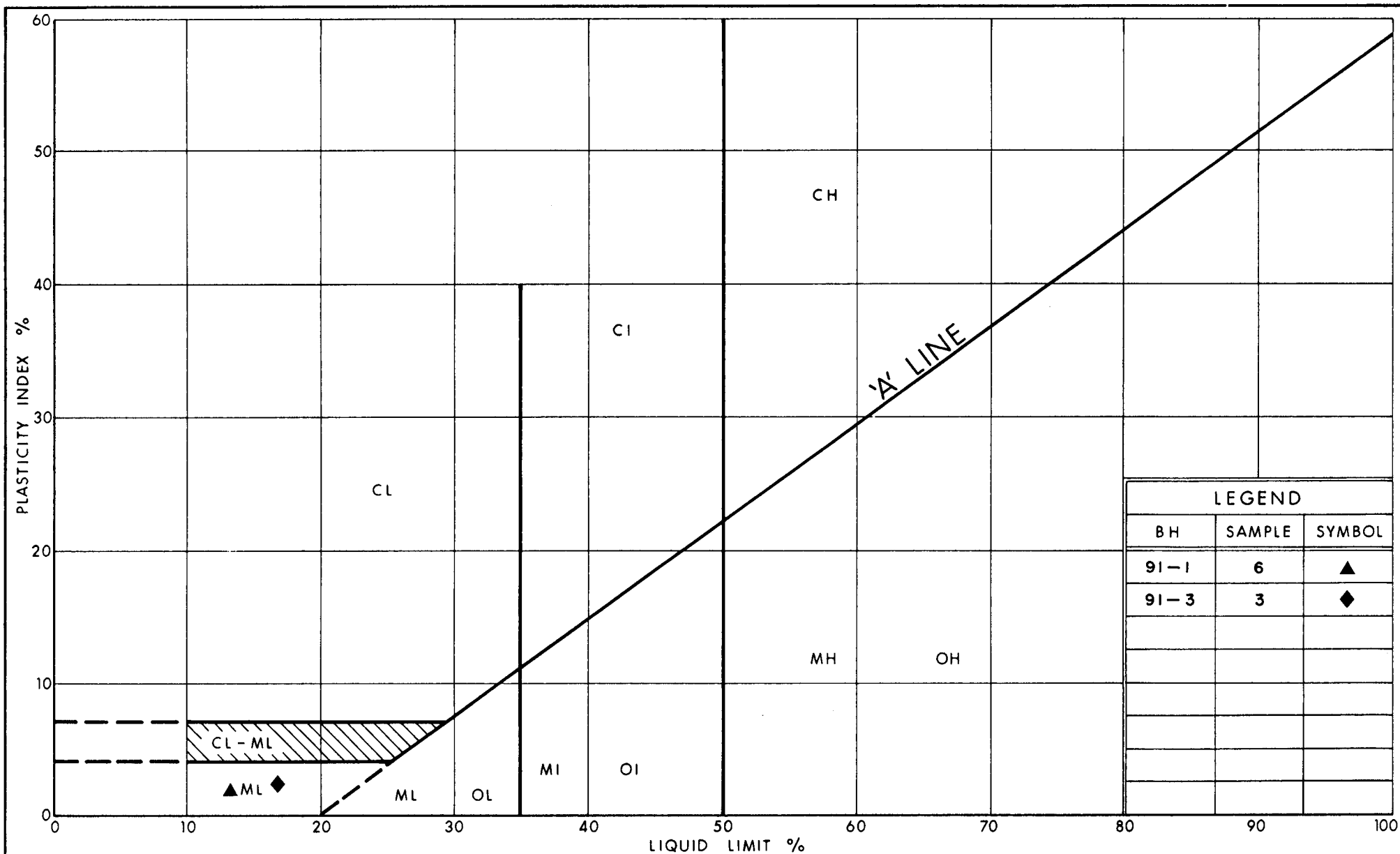
UNIFIED SOIL CLASSIFICATION SYSTEM


 Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
 HET MIXTURE OF SANDY SILT,
 SOME CLAY & GRAVEL, OCCASIONAL BOULDERS (Glacial Till)

FIG No 4

W P 374-89-03

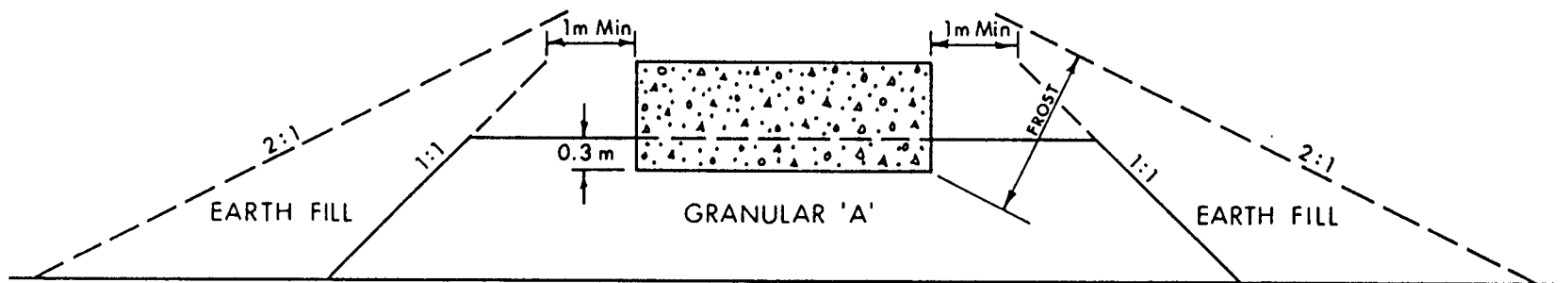


Ministry of
Transportation
Ontario

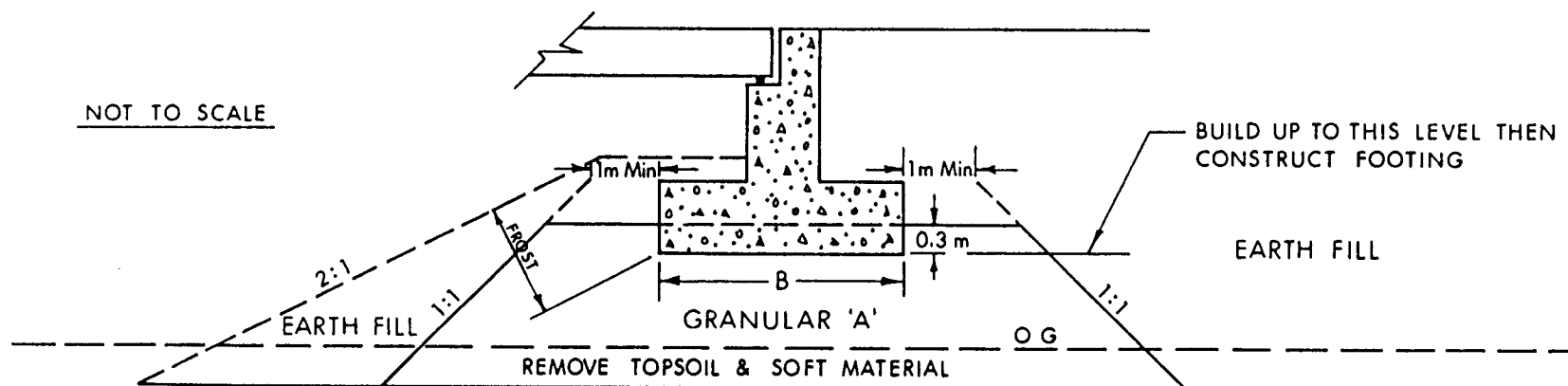
PLASTICITY CHART
HET MIXTURE OF SANDY SILT,
SOME CLAY & GRAVEL, OCCASIONAL BOULDERS (Glacial Till)

FIG No 5

W P 374 - 89 - 03



X SECTION



LONGITUDINAL SECTION

NOTES:

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



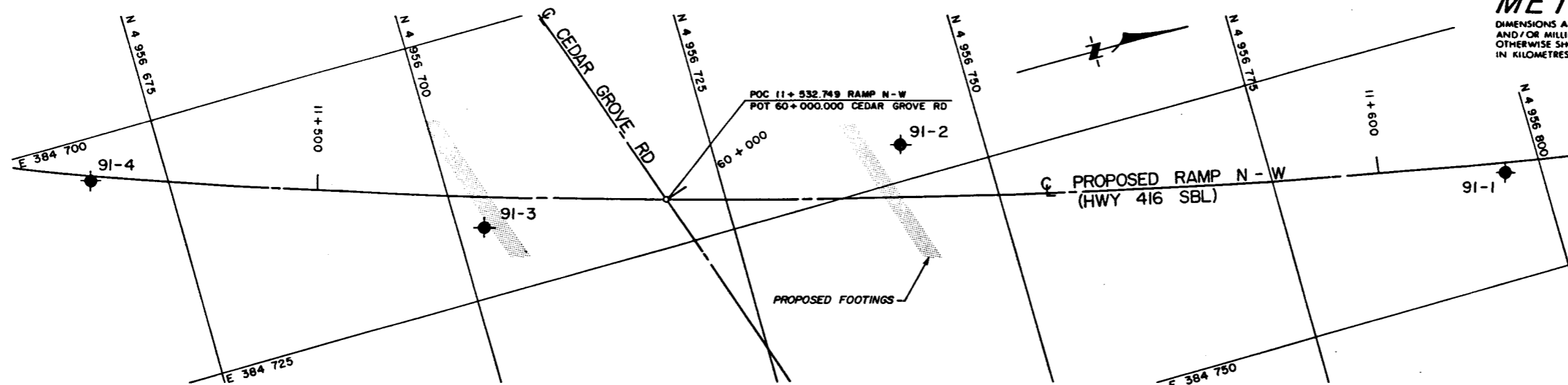
Ontario

Ministry of
Transportation

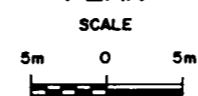
ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 6

W P 374 -89 -03



PLAN



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

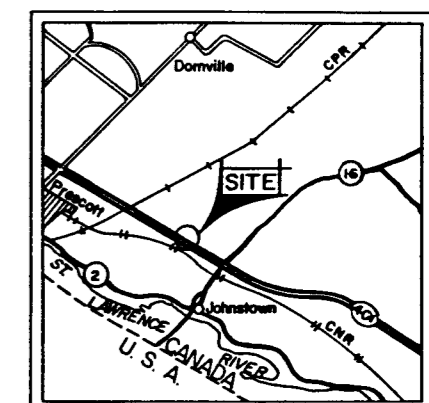
CONT No
WP No 374-89-03



HWY 401/416 INTERCHANGE
RAMP N-W OVER CEDAR GROVE RD
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

JACQUES, WHITFORD LIMITED

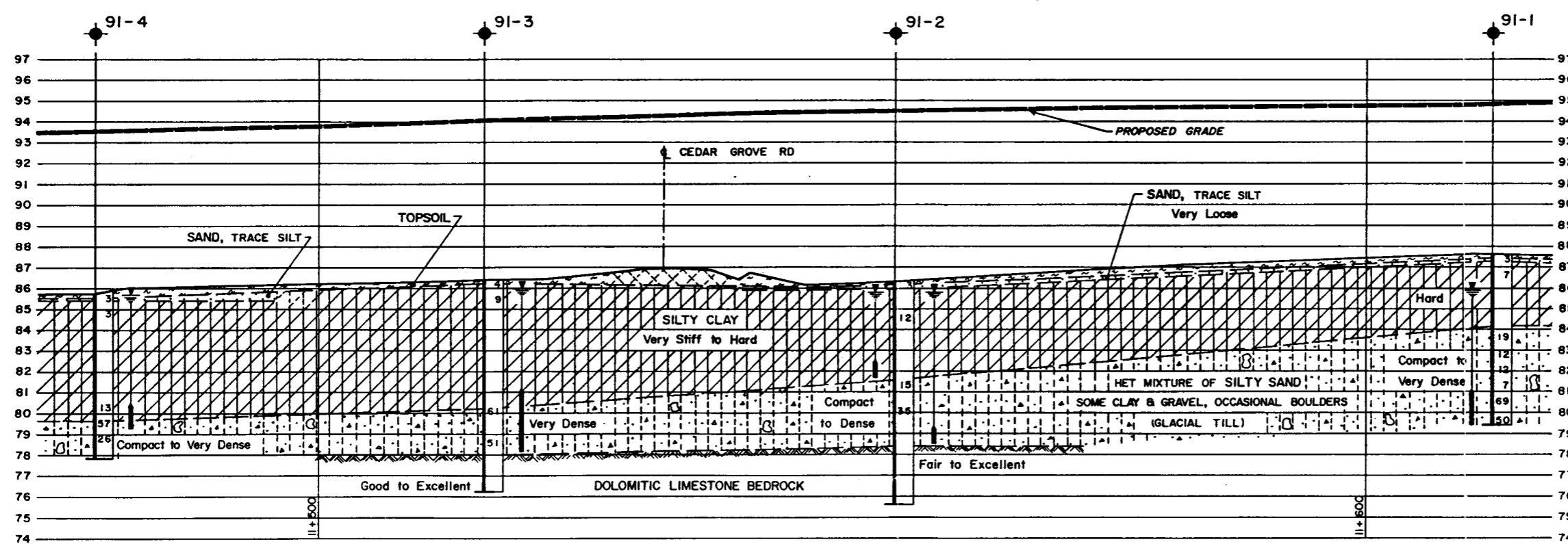


KEY PLAN
SCALE
1km 0 1 2km

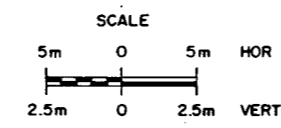
LEGEND

- ◆ 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 91 06
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
91-1	87.6	4 956 796.7	384 739.8
91-2	86.3	4 956 742.4	384 721.7
91-3	86.4	4 956 702.5	384 718.5
91-4	85.7	4 956 667.9	384 704.0



PROFILE PROPOSED RAMP N-W



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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION

Geocres No **31B-75**

HWY No 416	DIST 9
SUBM'D CKK	CHECKED DATE JULY 19, 1991 SITE 16-307
DRAWN GBB	CHECKED APPROVED DWG 3748903-A