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FOUNDATION INVESTIGATION

PROPOSED
UNDERPASS STRUCTURE
RICHARDSON ROAD OVER HIGHWAY 417
OTTAWA, ONTARIO
CONT 91-52
W.P. 34-81-04 SITE NO. 3-570

DISTRICT 9, EASTERN REGION

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

OUR FILE NO. 89348
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ABSTRACT

Terraprobe Limited was authorized by the Honourable E. Fulton, Minister, on behalf of The Ministry of Transportation, Ontario (MTO), to undertake a foundation investigation for a proposed bridge structure over Highway 417 (currently Highway 17), in Ottawa, Ontario.

The proposed structure will have two spans of approximately 30 m length which will support two 3 m wide lanes with 1 m shoulders. The current 2-lane Highway 17 will be widened to 4 lanes and aligned in an approximately east/west direction beneath the proposed structure. The alignment for the bridge structure will coincide with the existing Richardson Road (formerly called Cowan Road).

The field investigation for the project consisted of eleven (11) boreholes at the proposed bridge pier and abutment locations, and along the alignment of the proposed approach embankments.

The boreholes generally encountered a thin layer of granular fill (likely associated with the existing roads) at the ground surface. Beneath the fill, native sand and gravel was found to depths of 1.5 to 3.4 m. The sand and gravel was underlain by dense silty glacial till. Limestone bedrock was found at depths of 10 to 10.2 m.

The groundwater table was encountered at 1.3 to 1.6 m below the ground surface, in the sand and gravel materials.

In summary, it is considered that the underpass structure can be founded on shallow spread footings placed directly on the native sand and gravel, glacial till or engineered fill.

The ground conditions will pose few geotechnical constraints to the design and construction of the embankment fills. The compact to dense sand and gravel deposits

encountered along the approaches will permit construction of the embankment without stability concerns. Settlements will be primarily elastic, and will occur during fill placement.

Positive groundwater control will be required for open excavations carried below the water table in the sand and gravel soils.

1. INTRODUCTION

Terraprobe Limited was authorized by the Honourable E. Fulton, Minister, on behalf of The Ministry of Transportation, Ontario (MTO), to undertake a foundation investigation for a proposed bridge structure over Highway 417, in Ottawa, Ontario. The structure will have two spans of 30 m each, and will be constructed along the alignment of the existing Richardson Road (formerly Cowan Road). The width of the travelled portion of the structure will be approximately 8 m. Approach fills of up to 6 m height will be constructed.

The purpose of the investigation was to determine the subsurface conditions at the site, and to provide engineering recommendations for the geotechnical aspects of design and construction of bridge foundations and approach embankments.

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. W.P.34-81-04-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).

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.W.D. 34-81-04-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.

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

4. DISCUSSION AND RECOMMENDATIONS

The following discussions and recommendations are based on the factual data obtained from the boreholes and subsequent laboratory testing, and are presented for the guidance of the design engineer only. Contractors bidding on or conducting work associated with this project should review the factual information to assess their effect on proposed construction methods and scheduling. In addition, the comments and recommendations are based on information obtained from widely spaced boreholes, between which subsurface conditions may vary considerably.

The proposed alignment and position of the proposed overpass structure is shown on Drawing W.P. 34-81-04-A.

This investigation and report addresses only the foundations for the bridge structure and the west and east approach embankments. It should be noted that there are no existing bridge structures in the immediate vicinity of the proposed underpass, nor are there any proposed bridge structures which will be influenced by the structure or the approach embankments.

4.1 Foundations for East and West Abutment and Central Pier

The native sand and gravel, glacial till, and bedrock strata encountered in the investigation are considered to be competent materials which will be suitable for support of the proposed bridge structure. Several possible foundation alternatives can be considered for the abutment and central pier foundations, including shallow spread footings, engineered fill, caissons or piles.

However, based on the competent soil conditions, and the proposed grades and alignment of the structure, it appears that shallow spread footings are the most reasonable foundation alternative.

4.1.1 Shallow Foundations

Shallow spread footings may be constructed on the competent sand and gravel or glacial till strata. At the abutment locations, spread footings may also be placed on engineered fill materials. Each of these alternatives is described further below.

The native sand and gravel materials will be suitable for the support of shallow spread footings. The following soil properties may be used for preliminary design of foundations placed at least 0.5 m below the top of the sand and gravel stratum. These properties are not factored:

Effective angle of internal friction	32 degrees
Effective cohesion intercept	0
Unit weight	21 kN/cu.m

Based on a foundation width of approximately 2 m, and a minimum foundation depth of about 1.8 m (minimum frost cover the Ottawa area), then the following typical bearing capacities are calculated. These capacities are based on an assumed total settlement of about 25 mm.

Sand and Gravel - 2 m Wide Footing	
Bearing capacity - ultimate limit states	600 kPa
Bearing capacity - serviceability limit states, (Type 2)	300 kPa

It is noted that the ultimate and serviceability limit states bearing capacity will vary according to foundation width and depth.

Considerable seepage can be expected in excavations carried below the water table through sand and gravel materials. Unless the water table is lowered to below the base of the excavation prior to construction, this may result in loosening of the soils and loss of bearing capacity. Therefore, adequate groundwater control should be implemented as noted in subsequent sections of this report.

The glacial till materials will also be suitable for the support of shallow spread footings. The glacial till materials are considered to be an unyielding soil in the context of Section 6 - 5.3.1 of the OHBC. A nominal factored bearing capacity at ultimate limit states of 1,000 kPa can be assumed for design. The bearing capacity at the serviceability states, Type 2, does not generally apply to unyielding soils such as glacial till, since the loads required to produce detrimental settlement are generally larger than the recommended values of bearing capacity at the ultimate limit states.

Glacial Till

Bearing capacity - ultimate limit states	- 1,000 kPa
Bearing capacity - serviceability limit states (Type 2)	- not applicable, unyielding

The above bearing capacities can generally be applied for footings or drilled piers placed at any elevation within the till stratum.

It is noted that local pockets or zones of sandy material may be encountered within the glacial till. These zones may become loosened as the result of groundwater seepage. Local subexcavation of these zones may be required at the foundation base. The possible requirements for subexcavation can be determined by inspection at the time of construction.

The abutments can also be founded on a raft of select engineered fill such as OPSS Granular "A". The fill must be placed in maximum 200 mm loose lifts and compacted to a minimum of 100 percent of Standard Proctor Maximum Dry Density. The engineered fill will consist of an embankment about 6 m high, with side slopes and configuration as shown on Figure 4.

Prior to placement of the engineered fill, the subgrade should be stripped of all topsoil and deleterious materials and proof rolled with a heavy vibratory roller. All soft spots should be further subexcavated and backfilled with an approved granular material compacted to 100 percent of Standard Proctor Maximum Dry Density.

Spread footings placed on this engineered fill may be designed using a maximum factored bearing capacity at Ultimate Limit States of 900 kPa. This should be corrected for any load inclination. The recommended bearing capacity at Serviceability Limit States, Type 2 is 350 kPa. At this loading, a total settlement of about 25 mm is expected.

Engineered Fill	
Bearing capacity - ultimate limit states	- 900 kPa
Bearing capacity - serviceability limit states, Type 2	- 350 kPa

For cast-in-place concrete poured on the Granular "A" raft, the factored coefficient of friction against sliding at Ultimate Limit States would be approximately 0.6, assuming an unfactored friction angle of 35 degrees.

4.1.2 Deep Foundations

Finally, the underlying limestone bedrock materials will be suitable for the support of driven end-bearing piles or drilled caisson foundations.

Due to the dense nature of the overlying glacial till, some difficulty may be experienced in driving piles to end bearing and refusal on the underlying bedrock. A driving shoe or preaugering may be required in order to permit pile driving.

Since the underlying limestone bedrock is considered an unyielding strata, the factored bearing capacity at ultimate limit states will generally govern for design. For deep drilled piers which are founded on the limestone bedrock, a nominal bearing pressure of 3,000 kPa may be used for initial design purposes.

For driven piles, the following axial capacities can be assumed for design purposes:

Pile Type	Axial Capacity (SLS Type I)	Factored Axial Capacity ULS
HP 310 x 79	900	1150
HP 310 x 110	1150	1600

4.2 Excavations

Excavations through the native sand and gravel materials will generally stand at inclinations of about 1-1/2 to 1 above the water table. The water table was encountered at depths of about 1.3 to 1.6 m, in the sand and gravel soils. The sand and gravel is limited in thickness, and is underlain by a slightly cohesive glacial till. The water table is situated about 0.5 to 1 m above the till strata. Considerable seepage can be expected through the sand and gravel, causing sloughing of excavations carried below the water table. Only minor volumes of seepage are expected through the glacial till, due to its fine-grained and cohesive nature.

The site conditions suggest that adequate groundwater control could be achieved by constructing a perimeter ditch or series of sumps around the excavation. The sumps/ditch should be carried at least 0.5 m below the base of the excavation, and preferably into the underlying till. This will permit interception and collection of the seepage from the sand and gravel strata.

It may also be possible to reduce seepage through the sand and gravel strata by driving interlocking sheet piling into the underlying glacial till. However, considering the dense and granular nature of the soil, it may be difficult to ensure that the sheet piles are adequately seated into the underlying till, and that they do not become damaged during driving. Therefore, groundwater control as noted above appears more feasible.

As noted previously, the silty sand till soils are relatively fine grained and are not suited to conventional dewatering (ie. lowering of the groundwater table). It is expected that excavations carried through the silty sand till materials and below the water table will experience sloughing and caving only through local silty and sandy zones in the soil. Since the till is slightly cohesive, most of the soil mass is expected to remain stable even if excavations are carried below the water table. Local sandy pockets or zones in the till may become loosened as the result of seepage. These zones can be subexcavated at the time of construction, as required. A working mat or base of graded granular material, such as OPSS Granular "A", should be placed in the base of all excavations to improve trafficability. Seepage should be pumped away from the base of the excavation using a series of conventional sump pits and pumps. As noted previously, a ditch around the perimeter of the excavation would be useful in collecting the seepage.

Where workmen must enter excavations carried deeper than 1.2 m, the trench excavation should be sloped and/or braced in accordance with the Occupational Health and Safety Act.

4.3 Abutment Backfill

Select, free-draining granular fill, such as OPSS, Granular 'A' or 'B', should be used as backfill behind the bridge abutments. The select fill should be placed in a wedge shaped zone extended from 1.2 m behind the base of the wall and up at a 60 degree angle, as per OHBDC Section 6-9. The backfill material should be drained by perforated pipes or weep holes.

Heavy compaction equipment should not be used behind the wall within a lateral distance equal to the current height of fill above the wall footing, in order to minimize deflection or possible damage of the wall.

Provided the above backfill criteria are satisfied, the following soil properties and parameters may be used in calculation of lateral earth pressures, in accordance with the OHBDC. It is noted that these are based on the assumption that the backfill is close to horizontal.

	Granular 'A'	Granular 'B'
Effective Angle of Internal Friction (Phi),degrees, unfactored	35	30
Unit Weight (Gamma), kN/m_3	22.8	21.2
Active Earth Pressure Coefficient, K_a (SLS)	0.27	0.33
(ULS)	0.36	0.41
At rest Earth Pressure Coefficient, K_o (SLS)	0.43	0.50
(ULS)	0.53	0.58

It should be noted that the mobilization of the active earth pressure behind the wall will require an outward deflection of up to 0.5 per cent of the wall height, as measured at the top of the wall.

If the bridge is a rigid frame structure, and the abutments are constrained so that this deflection cannot occur, then the at-rest earth pressure should be used in design.

All new or additional fill materials placed beneath the future roadway area, and in the footing excavations, should be compacted to a minimum 95 percent Standard Proctor Maximum Dry Density in lifts not exceeding 200 mm. It is important to achieve adequate compaction to minimize future settlement of backfill behind the abutments.

4.4 Embankment Fill Placement

The underlying sand and gravel and glacial till soils are considered suitable for support of the proposed 6 m high embankment. Shear failure or settlement of the underlying native soil beneath the embankment will not pose a constraint to embankment design. Generally, the embankment slopes should be constructed at about 2 to 1 or flatter (horizontal to vertical) to facilitate construction and later maintenance of vegetation.

The subgrade beneath the embankments should be stripped of all topsoil and the exposed subgrade proof rolled prior to fill placement. All weak or soft spots should be further subexcavated and backfilled with suitable fill material compacted to 95 percent Standard Proctor Maximum Dry Density.

Settlement of the embankment foundation is expected to be elastic in nature, and will generally occur during fill placement.

4.5 Consideration of Embankment Seismic Stability

The proposed structure is located in an active earthquake zone, and may be subject to seismic loadings. Based on the data provided in the Canadian Foundation Engineering Manual, 2nd edition, the peak horizontal ground acceleration is estimated at 0.2g (City of Ottawa) for an earthquake probability of exceedance of 10 percent in 50 years.

The horizontal acceleration experienced by the embankment will be equal to the design acceleration at the ground level (0.2g in this case), and will increase with height to a maximum value at the crest of the embankment. This is the result of upward propagation and amplification of the earthquake forces. A detailed analysis of the seismic stability of the embankment fill was not conducted. However, based on site conditions, and the embankment configuration, there is not expected to be significant shear mobilization (movement) of the embankment fill. This is based on the following considerations:

i) the foundation subsoils are dense, and not subject to liquifaction

ii) the proposed approach embankment will have a high static factor of safety with respect to sliding or shear failure (likely greater than 2)

iii) the peak ground acceleration is relatively low.

Empirical knowledge suggests that embankments with a high static factor of safety, and a competent foundation, will generally not be subject to significant movement at the expected peak ground acceleration (0.2g).

Respectfully submitted,

TERRAPROBE LIMITED

K. Johnson

Kirk R. Johnson, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
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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/0.3 m)	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
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

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 W.P. 34-81-04-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.

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.

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

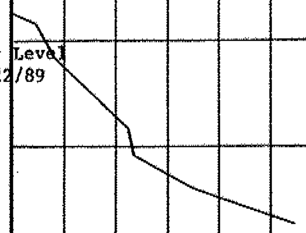
W P 34-81-04 LOCATION Sta 10 + 105 o/s 5.0m RT ℄ Richardson Rd. ORIGINATED BY KJ

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

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



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

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

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

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

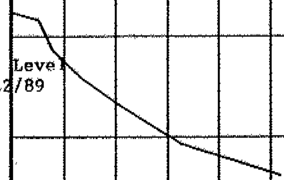
DATUM Geodetic DATE December 18, 1989 CHECKED BY PB

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

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL; WATER CONTENT (%) 10 20 30



OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

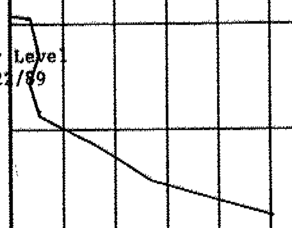
W P 34-81-04
LOCATION Sta 10 + 055 o/s 4m Rt. ℄ Richardson Rd.
ORIGINATED BY KJ

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	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT
131.0	Ground Surface						
0.0	Fill, sand and gravel, trace silt. Compact Brown	1	CS	-	Seal		o
130.3 / 0.7	Sand and Gravel, some silt, trace clay. Compact to Dense, Grey	2	SS	50/50mm		130	o
					Water Level Dec.21/89		
		3	SS	25		129	o
		4	SS	50			o
128.3 / 2.7	Silty Sand, trace gravel and some clay. Very Dense, Grey (Glacial Till) cobbles	5	SS	98		128	o
		6	SS	95		127	o
		7	SS	51/150mm		126	
						125	
124.4		8	SS	35			o
6.6	End of Borehole						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); 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 (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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 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	SOIL PROFILE 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						
		2	SS	19		130					
129.8 / 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					
		5	SS	95							
127.6 / 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					
		9	SS	48		124, 123					
		10	SS	40		122, 121					
120.9	Auger Refusal										
10.1	Bedrock Limestone. Grey	11	RC NQ	100% RQD 90%		120					
119.6 / 11.4	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; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

OFFICE REPORT ON SOIL EXPLORATION

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

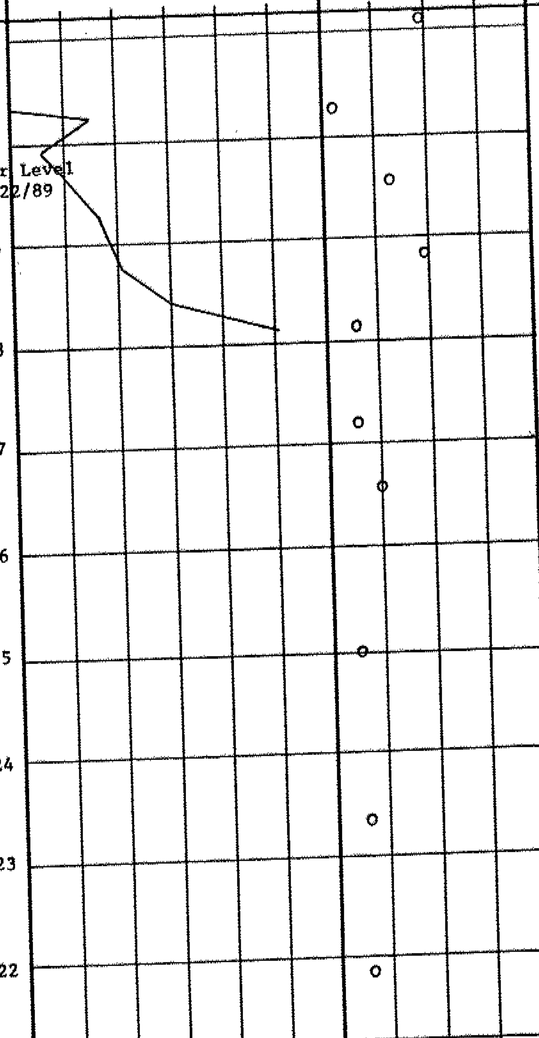
DATUM Geodetic DATE December 19, 1989 CHECKED BY PB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
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.		2	SS	62		130		
			3	SS	27	Water Level Dec.22/89			
	Compact, Brown		4	SS	25		129		
128.5 / 2.7	Silty Sand, trace gravel and some clay.		5	SS	70/150mm		128		
			6	SS	78		127		
	Very Dense, Grey		7	SS	60/150mm		126		
	(Glacial Till)		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, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE

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



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

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

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 7

METRIC

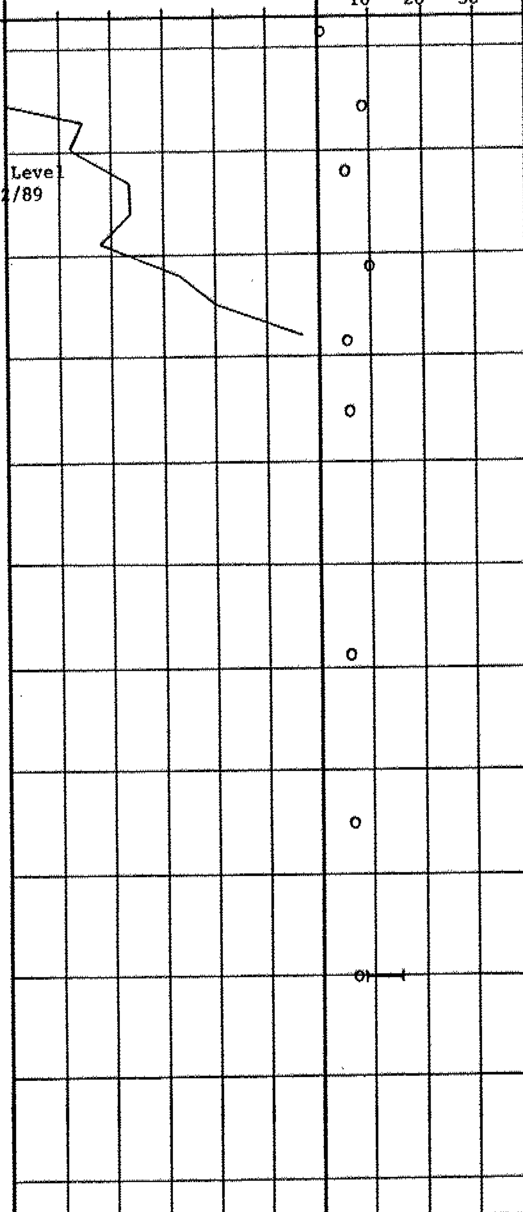
W P 34-81-04 LOCATION Sta 9 + 970 o/s 4.5m Rt ₵ Richardson Rd. ORIGINATED BY KJ

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	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. Very Dense Brown	2	SS	23							
		3	SS	62	Water Level Dec.22/89	130					
129.0 / 2.3	Silty Sand, trace gravel, some clay, occasional cobbles. Very Dense Grey (Glacial Till)	4	SS	88		129					
		5	SS	95		128					
		6	SS	58							
		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	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



REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^5$: Numbers refer to Sensitivity   15 ⌖ 5 (20, 10) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 8

METRIC

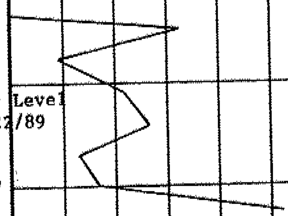
W P 34-81-04 | LOCATION Sta 9 + 970 o/s 4.5m Lt. ₵ Richardson Rd. | ORIGINATED BY KJ

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	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	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		
		2	SS	83				
130.5 / 1.0	Sand and Gravel, some silt, trace clay. Very Dense to Compact Brown	3	SS	6/150mm	Water Level Dec.22/89	130		30 50 12 8
		4	SS	27		129		
128.8 / 2.7	Silty Sand, trace gravel, trace to some clay. Very Dense Grey (Glacial Till)	5	SS	73			22.7	
						128		
		6	SS	53				
		7	SS	72		127		
						126		
		8	SS	60/150mm			22.3	
						125		
						124		
		9	SS	55				
						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 (20 40 60 80 100)



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

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT WL; WATER CONTENT (%) 10 20 30

OFFICE REPORT ON SOIL EXPLORATION

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

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

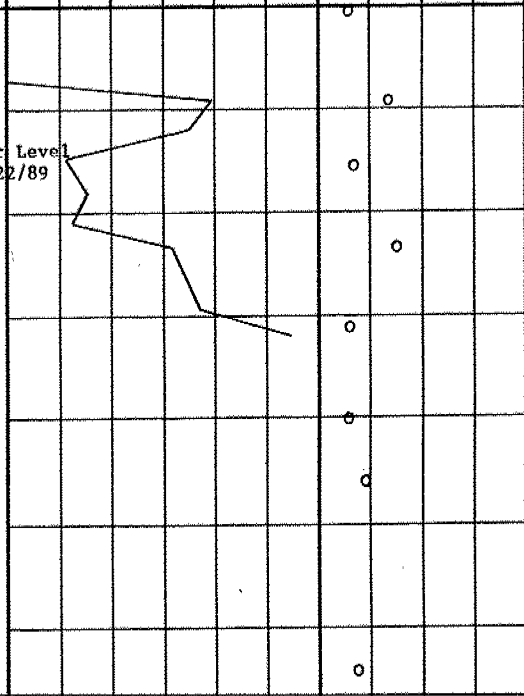
DIST 9 HWY 17 | BOREHOLE TYPE Hollow Stem Auger, Cone Test | COMPILED BY KJ

DATUM Geodetic | DATE December 12, 1989 | CHECKED BY PB

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	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	129
			4	SS	20		
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
							125
124.4			8	SS	60		
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

OFFICE REPORT ON SOIL EXPLORATION

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

15 ⌖ 5 (20 above, 10 below) (%) 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

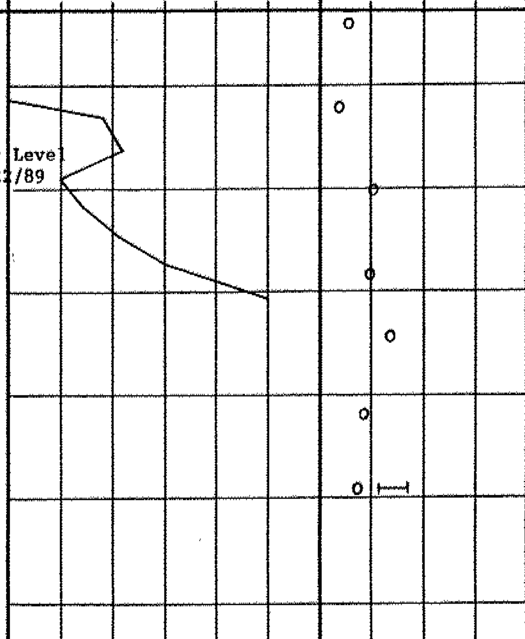
DATUM Geodetic DATE December 12, 1989 CHECKED BY PB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
130.7	Ground Surface								
0.0	Fill, sand and gravel.	1	CS	-	Seal	130			
129.7	Compact Brown								
1.0	Sand and Gravel, some silt, trace clay.	2	SS	47	Water Level Dec.22/89	129			
		3	SS	17					
128.0	Compact Brown	4	SS	32		128			
2.7	Silty Sand, trace gravel, some clay.	5	SS	60		127			
	Very Dense Brown	6	SS	73/150mm					
	(Glacial Till)	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: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

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



REMARKS & GRAIN SIZE DISTRIBUTION (%)

OFFICE REPORT ON SOIL EXPLORATION

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

Ministry of Transportation
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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

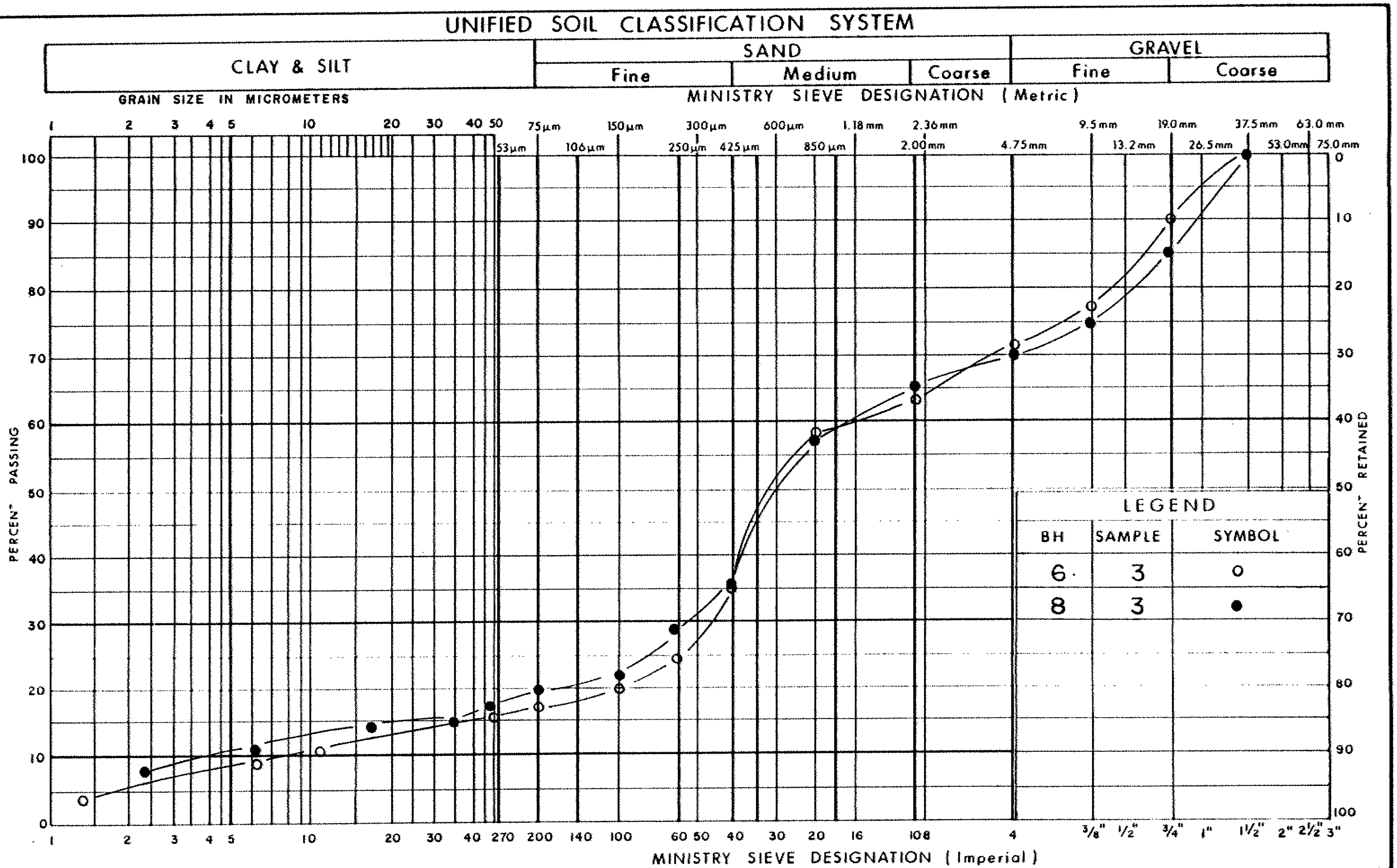
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL — WATER CONTENT (%) 10 20 30	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.22/89					
1.2	Silty Sand, trace gravel, some clay.		3	SS	64		129				
	Very Dense Brown (Glacial Till)		4	SS	88		128			21.4	
127.1			5	SS	113						
3.5	End of Borehole										

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

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

78 12 M



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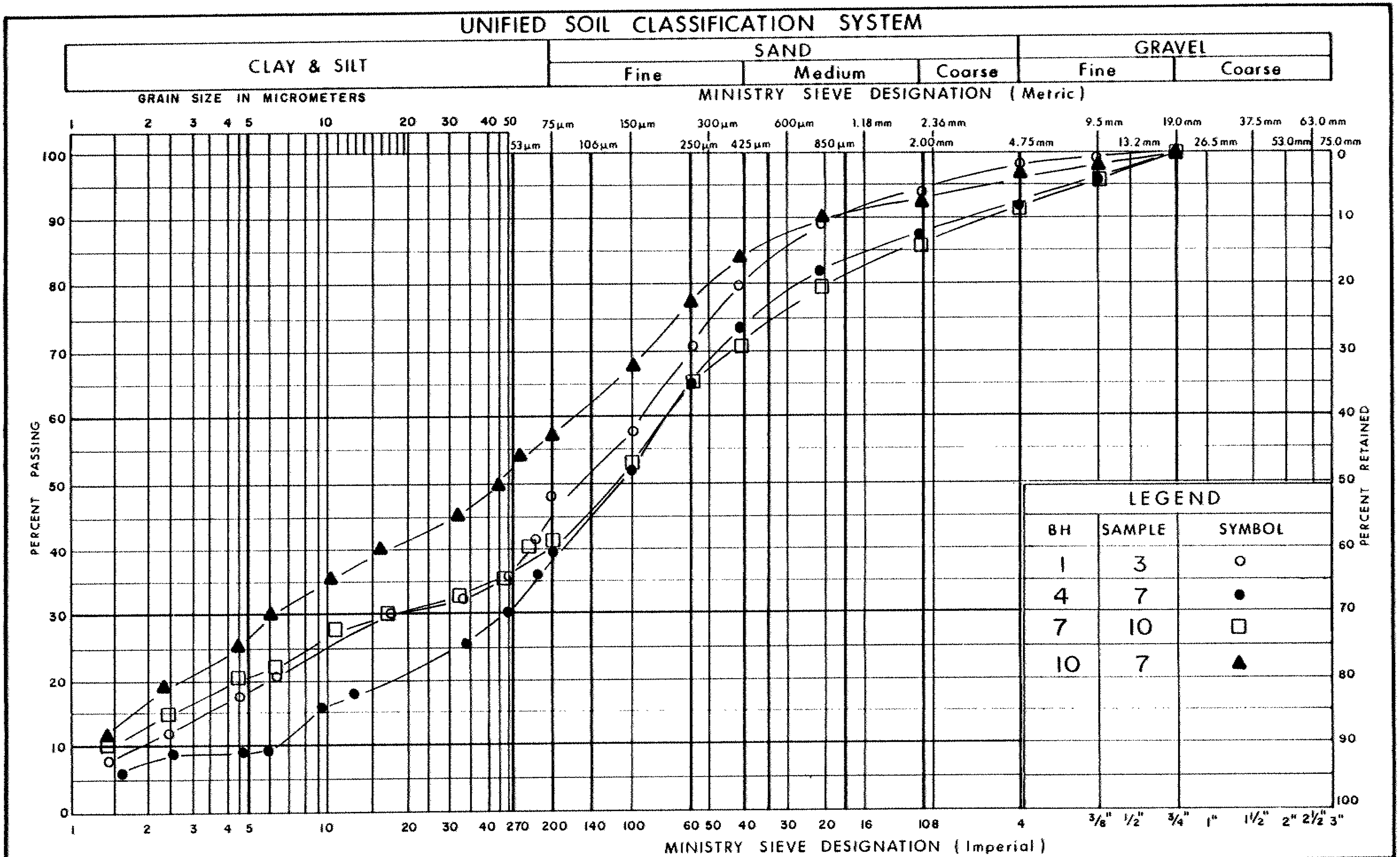
GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL SOME SILT, TRACE CLAY
(SM) (TILL)

FIG No 1

W P 34-81-04

78 12 M



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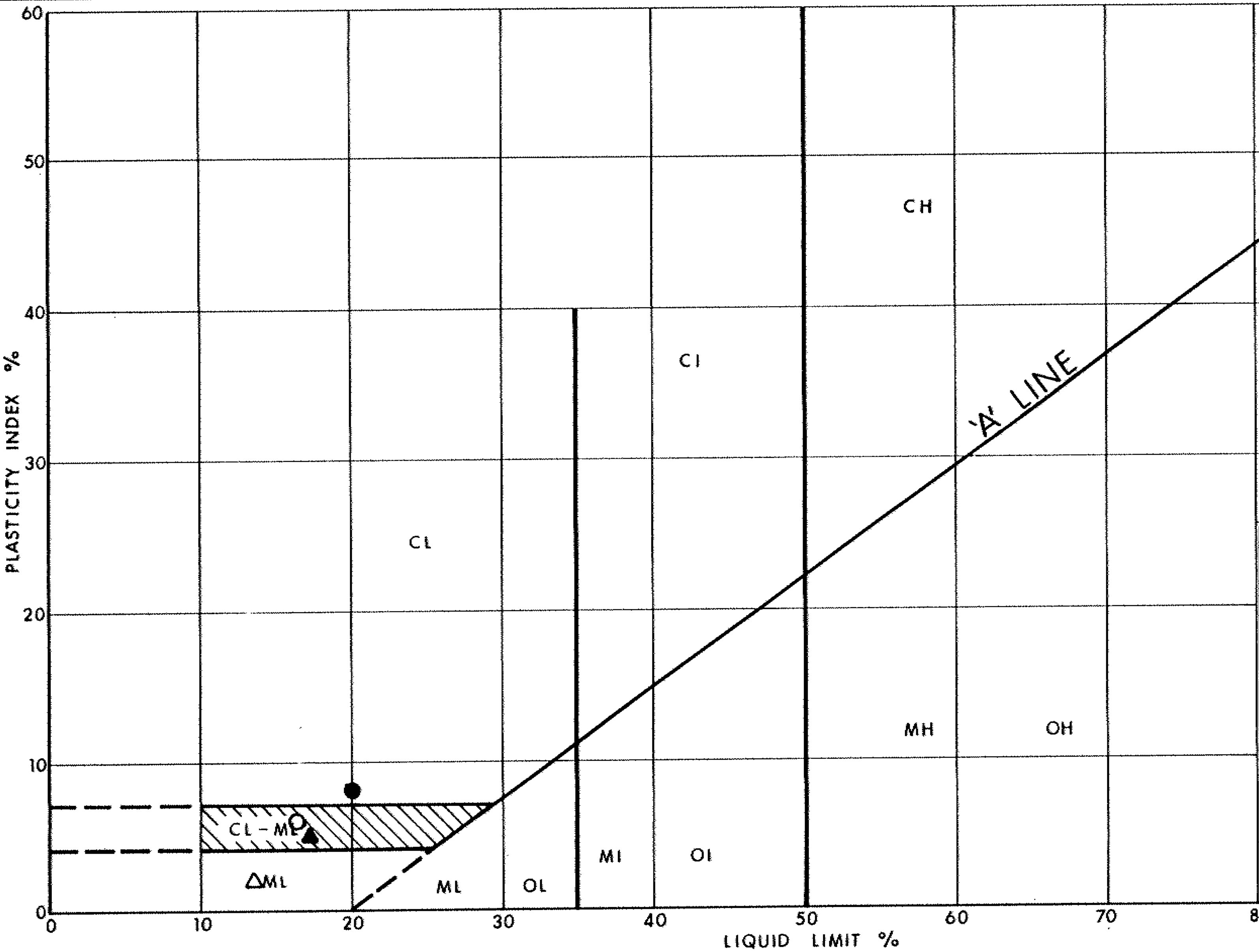
GRAIN SIZE DISTRIBUTION

SILTY SAND TRACE GRAVEL, SOME CLAY (TILL)
(SM)

FIG No 2

W P 34-81-04

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
1	3	●
4	7	△
7	10	○
10	7	▲

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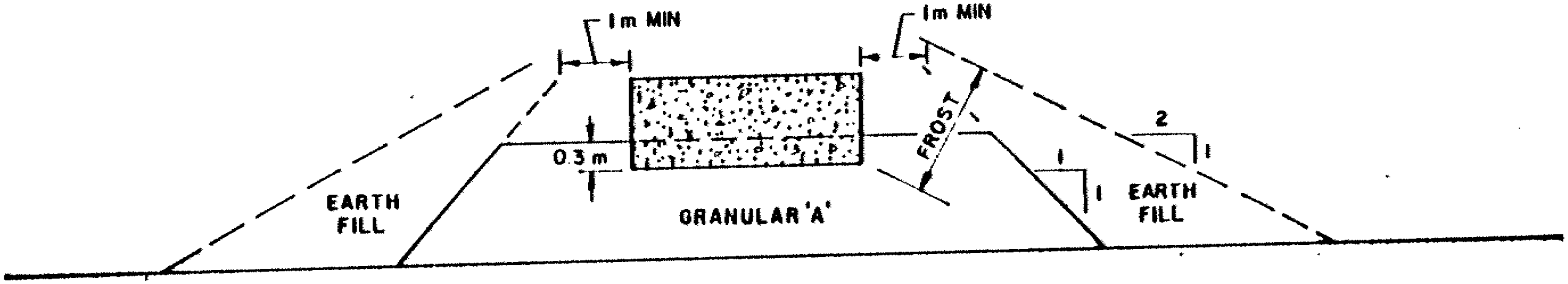
PLASTICITY CHART

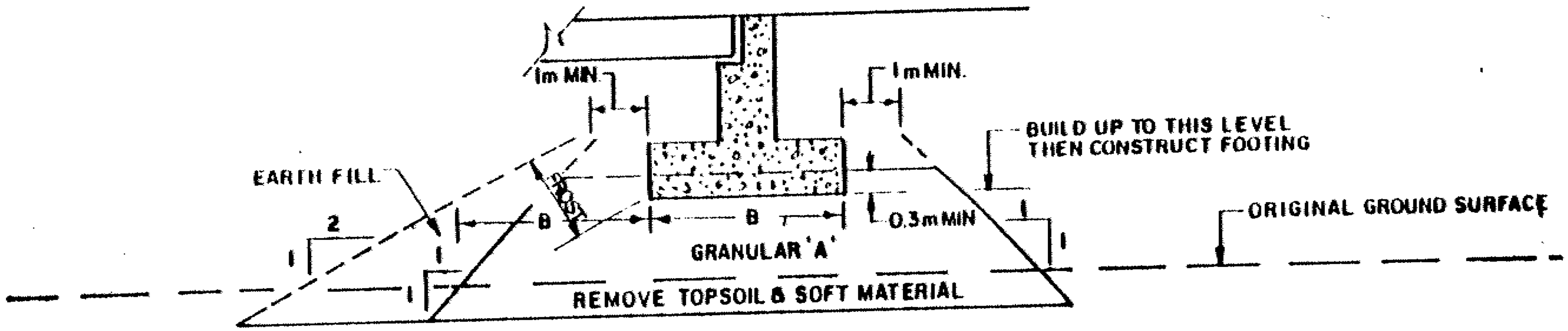
SILTY SAND TRACE GRAVEL, SOME CLAY (TILL)

FIG No 3

W P 34-81-04



CROSS - SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES :
1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
5. SOURCE M.T.C. 1982

Ministry of Transportation Ontario	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE	FIG No 4 W P 34-81-04

OVERSIZE DRAWING

METRIC

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

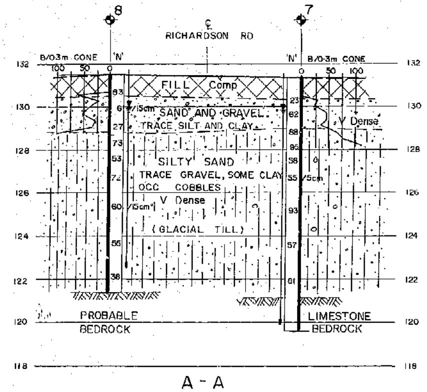
CONT No
WP No 34-81-04

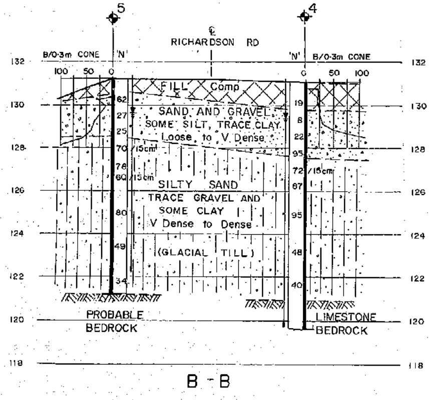
RICHARDSON ROAD

BORE HOLE LOCATIONS & SOIL STRATA

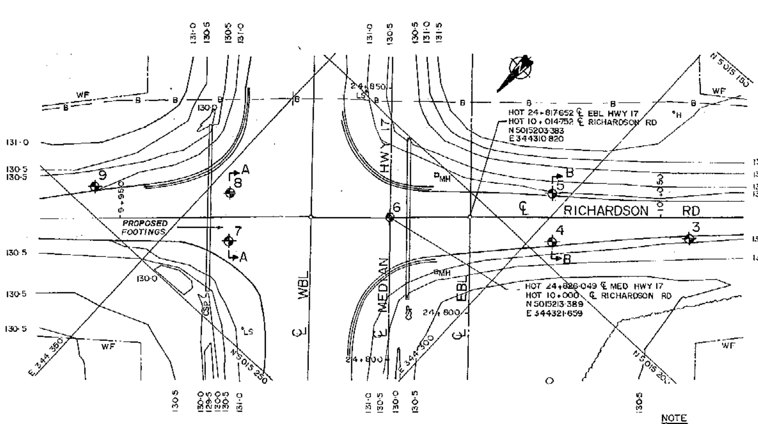
SHEET

TERRAPROBE LIMITED

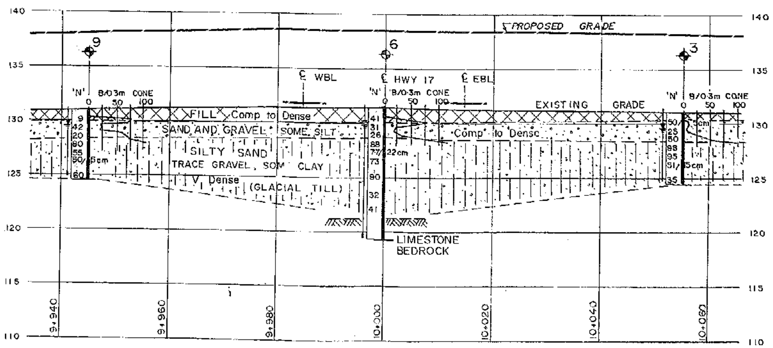




SECTIONS
SCALE

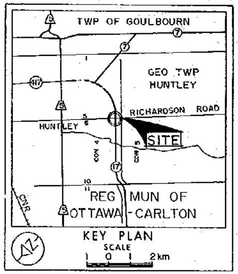


NOTE
Refer to Record of Borehole for subsoil stratigraphy for Boreholes 1 to 2 and 10 to 11



℄ PROFILE RICHARDSON ROAD

SCALE
10 0 10 20m HOR
5m 0 5 10m VERT



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N — Blows/0·3m (Std Pen Test, 475 J/blow)
- CONE — Blows/0·3m (60° Cone, 475 J/blow)
- WL at time of investigation 1989 12
- Piezometer

No	ELEVATION	STATION	OFFSET ℄ RICHARDSON
1	131·2	10+105	5·0m LT
2	131·1	10+080	4·0m LT
3	131·0	10+055	4·0m RT
4	131·0	10+030	4·5m RT
5	131·2	10+030	4·5m LT
6	131·3	10+000	
7	131·3	9+970	4·5m RT
8	131·5	9+970	4·5m LT
9	131·0	9+945	6·0m LT
10	130·7	9+920	5·0m RT
11	130·6	9+895	5·0m LT

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 31G5-177

HWY No 17			DIST 9
SUBM'D	CHECKED	DATE 1990 10 23	SITE 3-570
DRAWN	CHECKED	APPROVED	DWG 348104-A

REF No E-65-17- ; 1990 02