

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 3165-147

DIST. 9 REGION

W.P. No. 120-87-04

CONT. No. 90-36

W. O. No.

STR. SITE No.

HWY. No. 417

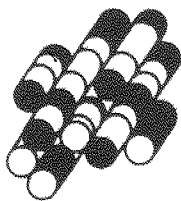
LOCATION S-W Ramp over E.B. Hwy 417 &
West to Acres Rd. Ramp (Structure #1)

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



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FOUNDATION INVESTIGATION

PROPOSED
SOUTH TO WEST RAMP OVER HIGHWAY 417 EB
STRUCTURE #1, OVERPASS
OTTAWA, ONTARIO

CONT. 90-36
W.P. 120-87-04 SITE NO. 3-52-531

DISTRICT 9, EASTERN REGION

PREPARED FOR : Ministry of Transportation, Ontario
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OUR FILE NO. 88311
OCTOBER 1988

*GEOCREG * 31G5-147*

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ABBREVIATIONS AND SYMBOLS

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Field Procedure

APPENDIX B

Embankment Stability Analysis

Embankment Settlement Analysis

BOREHOLE LOGS 1 TO 12

DWG. NO. WP1208704-A

FIGURES 1 TO 8

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, in Ottawa, Ontario.

The field investigation for the project consisted of twelve (12) boreholes at the proposed bridge pier and abutment locations and along the alignment of the proposed approach embankments.

The boreholes drilled along the east half of the proposed structure and approach generally encountered competent ground conditions, consisting of up to 0 to 4.2 m of silty sand till overlying dolostone bedrock.

The boreholes advanced at the west pier, west abutment, and along the west approach embankment, generally encountered 5.6 to 8.5 m of firm to hard silty clay underlain by compact silty sand to sandy silt (till). The glacial till was underlain by quartz sandstone bedrock at depths of 7.3 to 10.1 m.

Based on the difference in bedrock types and elevations, and review of geologic maps for the area, it appears that a geologic discontinuity, such as a fault, is present in the area.

In summary, it is considered that the east abutment, and east and central piers can be founded on shallow spread footings placed directly on the dolostone bedrock or on a raft of engineered fill. The west pier and west abutment can be founded on piles advanced to the underlying sandstone bedrock.

The ground conditions will pose few geotechnical constraints to the design and construction of the embankment fills. The firm clay soils encountered along the west approach, will permit construction of the embankment without stability concerns. However, it is estimated that up to 75 mm of settlement may occur beneath the highest portions of the fill. The primary settlement is expected to take place over a period of one to two years.

A preliminary review of seismic parameters associated with the site was carried out to consider the effect of earthquake loading on the embankments. This review indicates that a detailed analysis should be carried out in order to fully address the potential for earthquake damage to the proposed approach fills.

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 is designated as Structure 1, and will be constructed in conjunction with the Highway 416/417 interchange area in the western section of Ottawa, near Nepean.

The details of this project were discussed with Mr. Murty Devata, Mr. Tae Kim, and Mr. Tony Sangiuliano during the week of September 5, 1988 in order to define the appropriate scope of work and engineering analysis. These details were further addressed in our proposal letter dated September 8, 1988 and during subsequent telephone conversations with the above parties.

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 September 12 and 15, 1988. Twelve boreholes were drilled to depths between 1.6 and 14.6 m below existing ground surface at the locations shown on Drawing No. W.P.1208704-A. Eight of these boreholes were advanced at the proposed foundation locations while the remaining four 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 at each borehole location.

2. SITE AND GEOLOGY

The site is located at the west end of Ottawa (near Nepean), in the southwest sector of the intersection of existing Highway 417 and Acres Road.

The site is currently fairly flat farm field with occasional trees along fence lines. Portions of the site are cultivated ground, and were planted with corn.

A review of Map 1335A, Sheet 305, "Southern Ontario", Geological Survey of Ontario, indicates that the site is in a region of variable bedrock geology. It appears that the site is divided by a fault.

The site is underlain by the March Formation(interbedded quartz sandstone and sandy dolostone) west of the fault, and by the Rockcliffe Formation (silty dolostone with shale interbeds) east of the fault.

The Oxford Formation (dolostone) is also present immediately north and south of the site. The Nepean Formation (quartz sandstone) from the Lower Ordovician is also encountered south of the site.

The site is traversed by a north-south trending fault, which is a geologic structure reportedly common to the area.

A review of 'The Physiography of Southern Ontario' (Chapman & Putnam, 3rd E., 1984) indicates the Ottawa Valley to consist of clay plains interrupted by ridges of rock or sand. The Ottawa area is also known to be seismically active.

Proposed Structure No.1 is a four-span bridge about 180 m in length and about 10 m wide. Earth fill approach embankments will be constructed on the existing native ground to a maximum height of about 7 m.

The proposed structure will cross a proposed new section of Highway 417 as shown on Drawing No. WP1208704-A. The structure will connect future northbound Highway 416 to the existing westbound lanes of Highway 417. The structure will cross over the proposed eastbound lanes of Highway 417 and exit ramp for Acres Road. The alignment of Structure 1, will not interfere with existing or proposed structures.

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. WP1208704-A.

Details of the laboratory tests and field tests are summarized on the Borehole Logs and on Figures 1 to 7 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 change.

The ground surface along the west half of the proposed bridge alignment is flat and at about elevation 66 m Geodetic. The east half of the proposed alignment rises from about elevation 65 m to elevation 67.5 m Geodetic, in a series of shallow ridges.

The boreholes found a thin topsoil layer at the ground surface. The east half of the bridge and embankment alignment was underlain by 0 to 4.2 m of silty sand to sand (till) overlying dolostone bedrock.

The west half of the alignment was underlain by 5.6 to 8.5 m of firm to stiff silty clay underlain by compact silty sand to sandy silt (till). The till was underlain by sandstone bedrock at depths of 7.3 to 10.1 m.

3.1 Topsoil

At the ground surface in each of the twelve borehole locations, about 150 mm to 200 mm of black, silty topsoil was encountered. This layer and the underlying surface of the native soil were noted to be tilled as the result of use of this property as a cultivated corn field.

3.2 Centre Pier, East Pier, East Abutment, East Embankment

3.2.1 Silty Sand to Sand (Till)

Boreholes 6, 7, 10 and 11 were situated at the east abutment and along the east embankment across a series of small ridges. The boreholes encountered silty sand till directly below the topsoil to depths of 0.9 and 4.2 m, where bedrock was found.

In Boreholes 6 and 11, the till was brown and extended to the bedrock surface at depths of about 0.9 to 1.3 m (elev. 63.8 and 66.2 m).

In Boreholes 7 and 10 the till was brown to a depth of about 1.5 to 2.1 m where the colour changed to grey. Bedrock was encountered at depths of 2.1 to 4.2 m (elev. 64.2 and 62.4 m).

Grain size distribution curves for the silty sand stratum (Till) are presented on Figure 2. The till contained some gravel sizes and trace clay.

The water contents determined on samples obtained from this material, varied between 18 and 42 percent by weight in the upper 0.5 m where some mixing with the topsoil has occurred. Beneath this surficial zone, water contents were about 8 to 20 percent.

The Standard Penetration Test results (N' values) ranged between 5 and 14 (blows per 0.3 m) for the upper 0.5 m of the till which have been disturbed by farming. Deeper in the till, the N' values ranged between 14 and 37 (blows per 0.3 m) indicating a compact to dense deposit.

For design purposes, the following soil properties are estimated based on index properties of the deposit.

effective angle of internal friction, (Phi)	ϕ'	-	30 degrees
effective cohesion intercept,	c'	-	0 kPa
unit weight, (Gamma)	γ	-	22 kN / cu.m.

It is noted that the above parameters are unfactored.

3.2.1 Bedrock

Massive, fine to medium grained, light to dark grey, crystalline dolostone bedrock was exposed at the ground surface (elev. 65.2 and 65.1 m) at Boreholes 4 and 5 (centre pier and east pier).

In Boreholes 6 and 7 (east abutment) bedrock was encountered beneath the silty sand (till) at depths of about 0.9 and 2.1 m respectively (elev. 63.8 and 64.2 m).

The rock was cored for lengths of 1.3 to 2.1 m. Rock cores obtained at the pier and abutment locations indicated solid core recoveries of 100 percent and RQD's varying between 93 and 100 percent. The recovered rock cores were in a slightly weathered to sound condition.

The rock core examined had frequent calcitic filled vugs and minor sandstone beds.

Boreholes 10 and 11 (east embankment) met refusal to augering at depths of 4.2 and 1.3 m respectively (elev. 62.3 and 66.2 m). Due to the difference in elevation compared to Boreholes 6 and 7, the refusal in Boreholes 10 and 11 may represent the bedrock surface, or a boulder within the till.

3.3 West Pier, West Abutment, West Embankment

3.3.1 Silty Clay

Boreholes 1, 2, 3, 8, 9 and 12 all encountered a silty clay beneath the topsoil layer. The silty clay layer extended to a depth of approximately 1.8 m. The upper 1.0 m was found to be disturbed and/or slightly weathered due to the previous cultivation of the site for corn production and also due to natural disturbance from frost and surface water penetration. The portion of the layer between approximately 1.0 m and 1.8 m was brown changing to grey and found to be stiff to very stiff.

Water contents of the samples varied from about 30 percent to about 52 percent with an average of about 35 percent.

The following range of Atterberg Limits were measured for 3 of the silty clay samples:

	Range	Average
Liquid Limit	28 to 45 percent	39 percent
Plastic Limit	16 to 25 percent	20 percent
Plasticity Index	22 to 26 percent	24 percent

The Atterberg Limit results are presented graphically on the Borehole Logs and on Plasticity Charts (Figures 3 and 4). They indicate generally a 'CI' soil, being a clay of intermediate plasticity.

The Standard Penetration Test results within the silty clay stratum varied between 6 and 12 blows per 0.3 m.

3.3.2 Clayey Silt

Boreholes 1, 2, 3, 8, 9 and 12 all encountered a clayey silt beneath the silty clay layer. The clayey silt deposit varied in depth between 3.8 and 6.7 m. The material was grey in colour.

The majority of the borehole sampling in the clayey silt was carried out using thin-walled 75 mm diameter Shelby tubes. The samples were extruded for bulk unit weight, unconfined compressive strength testing, moisture content, Atterberg Limits, and uniaxial consolidation tests.

The clayey silt was found to be frequently interbedded with partings of silty fine sand and thin layers of fine sand and sandy silt typically 20 mm in thickness, with spacings ranging between 150 and 300 mm as observed in the Shelby Tube samples.

Figure 1 presents the results of grain size analyses on 6 samples of the clayey silt. There was about 20 to 50 percent by weight fine sand in the samples tested.

Water contents of the samples varied from about 10 percent in the fine sand layers, to about 52 percent in the clayey silt, with an average of about 34 percent.

The following range of Atterberg Limits were measured for 12 of the clayey silt samples:

	Range	Average
Liquid Limit	23 to 32 percent	28 percent
Plastic Limit	10 to 25 percent	17 percent
Plasticity Index	11 to 19 percent	15 percent

The Atterberg Limit results are presented graphically on the Borehole Logs and on Plasticity Charts (Figures 3 and 4). They indicate generally a 'CL' soil according to the Unified Soil Classification System, which is a clay or clayey silt of low plasticity.

The Standard Penetration Test results within the clayey silt stratum varied between 1 and 8 blows per 0.3 m.

In situ vane testing carried out through the clayey silt, indicated variable undrained shear strengths ranging from 45 to more than 155 kPa. The field vane shear strengths were not corrected for plasticity since the correction is small (Bjeerum, 1972) for clays with a Plasticity Index of less than 15.

Tests after remolding indicated a moderate sensitivity, with values ranging typically between 4 and 8. It is likely that the wide range and lack of trend with depth in field vane test results (see Figure 7), are a result of the numerous thin sand layers within the clayey silt.

Unconfined compressive strength tests conducted on selected samples indicated undrained shear strengths varying between 18 and 72 kPa. Much lower strengths were measured from the Shelby tube samples than by the field vane tests. This is attributed to sampling disturbance, and the sand layers which would have a greater effect on unconfined samples.

The bulk unit weights determined on shelly tube samples varied from 24.0 to 20.4 kN/cu.m. These values are relatively high for clayey silt, possibly reflecting the high sand content and high water content.

Consolidation tests were carried out using standard oedometer test methods on specimens obtained from Borehole 9, Sample 4 (2.7 m) and from Borehole 2, Sample 3 (1.9 m). The results of these tests are summarized on the void ratio vs log pressure curves presented on Figures 5 and 6 respectively. The properties are summarized below:

BH No.	SA No.	Depth (m)	w%	wp%	wl%	γ kN/cu.m	Cv 10 ⁻³ cm ² /s	Cc	Cr	σ'_p kPa	α' kPa
2	3	1.9	40	15	31	22.8	5 to 10	0.32	0.03	150	45
9	4	2.7	50	25	41	23.3	2 to 11	0.55	0.04	385	50

For design purposes, the following soil strength properties are estimated from the laboratory and field testing:

effective angle of internal friction, (Phi)	-	28 degrees
unit weight, (Gamma)	-	20 kN / cu.m.
apparent cohesion intercept (drained), c'	-	10 kPa
undrained shear strength cu-		40 kPa to >150 kPa (field vane)
undrained shear strength cu-		17 kPa to 72 kPa (unconfined, lab)

3.3.3 Sandy Silt (Glacial Till)

Beneath the clayey silt in Boreholes 1, 2, 3, 8, and 12, a silty sand to sandy silt till was encountered to depths of 7.3 to 9.8 m (elev. 55.2 to 58.8 m). This appears to be the same deposit which was encountered in Boreholes 6, 7, 10 and 11 along the central and eastern portions of site. This sandy till contains some gravel with a trace of clay, and exhibits only minor cohesion. Figure 2 shows the grain size distribution of 2 till samples.

The sandy till was grey in colour and the Standard Penetration Test results (N values) were measured between 4 and 63 blows per 0.3 m. This indicates a loose to very dense condition as was also reflected in the Dynamic Cone Test results shown on the Borehole Logs. The water contents were consistently in the order of 10 percent.

3.3.4 Sand

A uniform fine to medium sand layer was encountered in Borehole 12 from a depth of 8.2 to 10.1 m where grinding auger refusal was met. The measured N value in the sand was 38 blows per 0.3 m, indicating a dense condition. The sand was wet and had a water content of about 15 percent.

3.3.5 Bedrock

A massive, fine to medium grained, light grey, quartz sandstone was encountered in Boreholes 1, 2, and 3 at depths of 8.5 to 9.8 m (elev. 55.2 to 57.6 m). The sandstone was cored for depths of 1.5 to 6.1 m. Rock core recovery varied between 77 and 100 percent and RQD values ranged between 14 and 65 percent. These low RQD values are the result of closely spaced horizontal and vertical fractures in the recovered core. There was little weathering evident on the fracture surfaces. Small interbeds of dolostone occur at typically 10 to 12 cm spacing.

Refusal to augering was met in Boreholes 8, 9, and 12 at depths of 7.3, 8.5, and 10.1 m respectively (elev. 58.8, 57.5, and 55.8 m).

It should be noted that a rock ridge can be observed at the ground surface traversing the proposed structural alignment between Boreholes 3 and 4 and trending north-south. This suggests the presence of a geologic discontinuity such as a fault. This is further supported by the sudden increased depth in overburden and the change in rock type between Boreholes 3 and 4.

3.4 Groundwater

The groundwater levels measured October 1, 1988 are summarized below. In Boreholes 7 and 10, at the east end of the site the levels were at Elev. 64.2 m and Elev. 64.4 m respectively. Boreholes 1, 2, 3, 8, 9 and 12 advanced through the deeper overburden at the west end of site, indicated water levels between Elev. 58.9 and Elev. 64.4 m. At Borehole 1, the water level measured in the bedrock was lower than that in the overlying clayey silt. This indicates

downward drainage from the clayey silt to the rock.

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

Water levels encountered in the boreholes on the day of drilling are summarized on the Borehole Logs and as follows:

Borehole	Water Depth/Elev.
1	4.6m/61.5m
2	3.8m/62.3m
3	2.4m/63.6m
4	dry
5	dry
6	dry
7	dry
8	3.0m/63.1m
9	4.6m/61.4m
10	3.0m/63.5m
11	dry
12	4.6m/61.3m

Water levels measured in the installed standpipe type piezometers on October 1, 1988 are summarized below.

Borehole	Piezometer	Water Depth/Elev.
1	Sandstone	3.7m/62.4m
1	Clayey Silt	2.4m/63.7m
3	Sandstone	3.4m/62.6m
4	Dolostone	1.4m/63.8m
7	Dolostone	2.1m/64.2m
8	Clayey Silt	2.7m/63.4m
9	Clayey Silt	2.1m/63.9m
10	Sandy Silt (Till)	2.1m/64.4m

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.P1208704-A. . The proposed structure will be located some 70 to 130 m south of the existing Hwy 417 and will connect northbound Hwy 416 (also proposed) traffic to westbound Hwy 417. A new eastbound Hwy 417 set of lanes and an exit ramp to Acres Road, will be constructed beneath the proposed structure.

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 structures in the immediate vicinity of Structure No. 1, nor are there any proposed structures which will be influenced by the structure or the approach embankments.

4.1 East Abutment

Boreholes 6 and 7 were drilled at the location of the proposed east abutment. The boreholes encountered 100 to 150 mm of topsoil, over loose to dense silty sand to sand (till) which extended to dolostone bedrock at depths of 0.9 to 2.1 m (elev. 63.8 m and 64.2 m respectively).

The east abutment can be founded on conventional shallow spread footings placed directly on the dolostone bedrock. For design, the recommended factored bearing capacity at the Ultimate Limit State (ULS) is 3,000 kPa. This value is suggested in the Ontario Highway Bridge Code, Section 6-5.3.1 for sound bedrock. The rock is considered to be an 'unyielding' foundation base. Therefore the settlement associated with the Serviceability Limit State (SLS) Type II would be negligible, and the ULS will govern for design purposes.

The rock foundation base should be scaled of all loose surficial overburden and weathered rock and the concrete cast in tight contact to the rock surface. For cast in place concrete poured on the rock surface, the coefficient of friction against sliding may be taken as $\tan \phi$ where ϕ is the angle of friction between the concrete and the rock. For ϕ equal to about 35 degrees the factored coefficient of friction against sliding at Ultimate Limit States would be approximately 0.6 . If sufficient resistance to sliding is not attainable, the use of dowels drilled into the rock may be considered.

Alternatively, the east abutment can be founded on a raft of engineered fill placed in maximum 150 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 5 m high, with side slopes and distances as shown on Figure 8.

The fill material must comprise approved MTO Granular 'A'. 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 II is 350 kPa.

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.

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 a friction angle of 35 degrees.

The soil cover to provide sufficient frost protection in the Ottawa area is about 1.8 m (CFEM, 2nd, 1985). Based on the sound bedrock at this site, it is considered that foundations can be placed on the bedrock without requiring a minimum frost cover, since the rock is considered to be non-frost susceptible. It is important to ensure that water cannot migrate beneath the footing where, if subjected to freezing conditions, might heave the footing. This can usually be avoided by proper foundation preparation and a clean and tight contact, and proper sealing and drainage between the footing concrete and the bedrock surface. If the footings are placed on a raft of engineered fill then the 1.8 m of soil cover will be required.

4.2 East Pier and Centre Pier

At Boreholes 4 (centre pier) and 5 (east pier) the dolostone bedrock was found at the ground surface (elev. 65.2 m and 65.1 m respectively). The east and centre piers may be founded on conventional spread footings directly on the sound dolostone bedrock. The recommended capacities and conditions given for the east abutment and noted above, will also apply to the east and centre piers.

4.3 West Abutment

Boreholes 1 and 2 at the west abutment location, encountered a thin topsoil layer over firm to stiff silty clay and clayey silt layers which extended to depths of 5.4 to 6.1 m. The cohesive layers were underlain by loose to compact silty sand to sandy silt till to depths of 8.5 and 9.1 m where sandstone bedrock was encountered (elev. 57.6 m and 57.1 m respectively).

It is recommended that the west abutment be founded on low displacement steel H-piles, driven to refusal in the sandstone bedrock at approximate Elev. 57.1 to 57.6 m.

As with the dolostone bedrock at the east abutment, the sandstone bedrock may be considered "unyielding" and therefore the Ultimate Limit State design governs.

The maximum foundation loading applied to the pile section must not exceed 0.3 times the yield stress for steel. Therefore, for a typical steel yield stress of 300 MPa, the maximum pile loading would be about 90 MPa. A pile load test should be considered for driven piles in the rock, to confirm ultimate pile capacity, if this data is not already available.

Piles should be driven to 25 blows per 25 mm of penetration in the sandstone bedrock. It is recommended that careful monitoring be carried out during the driving to avoid excessive driving and possible damage to the piles.

Although cobbles were encountered only in Borehole 12, it is our understanding that the lower silty sand (till) has been characterized with frequent cobbles, during other investigations in the area. A driving shoe may be required if resistance is encountered. In addition, the final elevation of the piles should be carefully monitored to ensure that the bedrock has been reached, as compared with the boreholes.

The maximum driving energy used for this project will be dependent on the type and size of pile selected. Driving restrictions will be dependent on the above and on the driving resistance encountered on site.

The following design capacities for two typical HP sections are presented:

Pile/Structure	Factored Axial Capacity ULS	Factored Axial Capacity SLS
<u>West Abutment</u>		
HP 310 x 79	1050 kN	800 kN (with negative skin friction)
HP 310 x 110	1450 kN	1050 kN
<u>West Pier</u>		
HP 310 x 79	1150 kN	900 kN (without skin friction)
HP 310 x 110	1600 kN	1150 kN

The negative skin friction (estimated between 100 kN and 150 kN) is the result of the anticipated settlement of the embankment foundation soil. This settlement will apply a downward load to the pile section and therefore reduce the available capacity to support the structural loads. It is anticipated that the settlement of the embankment foundation will occur over a period of 1 to 2 years. The negative skin friction values have been estimated assuming a pile/soil adhesion of 40 kPa and a consolidating layer thickness of 6 metres which are appropriate for the west abutment.

If the construction schedule can permit placement of the west embankment

and total settlement of the embankment foundation prior to pile installation, then the factored load capacities (SLS) of 1150 kN or 900 kN may be used for design.

The final pile selection and proposed construction schedule should be discussed with the geotechnical engineer prior to tendering to ensure the design capacity is appropriate.

The site is also in an active seismic region and, therefore, the bridge must be designed with seismic considerations. For design purposes, this site is in Seismic Zone 3 with a peak horizontal ground acceleration of 0.2 g and a peak horizontal velocity of 0.098 m/s (CFEM, 2nd, 1985).

4.4 West Pier

At Boreholes 3 and 12, sandstone bedrock was encountered at depths of about 9.8 m and 10.1 m (elev. 55.2 m and 55.8 m respectively). The overburden consisted of firm to stiff silty clay to clayey silt, over sand to sandy silt.

The west pier can be founded on piles driven to refusal on the underlying sandstone bedrock. The recommended capacities and driving details as discussed previously for the west abutment, also apply for the west pier. It is our understanding however, that no fill will be placed at the pier location. In this case, no downward drag capacity reduction is required.

4.5 Excavations

Excavation on the site are expected to depths of up to 2 m for topsoil stripping, subgrade preparation for the approach embankment foundations, spread footings, and for pile caps. Although the groundwater table was encountered at depths of 1.4 to 3.7 m, groundwater is not expected to be a problem. Minor seepage volumes are expected due to the low permeability. Some minor seepage from the overburden/bedrock interface at the east abutment may also occur. It is anticipated that groundwater control at the site can be handled with conventional sump pumping.

Temporary unbraced excavations through the silty sand till encountered along the east/south approach are expected to remain stable at inclinations of about 1 to 1 (horizontal to vertical). Slightly flatter side slopes may be required if excavations are carried deeper than 2 m in the clayey silt.

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.

4.6 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 parameters may be used in calculation of lateral earth pressures, in accordance with the OHBDC:

	Granular 'A'	Granular 'B'
Effective Angle of Internal Friction (Phi), degrees	35	30
Unit Weight (Gamma), kN/cu.m.	22.8	21.2
Active Earth Pressure Coefficient, Ka	0.27	0.33
At rest Earth Pressure Coefficient, Ko	0.43	0.50

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 SPMDD in lifts not exceeding 200 mm. It is important to achieve adequate compaction to minimize future settlement of backfill behind the abutments.

4.7 Embankment Fill Placement

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 SPMDD.

The native silty clay and clayey silt soils encountered in the borings are generally too wet to be suitable for use as compacted fill for the approach embankments. The native silty sand (till) is suitable for use as embankment fill. The approach embankment may be constructed with an approved organic-free earth fill placed in maximum 200 mm thick loose lifts and uniformly compacted to a minimum of 95 percent Standard Proctor Maximum Dry Density (SPMDD). The moisture content of the fill material should be maintained near the optimum

moisture content for compaction, to minimize the effort and to minimize post construction settlement of the fill.

Post construction settlement of the embankment fill is estimated to be on the order of about 1 percent of the fill height. This is expected to occur within the first year after filling.

4.8 Embankment Settlement

4.8.1 East Embankment

The embankment fill near the abutment of the east approach will be resting directly on limestone bedrock. Consolidation and settlement of the bedrock will be insignificant. Further away from the abutment, the east embankment height will diminish, and the foundation material will be silty sand (till) between 1 and 4 m deep.

The 7 m height of fill represents a vertical loading of about 150 kPa. Under an assumed worst case of 7 m of embankment fill resting on 4 m of silty sand till, calculations indicate the foundation soil can be expected to settle up to 25 mm. Most of this settlement should be elastic in nature, and would occur within the duration of construction. The fill itself may be anticipated to settle in the order of 1 percent of the total height. This also is anticipated to occur 3 to 6 months following construction.

4.8.2 West Embankment

The borehole data indicates the west embankment will rest on firm to stiff, native silty clay which extends to depths of 5.4 to 8.5 m (elev. 57.5 to 60.7 m). The silty clay and clayey silt layers were underlain by silty sand to sandy silt (till), or bedrock.

Near the west abutment, the fill height will be about 7 m (loading of about 150 kPa) and the underlying silty clay/clayey silt about 6 m deep. Away from the abutment near Borehole 9, the embankment fill height will be about 4 m (loading of about 90 kPa) and the underlying silty clay/clayey silt about 8.5 m deep.

A detailed analysis of consolidation settlement was carried out for the west approach embankment. The results of the laboratory testing were considered, to determine the soil properties utilized in the analysis.

The laboratory consolidation test results are summarized below:

	BH 2 SA 3	BH 9 SA 4
$e(o)$ =	0.97	1.15
Ov' =	45 kPa	50 kPa
Op' =	150 kPa	385 kPa
Cv =	$(5 \text{ to } 10) \times 10^{-3} \text{ cm}^2/\text{s}$	$(2 \text{ to } 11) \times 10^{-3} \text{ cm}^2/\text{s}$
Cc =	0.32	0.55
Cr =	0.03	0.04

The details of the settlement analysis and the results are summarized in Appendix B.

In summary, the analyses indicate that the foundation soils may settle 80 mm (west abutment) and 75 mm (with 4 m fill) as the result of primary consolidation of the silty clay and clayey silt layers. The embankment fill itself will also settle an estimated 1 percent of the fill height.

An estimate of the elastic (immediate) settlement of the embankment foundation would see approximately 60 mm of settlement at the west abutment and 35 mm at Borehole 9 (ie: 4 m of fill). This is expected to occur during construction.

The time required for primary consolidation depends greatly on the assumed drainage conditions of the silty clay and clayey silt. This is complicated by the presence of fine sand partings and thin layers which were found throughout the clayey silt deposit. The sand layers are typically 20 mm in thickness and spaced at 150 mm to 300 mm intervals.

The time for essential completion of primary consolidation was calculated using three assumptions regarding the drainage boundary conditions in the silty clay/clayey silt, as follows:

	<u>Time in Months</u>	
	<u>At West Abutment</u>	<u>At Borehole 9</u>
a) drainage to the ground surface and to the bedrock or till	80 to 350	180 to 700
b) sand layers within the clayey silt, at spacings of 1.5 m	5 to 24	6 to 24
c) sand layers in the clayey silt, at spacings of 300mm	0.2 to 1	0.2 to 1

Estimates on the time required for the foundation subgrade to settle range between less than a month to thirty years. It is considered that the first case (no sand layers) is unrealistic based on the observed sand layers in the borehole samples. The actual field performance will likely be somewhere between the next two cases since sand layers are present in the clayey silt.

For preliminary design purposes a one to two year period for completion of primary consolidation should be anticipated. In this regard, it is recommended that the west embankment be constructed as far ahead in the bridge construction schedule as possible, to minimize post construction settlement. The settlement be monitored in order to identify completion of primary consolidation.

Alternatively, the consolidation settlement can also be accommodated by repair and maintenance as it occurs.

4.9 Embankment Stability

MTO has indicated that up to 7 m of approach fill will be placed at the abutments. Based on the borehole data, the east embankment will be constructed directly on bedrock or shallow depths of competent native silty sand (till). Failure through the till or rock foundation is not a concern due to their competent nature.

The west approach embankment will rest on 5 to 8.5 m of firm to very stiff native silty clay to clayey silt. Due to the less competent condition of the silty clay/clayey silt, foundation failure was examined in detail.

From a stability consideration, the 2 to 1 side slope inclination of the fill was examined for hypothetical slope failure surfaces through the fill only, and through the fill and foundation soil. It was assumed that the embankment fill would consist of properly compacted clean, cohesive earth fill with a minimum undrained shear strength of 80 kPa. The use of an imported free draining granular fill for the embankment construction was also examined.

The details of the slope stability analyses and the results are summarized in Appendix B. As an initial step in our analysis, the Simplified Bishop's slope stability analysis method was used to calculate the Factors of Safety for a grid of radii for circular failure surfaces. These were analyzed to identify the most critical failure circle(s). This failure surface was further checked using a Simplified Janbu method and refined using a Sarma type program which included the effects of tension cracks.

A non circular failure surface was also analyzed with the Sarma method. The analyses were carried out for the undrained (short term) case since this is the most critical case for stability of the embankment foundation. The long term (drained) stability of the embankment will be greater, as the result of improvement of soil strength due to consolidation. A combination of assumed soil parameters were used in the analyses as summarized in Appendix B on Figure B-1.

In addition, a drained analysis was carried out for several critical failure surfaces using the Sarma program. These results were used in conjunction with the undrained analyses to permit a simplified analysis for earthquake loading on the proposed embankments. The earthquake loading evaluation will be discussed in a subsequent section.

In summary, the analyses indicate the following minimum Factors of Safety for the most likely soil strength properties. These properties are based on a 2m thick crust with an undrained shear strength of 80 kPa and the underlying clayey silt with an undrained shear strength of 40 kPa. The cases of the embankment constructed with clayey fill and with sand fill were examined with introduction

of tension cracks in all cases. A unit weight of 20 kN/cu m, 22 kN/cu m and 18 kN/cu m were assumed for the clayey foundation soils, clay fill and sand fill respectively.

	UNDRAINED ANALYSIS	
	<u>Clay Fill</u>	<u>Sand Fill</u>

Factor of Safety	1.4	1.7
------------------	-----	-----

Provided the embankment fill is constructed with the proposed 2 to 1 (horizontal to vertical) side slopes, and in the manner discussed in the previous subsection, then no slope stability problems are anticipated.

The side slopes should be vegetated to minimize surface erosion and final grading, both at the crest and toe, should be such as to control surface runoff away from the slopes, or to adequately filtered channels.

4.10 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.

For the purpose of this study, a simplified analysis of the effects of earthquake loading on the embankments was undertaken. The analysis was conducted using the Sarma Method; which provides the critical acceleration of a sliding mass subjected to earthquake loading. The critical acceleration is the horizontal loading (expressed as a percentage of static gravity loading, or "g") which will result in a Factor of Safety of 1.0. The critical acceleration is assumed to act at the centroid of the sliding mass.

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.

There are several methods of calculating the distribution of horizontal acceleration in the embankment as the result of earthquake loading, including that described by Makdisi & Seed, (1979). These methods are based on the configuration of the embankment, the dynamic or damping characteristics of the soil and the characteristics of the earthquake. These soil characteristics are difficult to assess and were not obtained for this study.

However, it is noted that the centroid of the critical static failure surface is situated at an elevation which is close to the existing ground surface. Therefore, it was considered that a simplified pseudo-dynamic analysis could be

undertaken by comparing the critical acceleration obtained from the Sarma analysis, to an assumed maximum acceleration of 0.2g at the ground level.

This analysis was conducted using the critical slip surfaces identified in the static analysis. The undrained case with shear strengths of 80 and 40 kPa for the silty clay and clayey silt was considered the most conservative, since this does not account for consolidation and improved strength of the soil beneath the embankment. In addition, the study was also conducted for the drained case using drained soil parameters of $\phi' = 25$ degrees and $c' = 10$ kPa for the clayey stratum. These parameters were estimated from the index properties of the soil.

Considering the low permeability and clayey nature of the native soils, it was assumed that there would be no significant increase in pore pressure as the result of cyclic loading of the soil.

The results of the analysis which were carried out assuming a sand fill embankment, are summarized below:

Failure Surface	Soil Parameters	Critical Accel	Design Accel
shallow foundation failure	undrained cu = 80 kPa crust cu = 40 kPa	0.22g	0.2g
	drained c' = 10 kPa, $\phi' = 25$ sand fill- $\phi' = 30$	0.40g	0.2g

In summary, the results indicate that shear mobilization of the embankment foundation is likely. The embankment appears to be stable against foundation failure for the parameters assumed in the drained case.

It is considered that the drained case will be representative of actual embankment performance during the earthquake and that the embankment is adequately stable against foundation failure for the design earthquake loading.

The critical acceleration values defined for a shallow slip surface within the embankment are summarized below.

Failure Surface	Soil Parameters	Critical Accel	Design Accel
	drained $\phi' = 30^\circ$, c' = 0	0.27g	0.2g (at ground level)

Since the centroid of the sliding mass for the shallow slip surface within the embankment is considerably above the ground surface, the critical acceleration experienced during the earthquake will be in excess of the ground acceleration of 0.2g. Accelerations at the embankment crest are expected to approach 0.4g at the crest. Based on this assumption, it is evident that the embankment itself may be subject to surficial large shear mobilization or

deformation during the earthquake loading. This can be accommodated by:

- 1) accepting that shallow embankment shear mobilization may occur during the design earthquake and would require repair of the embankment, or
- 2) providing flatter side slopes or stabilizing berms to the embankment to improve stability.


If the first alternative cannot be accepted, then the embankment should be analyzed by a more rigorous method. The results of this analysis can then be used to assess the requirements for possible design changes to improve seismic stability.


It should be recognized that bridge footings placed on engineered fill constructed within the embankment may be subject to higher seismic loadings than footings constructed at the ground level. The stability of the engineered fill and the footings under seismic loading should be considered.

It should be noted that the above analysis is a preliminary and simplified assessment of the seismic effects. If more specific details are required, or if the seismic effects are considered significant in the final design, then further investigation and analysis should be conducted.

Respectfully submitted,

TERRAPROBE LIMITED


Kirk R. Johnson, P. Eng.


Michael Tanos, P. Eng.



EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS N_c .

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON "A" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

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

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCK QUALITY: 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 DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4" 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	2"	2" - 12"	1' - 3'	3' - 10'	> 10'
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAXIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER THEY ARE CONSOLIDATED (C) OR NOT (U) ISOTROPICALLY (I) OR NOT (A) AND SHEARED DRAINED (D) OR UNDRAINED (U) WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS) EG. $\bar{C}U$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

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

EARTH PRESSURE TERMS

μ COEFFICIENT OF FRICTION
 δ ANGLE OF WALL FRICTION
 k_o COEFFICIENT OF EARTH PRESSURE AT REST
 k_A COEFFICIENT OF ACTIVE EARTH PRESSURE
 k_P COEFFICIENT OF PASSIVE EARTH PRESSURE
 i ANGLE OF INCLINATION OF SURCHARGE
 w SLOPE ANGLE-BACKFACE OF WALL
 β ANGLE OF SLOPE
 N_q, N_c BEARING CAPACITY FACTORS
 D_f DEPTH OF FOOTING
 B, L FOOTING DIMENSIONS

INDEX PROPERTIES

γ UNIT WEIGHT OF SOIL (BULK DENSITY)
 γ_w UNIT WEIGHT OF WATER
 γ_d UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
 γ' UNIT WEIGHT OF SUBMERGED SOIL
 G_s SPECIFIC GRAVITY OF SOLIDS
 e VOIDS RATIO
 e_o INITIAL VOIDS RATIO
 e_{max} e IN LOOSEST STATE
 e_{min} e IN DENSEST STATE
 D_r RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
 n POROSITY
 w WATER CONTENT
 w_L LIQUID LIMIT
 w_p PLASTIC LIMIT
 w_s SHRINKAGE LIMIT
 I_p PLASTICITY INDEX = $w_L - w_p$
 I_L LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
 I_c CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
 A_c ACTIVITY = $\frac{I_p \text{ of soil}}{2.2 \mu \text{m Soil Fraction}}$
 O_m ORGANIC MATTER CONTENT
 S_r DEGREE OF SATURATION
 S SENSITIVITY = $\frac{S_u \text{ (undisturbed)}}{S_u \text{ (remoulded)}}$

STRENGTH PARAMETERS

ϕ ANGLE OF SHEARING RESISTANCE
 τ_f PEAK SHEAR STRENGTH
 τ_R RESIDUAL SHEAR STRENGTH
 c COHESION INTERCEPT
 $\sigma_1, \sigma_2, \sigma_3$ NORMAL PRINCIPAL STRESSES
 u PORE WATER PRESSURE
 u_e EXCESS u
 r_u PORE PRESSURE RATIO
 q_u UNCONFINED COMPRESSIVE STRENGTH
 s_u UNDRAINED SHEAR STRENGTH
 ϵ LINEAR STRAIN
 γ SHEAR STRAIN
 ν POISSON'S RATIO
 E MODULUS OF ELASTICITY
 G MODULUS OF SHEAR DEFORMATION
 k_s MODULUS OF SUBGRADE REACTION
 m, n STABILITY COEFFICIENTS
 A, B PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS:
 σ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE;
 σ'_n = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

h HYDRAULIC HEAD OR POTENTIAL
 q RATE OF DISCHARGE
 v VELOCITY OF FLOW
 i HYDRAULIC GRADIENT
 j SEEPAGE FORCE PER UNIT VOLUME
 η COEFFICIENT OF VISCOSITY
 k COEFFICIENT OF HYDRAULIC CONDUCTIVITY
 k_h k IN HORIZONTAL DIRECTION
 k_v k IN VERTICAL DIRECTION
 α_v COEFFICIENT OF VOLUME CHANGE
 c_v COEFFICIENT OF CONSOLIDATION
 C_c COMPRESSION INDEX
 C_r RECOMPRESSION INDEX
 d DRAINAGE PATH DISTANCE
 T_v TIME FACTOR
 U DEGREE OF CONSOLIDATION
 O_c OVERCONSOLIDATION RATIO (OCR)

APPENDIX A

Field Work

APPENDIX - A

FIELD PROCEDURE

The field investigation for this project was conducted between September 12 and 15, 1988, when 12 boreholes were advanced to depths of 1.6 to 14.6 m below existing grades, at the locations shown on Drawing WP1208704-A. The drilling was conducted using machinery supplied and operated by Marathon Drilling Company Limited of Ottawa, Ontario. The drilling operations were directed and supervised by Mr. Renato Pasqualoni, B.A.Sc., Terraprobe Limited.

The boreholes were put down in the vicinity of the proposed piers and abutments for the proposed bridge. In addition, 2 boreholes were advanced along the centre alignment of the proposed west and the east approach embankments.

The borings were put down using a truck-mounted CME 55 power auger or truck-mounted diamond drill. Split-spoon samples of the overburden materials were obtained in conjunction with thin walled tube sampling and field vane testing as detailed on the Borehole Logs and Sections. Dynamic Cone Tests were also carried out at the majority of the Boreholes. All samples obtained in this investigation were either sealed in glass jars in the case of split-spoon samples, or sealed in waxed thin-walled Shelby tube samples and transported to our laboratory for detailed inspection and testing.

Standpipe type piezometers were sealed into selected boreholes where overburden was encountered, 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.

The locations of the borings were determined by measuring relative to the survey stakes placed and marked by Ministry of Transportation, Ontario, representatives. The ground surface elevations at the borehole locations were determined and provided by MTO representatives subsequent to the investigation.

The site was revisited on October 1 and 2, 1988 to measure groundwater levels in the installed piezometers and to provide additional site information.

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, Atterberg Limits, unit weights, unconfined compressive strength, and Oedometer tests. The results of the testing are presented on the Borehole Logs and on Figures 1 to 7.

Based on the information obtained during the field investigation and during the subsequent laboratory testing, engineering analysis was then carried out to study the slope stability characteristics of the proposed embankments and also the settlement behaviour of the proposed embankments.

APPENDIX B

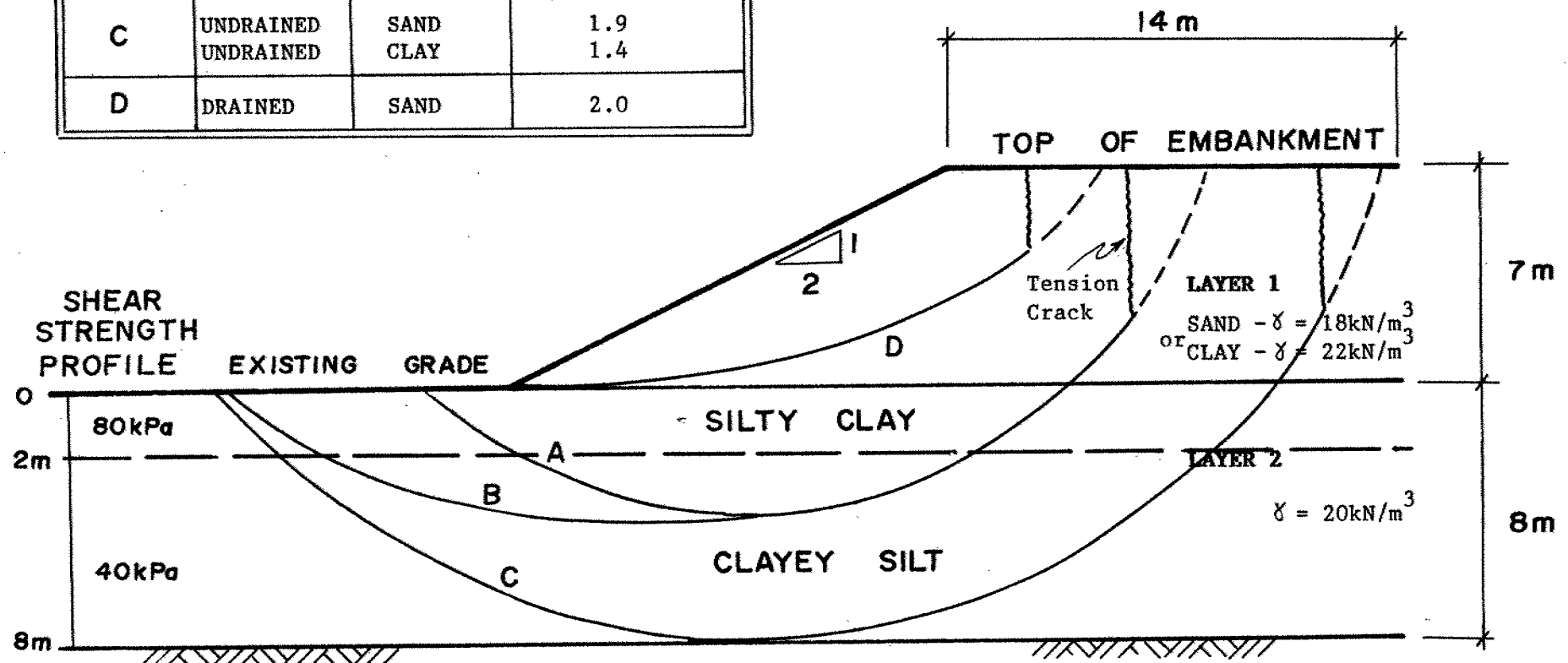
Embankment Stability Analysis

Embankment Settlement Analysis

FAILURE	ANALYSIS	FILL TYPE	FACTOR OF SAFETY
A	UNDRAINED	SAND	1.7
	UNDRAINED	CLAY	1.4
	DRAINED	SAND	2.8
B	UNDRAINED	SAND	2.0
	UNDRAINED	CLAY	1.7
C	UNDRAINED	SAND	1.9
	UNDRAINED	CLAY	1.4
D	DRAINED	SAND	2.0

NOTE: 1. Sarma Analysis used which includes tension crack.

2. For drained analysis, sand fill, Failure Surface A, $\phi=25^\circ$ and $c'=10\text{kPa}$ for Layer 2.

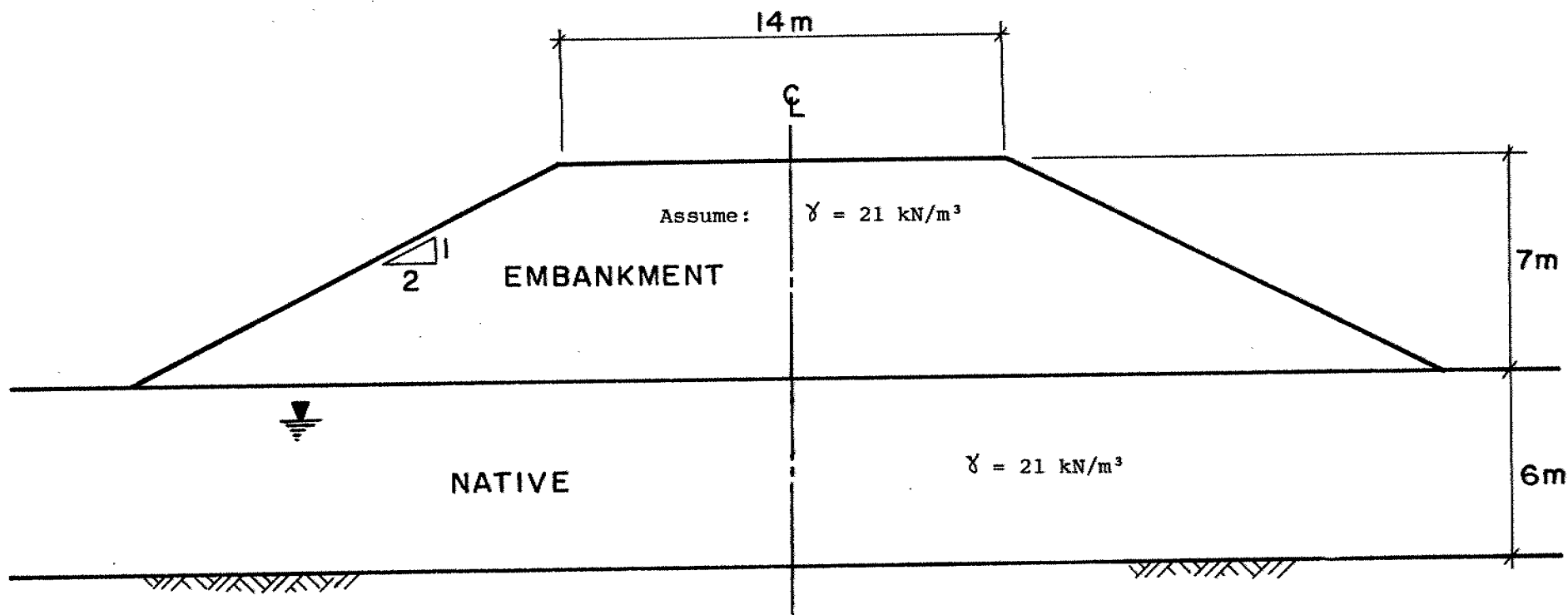


Ministry of
Transportation

SLOPE STABILITY ANALYSIS

FIG No B-1

W P 120-87-04



Properties
of Native :

$$C_R = 0.03$$

$$e_o = 1.0$$

$$\sigma'_p = 150 \text{ kPa}$$

Settlement of
Native at Q_L = 80mm

Note: Refer to Text and Figures 5 and 6

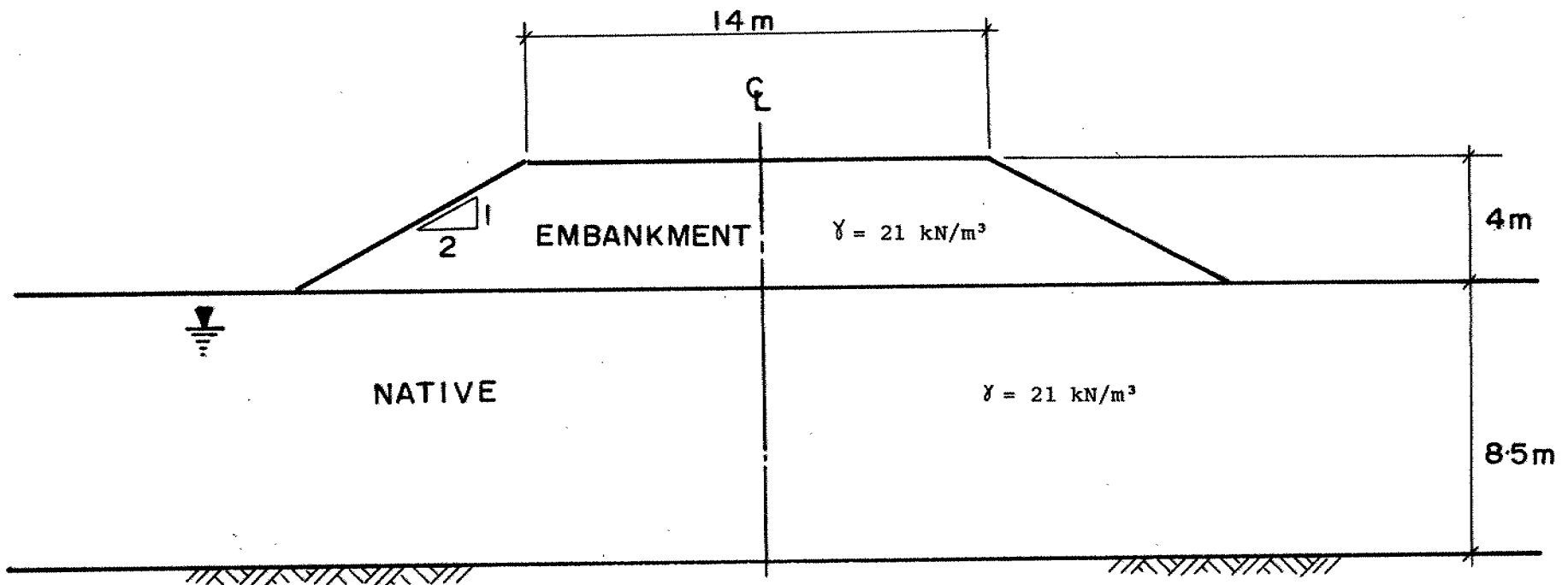


Ministry of
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Ontario

EMBANKMENT SETTLEMENT
AT WEST ABUTMENT

FIG No B-2

W P 120-87-04



Properties:
of Native

$$C_R = 0.03$$

$$e_o = 1.0$$

$$\sigma'_p = 150 \text{ kPa}$$

Settlement of
Native at G_L = 70mm

Note: Refer to Text and Figures 5 and 6



Ontario

Ministry of
Transportation

EMBANKMENT SETTLEMENT
BH 9 (ie. 4m EMBANKMENT HEIGHT)

FIG No B-3

W P 120-87-04



RECORD OF BOREHOLE No 1

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 540; E 358 146 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger, Bx Rock Core & Cone Test COMPILED BY KJ
DATUM Geodetic DATE September 12, 1988 CHECKED BY MT

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					
66.1	Ground Level																
65.9	Topsoil																
0.2	Silty Clay, some sand, very stiff brown becoming grey at approximately 64.6 m		1	SS	6	Rea											
			2	TW	PH		65									23.8	
64.3																	
1.8	Clayey Silt, with interbedded silty sand and sandy silt layers Firm to stiff Grey		3	TW	PH	Water level Oct 1/88										23.5	0 23 67 10
						Water level Oct 1/88											
			4	SS	1	Sept 12/88											
			5	SS	1												
60.0																	
6.1	Silty Sand to Sandy Silt, some gravel, occasional cobbles, trace clay Compact to Loose Grey (Glacial Till)		6	SS	20		60										
			7	SS	7												
57.6																	
8.5	Bedrock Sandstone, slightly to moderately weathered; very poor to poor Grey and White		8	RC Bx	77% RQD 14%	Seal											
			9	RC Bx	100% RQD 27%												
			10	RC Bx	100% RQD 31%												
			11	RC Bx	100% RQD 21%												
51.5																	
14.6	End of Borehole																

+3, x⁵: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 2

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 545; E 358 147 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger, Bx Rock Core & Cone Test COMPILED BY KJ
DATUM Geodetic DATE September 13, 1988 CHECKED BY MT

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					
66.1	Ground Level																
0.0	Topsoil																
0.15	Silty Clay, some sand, occasional sand partings Firm to Stiff Brown, becoming grey at approximately 64.6m		1	SS	7												
			2	SS	10												
64.3			3	TW	PH												
1.8	Clayey Silt, with imbedded silty sand and sandy silt layers firm to stiff Grey																
			4	SS	4												
			5	TW	PH												
60.7	Silty Sand to Sandy Silt, some gravel, trace clay, Loose to Compact Grey (Glacial Till)		6	SS	4												
			7	SS	13												
			8	SS	15												
57.1	Bedrock Sandstone, slightly weathered, very poor Grey and White		RC		82%												
9.1			9	Bx	RQD 18%												
55.5																	
10.6	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15
10
5
[%] STRAIN AT FAILURE



RECORD OF BOREHOLE No 3

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 530; E 358 180 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger, Bx Rock Core & Cone Test COMPILED BY KJ
DATUM Geodetic DATE September 13, 1988 CHECKED BY MT

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	x LAB VANE	20						40	60	20
66.0	Ground Level						20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL				
65.8	Topsoil - 200 mm																			
0.2	Silty Clay, some sand, occasional sand partings Firm to Very Stiff Brown		1	SS	11	Seal										24.1				
			2	TW	PH															
64.2	Clayey Silt, with interbedded silty sand and sandy silt layers firm to very stiff brown becoming grey at approximately 63.9m		3	TW	PH	Sept 13/88										23.9	0 17 59 24			
1.8																				
			4	TW	PH	Water level Oct 1/88										21.2				
			5	SS	8															
60.4																				
5.6	Silty Sand and Gravel trace clay Compact Grey		6	SS	12															
58.6																				
7.4	Silty Sand to Sandy Silt, some gravel, trace clay Very Dense Grey (Glacial Till)		7	SS	63															
			8	SS	43															
55.2																				
9.8	Bedrock																			
	Sandstone, slightly weathered, fair		9	RC Bx	100% RQD 65%															
	Grey and White																			
54.6																				
11.4	End of Borehole																			

METRIC

W P 120-87-04

LOCATION Co-ordinates N 5 022 485; E 358 270

ORIGINATED BY **RP**

DIST 9 HWY 417

BOREHOLE TYPE..... Nx Rock Core

COMPILED BY RJ

DATUM Geodetic

DATE September 13, 1988

CHECKED BY MT

[illegible]

⁺₃, ⁺₅: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 6

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 470; E 358 302
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, Nx Rock Core
 DATUM Geodetic DATE September 13, 1988
 ORIGINATED BY RP
 COMPILED BY KJ
 CHECKED BY MT

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	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100						
64.7	Ground Level																	
0.0	Topsoil																	
0.15	Silty Sand, some gravel, trace clay (Glacial Till)		1	SS	14													
63.8	Compact Brown																	
0.9	Bedrock Dolostone, frequent calcitic vugs light grey		2	RC Nx	100% RQD 100%													
62.5																		
2.2	End of Borehole																	

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 7

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 462; E 358 295 ORIGINATED BY RP
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, Nx Rock Core COMPILED BY KJ
 DATUM Geodetic DATE September 12, 1988 CHECKED BY MT

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
66.3	Ground Level															
0.0	Topsoil															
0.1	Silty Sand, some gravel, trace clay		1	SS	9											
	Compact to Dense		2	SS	37											
	Brown becoming grey at 64.8 m (Glacial Till)		3	SS	30											
64.2																
2.1	Bedrock															
	Dolostone, frequent calcareous vugs Light Grey		4	RC Nx	100% RQD 90%											
			5	RC Nx	100% RQD 100%											
61.1																
5.2	End of Borehole															

+3, +5: Numbers refer to
Sensitivity

20
15 + 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 8

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 553; E 358 080 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, Cone Test COMPILED BY KJ
DATUM Geodetic DATE September 14, 1988 CHECKED BY MT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
66.1	Ground Level																
65.9	Topsoil																
0.2	Silty Clay, some sand, occasional sand partings		1	SS	10		Seal							0			
	Firm to Very Stiff Brown becoming grey at approximately 64.7m		2	SS	8		65							0			
64.3	Clayey Silt, with interbedded silty sand and sandy silt layers		3	SS	2									0			
1.8	Firm to Very Stiff grey		4	TW	PH		Water level Oct 1, 88 Sept 4/88									23.3	0 28 59 13
			5	TW	PH												
			6	TW	PH												
60.0	Sandy Silt, some gravel, trace clay		7	SS	54		60									22.5	
6.1	Very Dense Grey																
58.8	(Glacial Till)																
7.3	End of Borehole Auger Refusal on probable bedrock																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 9

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 560; E 358 000 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, Cone Test COMPILED BY KJ
DATUM Geodetic DATE September 14, 1988 CHECKED BY MT

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					
66.0	Ground Level																
65.8	Topsoil																
0.2	Silty Clay, some sand, occasional sand partings		1	SS	12		Seal							0			
	Firm to Very Stiff		2	SS	6		65							0			
64.2	Brown becoming grey at approximately 64.5m		3	SS	1									0			
1.8	Clayey Silt, with interbedded sandy silt and silty sand layers		4	TW	PH									0		23.3	0 20 53 27
	Firm to Very Stiff grey		5	SS	2									0			
			6	SS	2		Sept 14/88							0			
			7	TW	PH		60							0		23.5	
			8	SS	1		Water level Oct 1/88							0			
57.5																	
8.5	End of Borehole Auger Refusal on probable bedrock																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 10

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 436; E 358 330 ORIGINATED BY RP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY KJ
DATUM Geodetic DATE September 14, 1988 CHECKED BY MT

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 11

METRIC

W P 120-87-04 LOCATION Co-ordinates N 5 022 400; E 358 360 ORIGINATED BY RP
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY KJ
 DATUM Geodetic DATE September 14, 1988 CHECKED BY MT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
67.5	Ground Level																
0.0	Topsoil																
0.15	Sandy Silt, some gravel, trace clay Compact Brown (Glacial Till)		1	SS	6									0			
66.2			2	SS	15									0			
1.3	End of Borehole Auger Refusal on probable bedrock																
							65										

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15 ± 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 12

METRIC

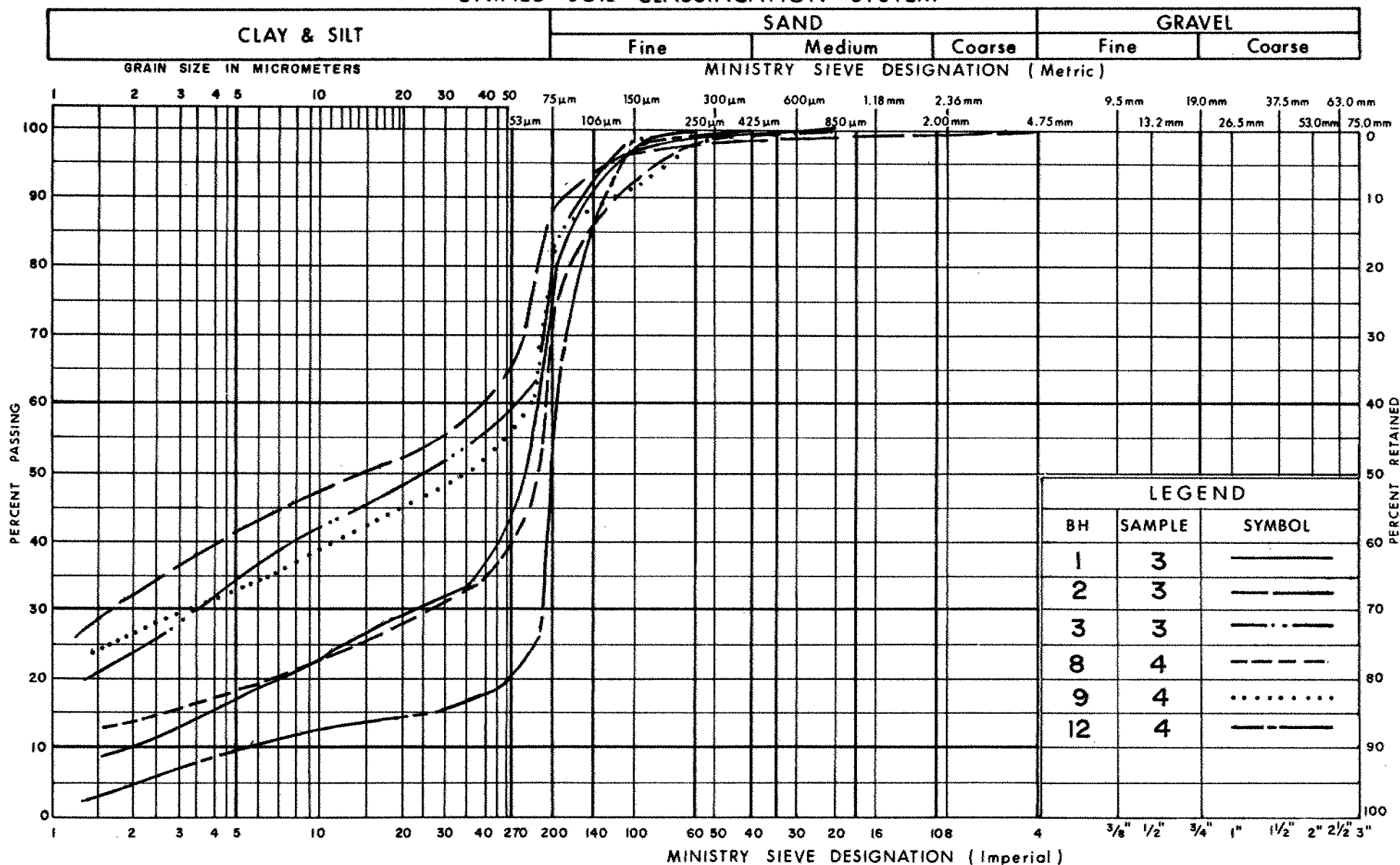
W P 120-87-04 LOCATION Co-ordinates N 5 022 529; E 358 185 ORIGINATED BY RP
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, Cone Test COMPILED BY KJ
 DATUM Geodetic DATE September 15, 1988 CHECKED BY MT

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100				
65.9	Ground Level														
65.7	Topsoil														
0.2	Silty Clay, some sand, occasional sand partings		1	SS	12										
	Firm to Very Stiff		2	TW	PH										
64.1	Brown becoming grey at approximately 64.4m														
1.8	Clayey Silt, with interbedded silty sand and sandy silt layers		3	TW	PH										
	firm to very stiff grey		4	TW	PH										
			5	TW	PH										
60.3															
5.6	Silty Sand, trace gravel, occasional cobble		6	SS	17										
	Compact Grey														
58.9															
7.0	Sandy Silt, trace gravel, trace clay		7	SS	24										
	Compact Grey														
57.7	(Glacial Till)														
8.2	Sand, fine to medium		8	SS	38										
	Dense Grey														
55.8															
10.1	End of Borehole Auger Refusal on probable bedrock														

+3, x5: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM



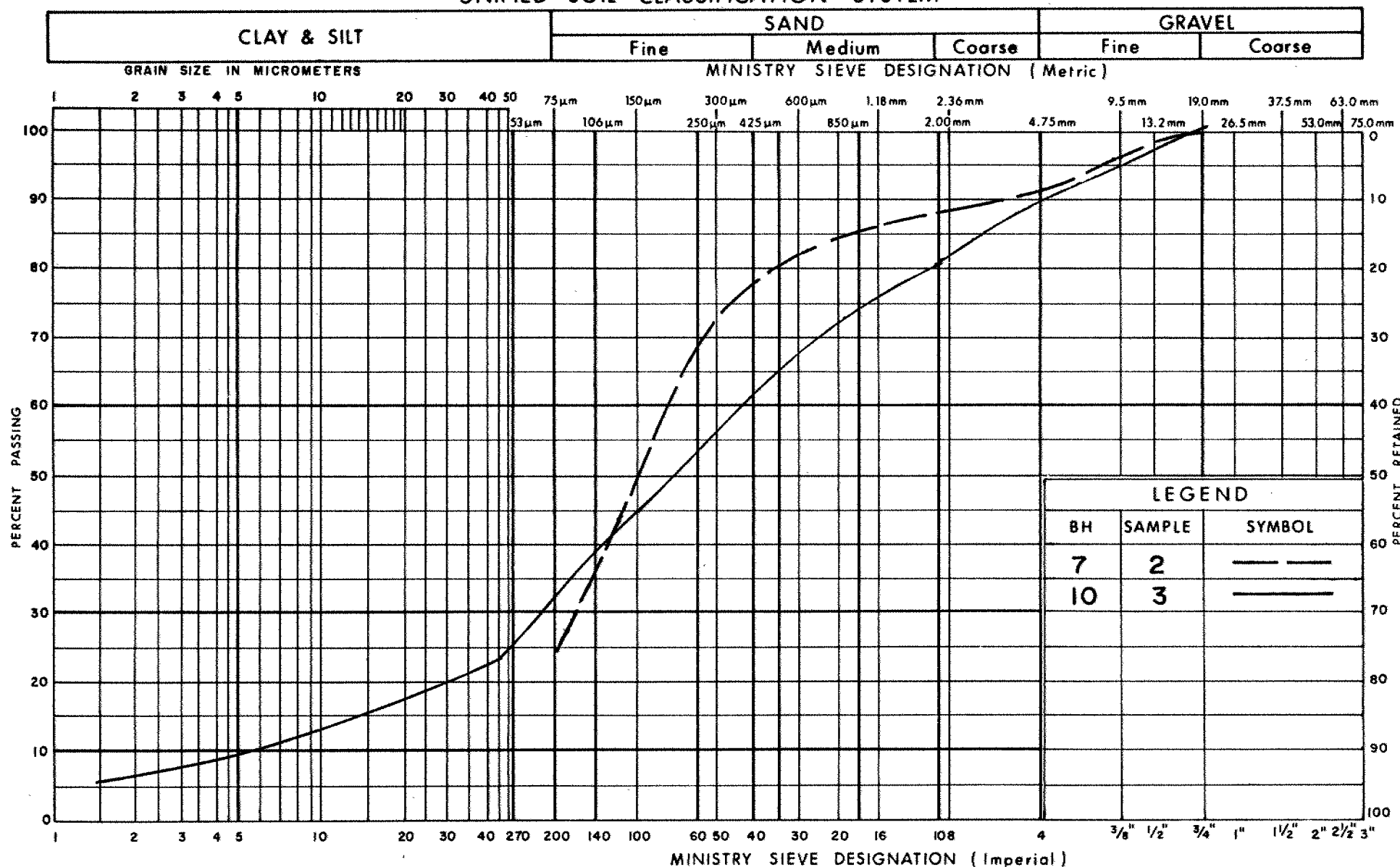
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY CLAY, SOME SAND (CL)

FIG No 1

W P 120-87-04

UNIFIED SOIL CLASSIFICATION SYSTEM

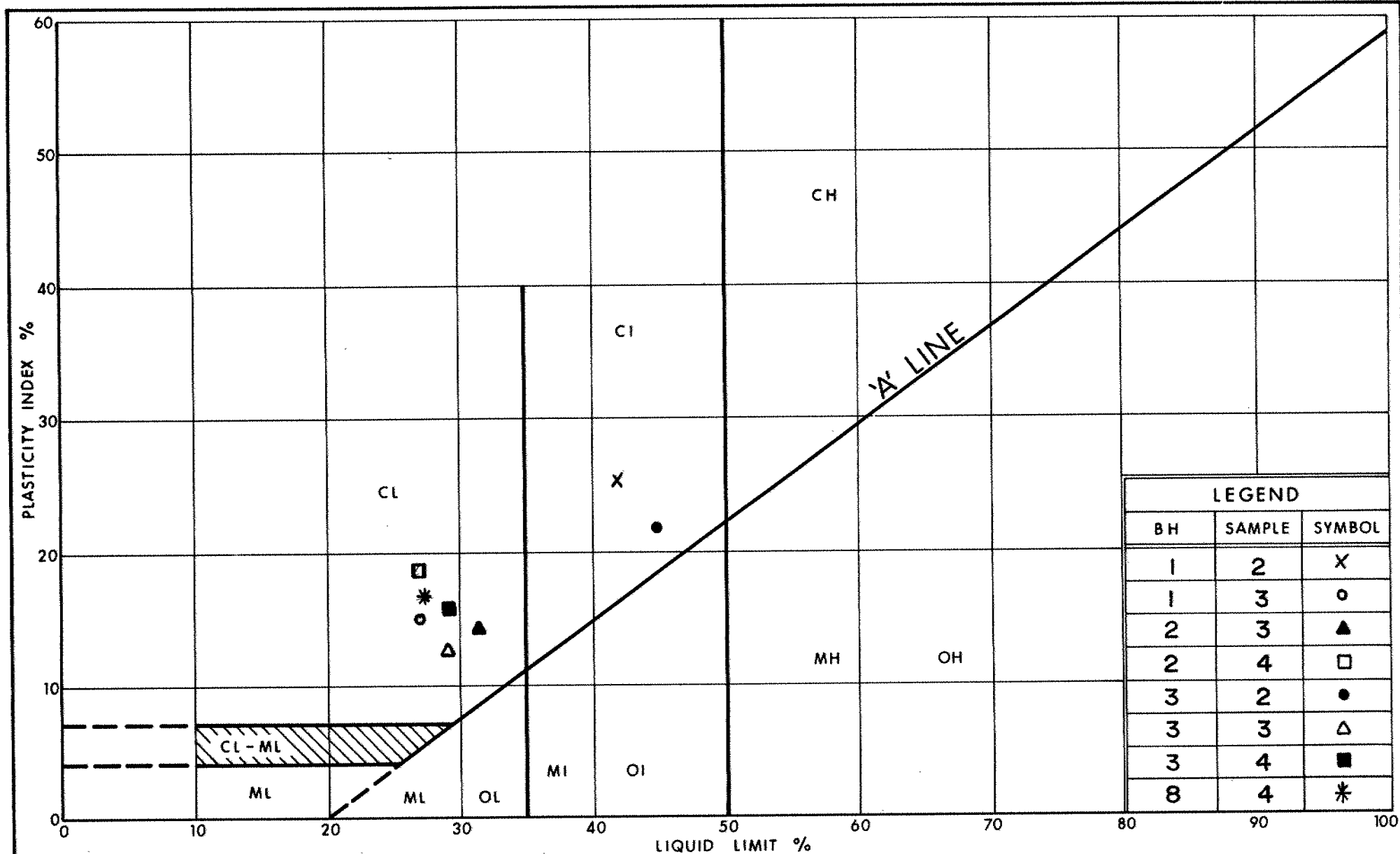


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Transportation

GRAIN SIZE DISTRIBUTION
SILTY SAND TILL (SM)

FIG No 2

W P 120-87-04



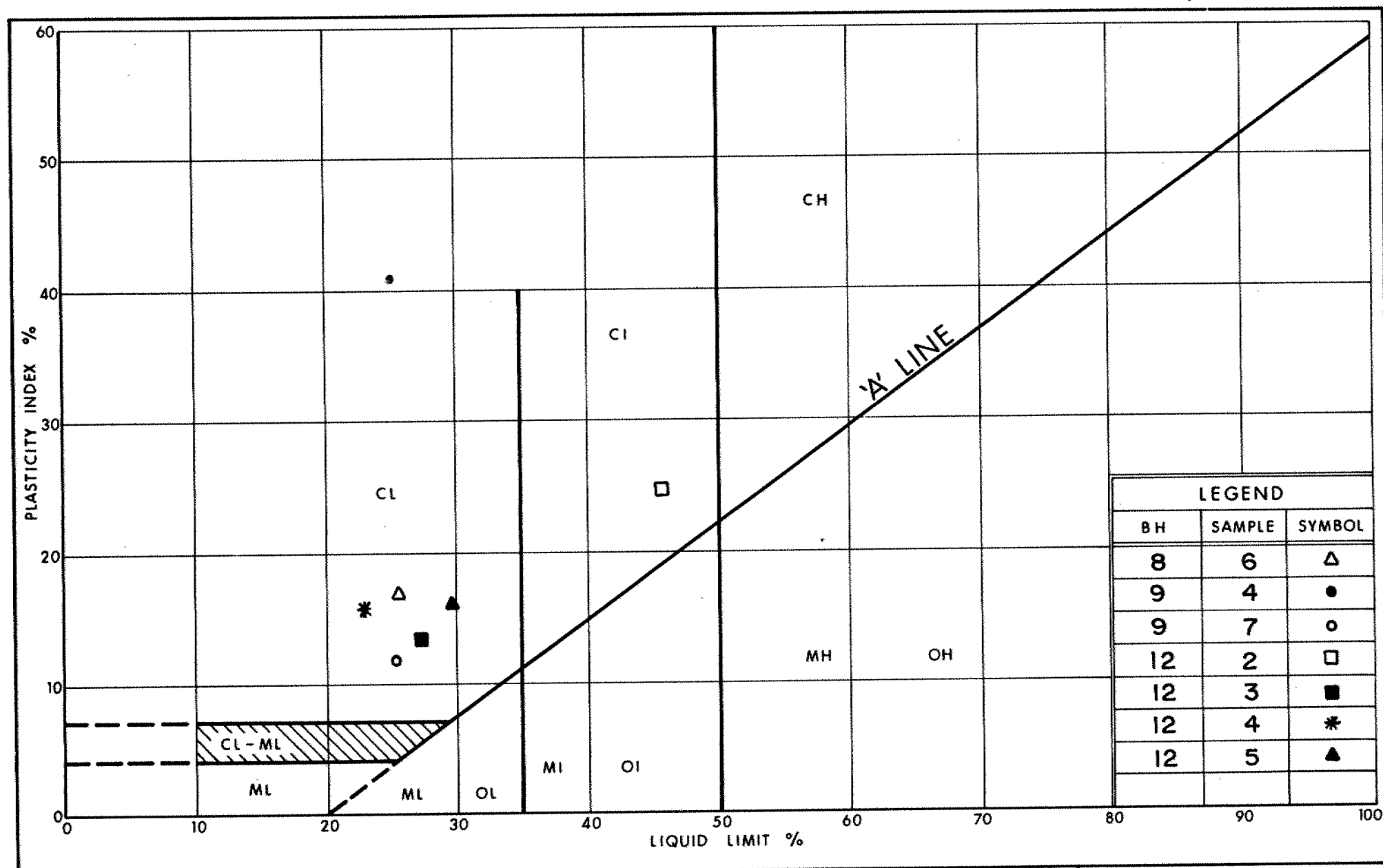
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Ontario

PLASTICITY CHART SILTY CLAY, SOME SAND

FIG No 3

W P 120-87-04



Ministry of
Transportation

Ontario

PLASTICITY CHART SILTY CLAY, SOME SAND

FIG No 4

W P 120-87-04

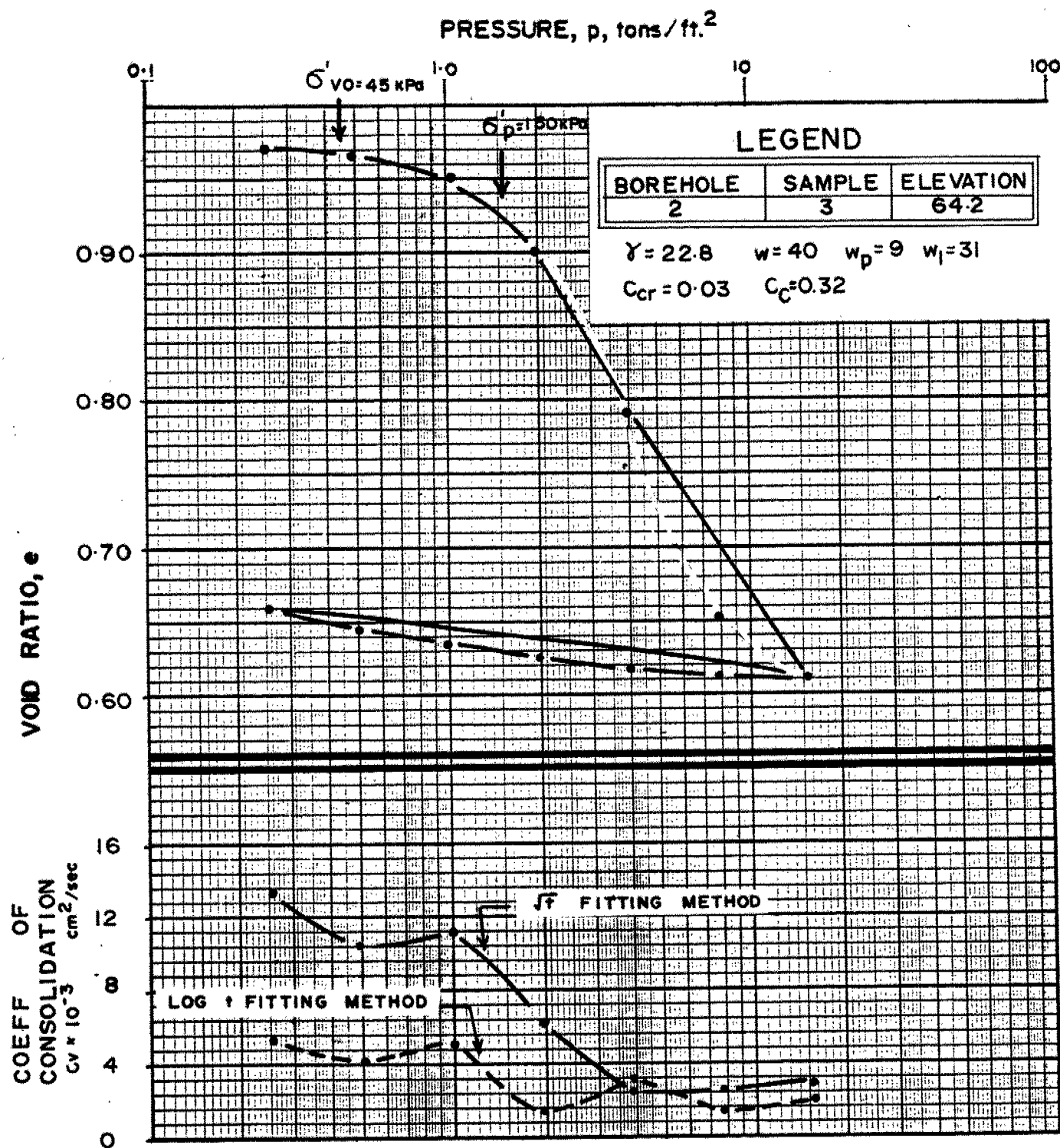
CONSOLIDATION TEST ($e - \log p$)

FIG 5

TESTED BY RC
 CALCULATED BY RC
 CHECKED BY KJ
 DATE OF TESTING SEPT 19 - OCT 3 1988

PROJECT NUMBER 120-87-04
 BOREHOLE NUMBER 2
 SAMPLE NUMBER 3
 DEPTH OF SAMPLE 1.8 m

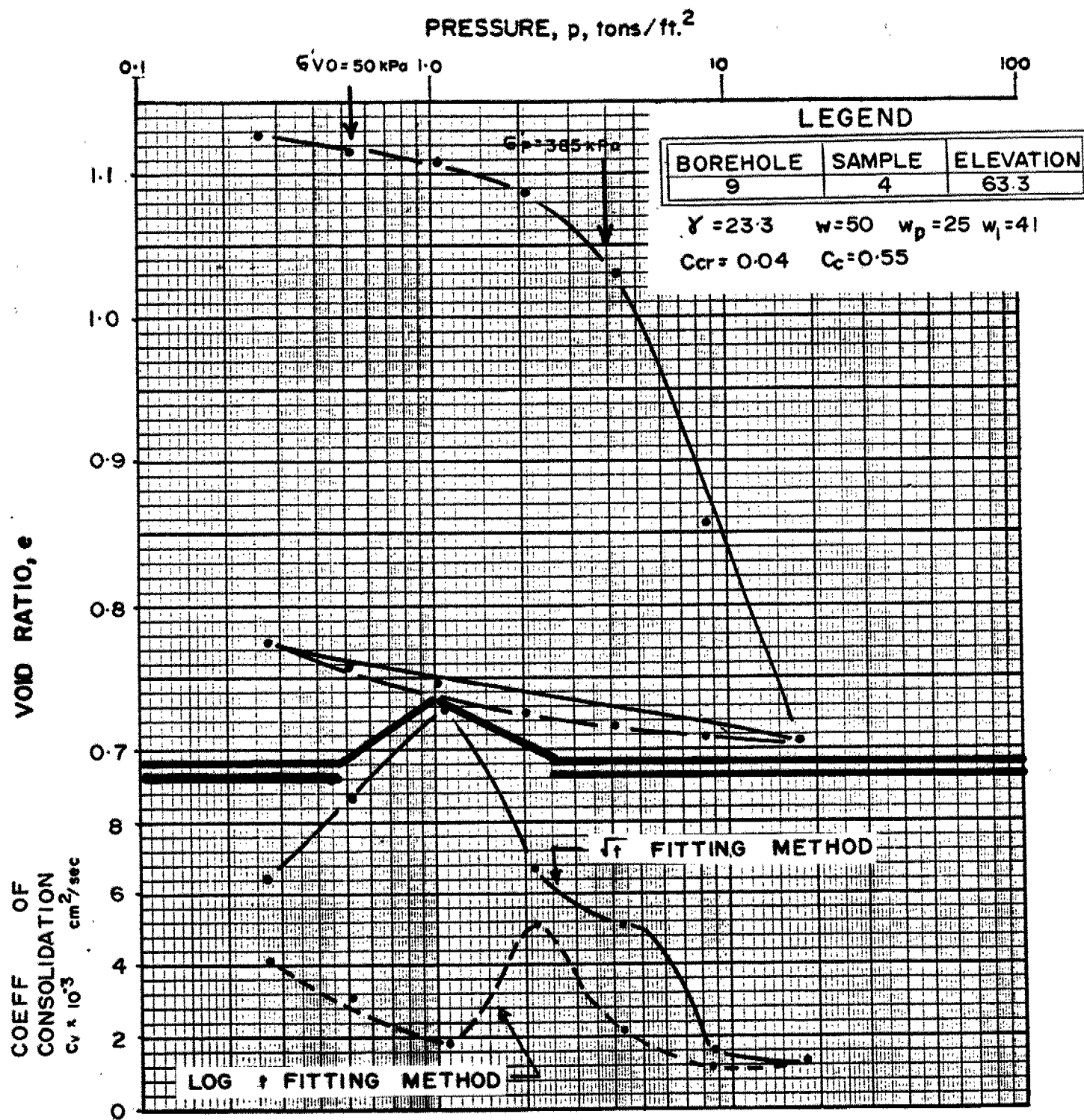
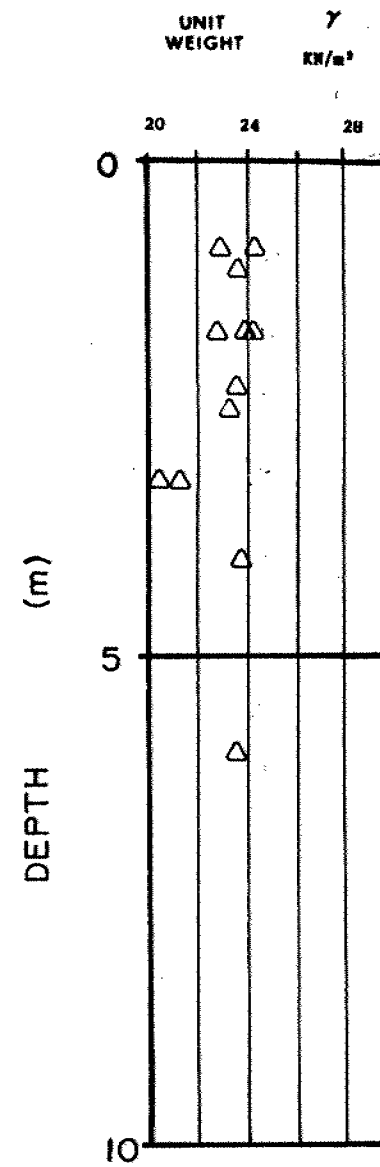
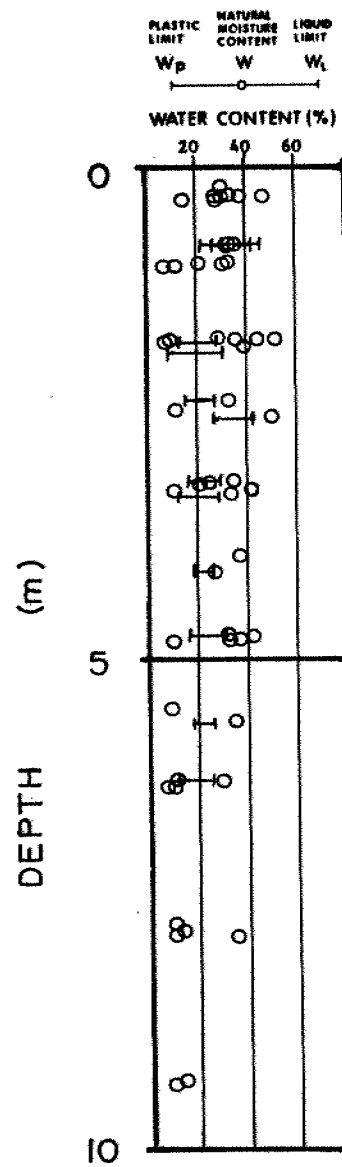
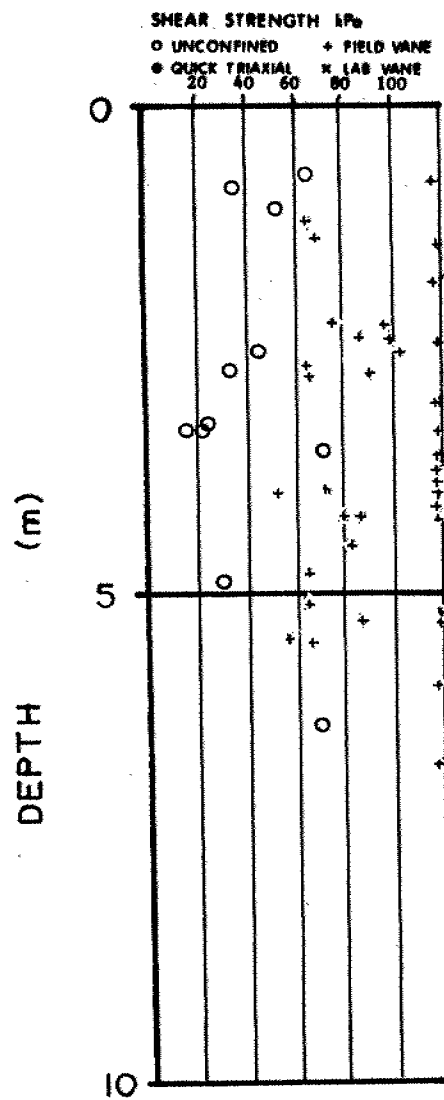
CONSOLIDATION TEST (e -log p)

FIG 6

TESTED BY _____ RC
 CALCULATED BY _____ RC
 CHECKED BY _____ KJ
 DATE OF TESTING OCT 3 - OCT 16 1988

PROJECT NUMBER 120-87-04
 BOREHOLE NUMBER 9
 SAMPLE NUMBER 4
 DEPTH OF SAMPLE 2.7m



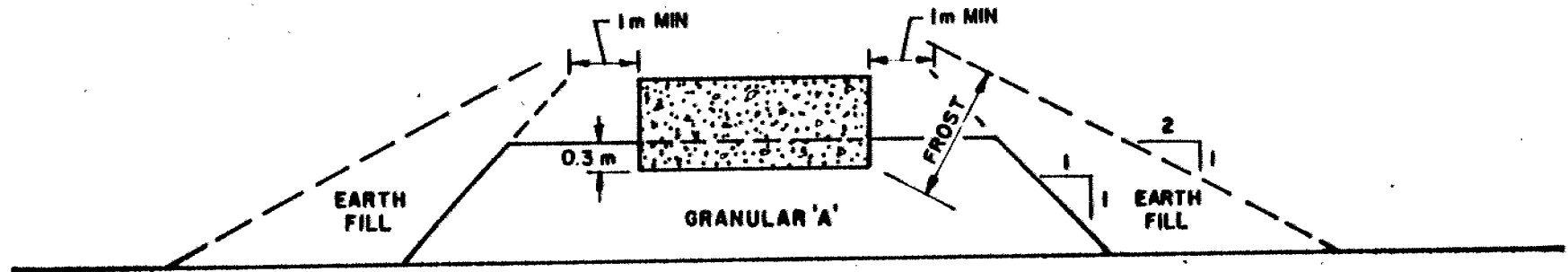
Ministry of
Transportation

Ontario

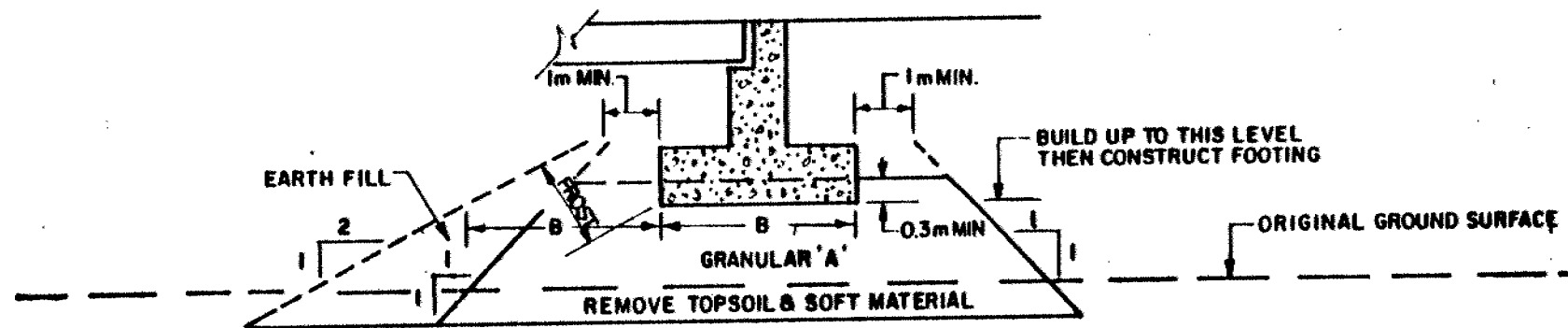
SOIL PROFILE SUMMARIES

FIG No 7

W P 120-87-04



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



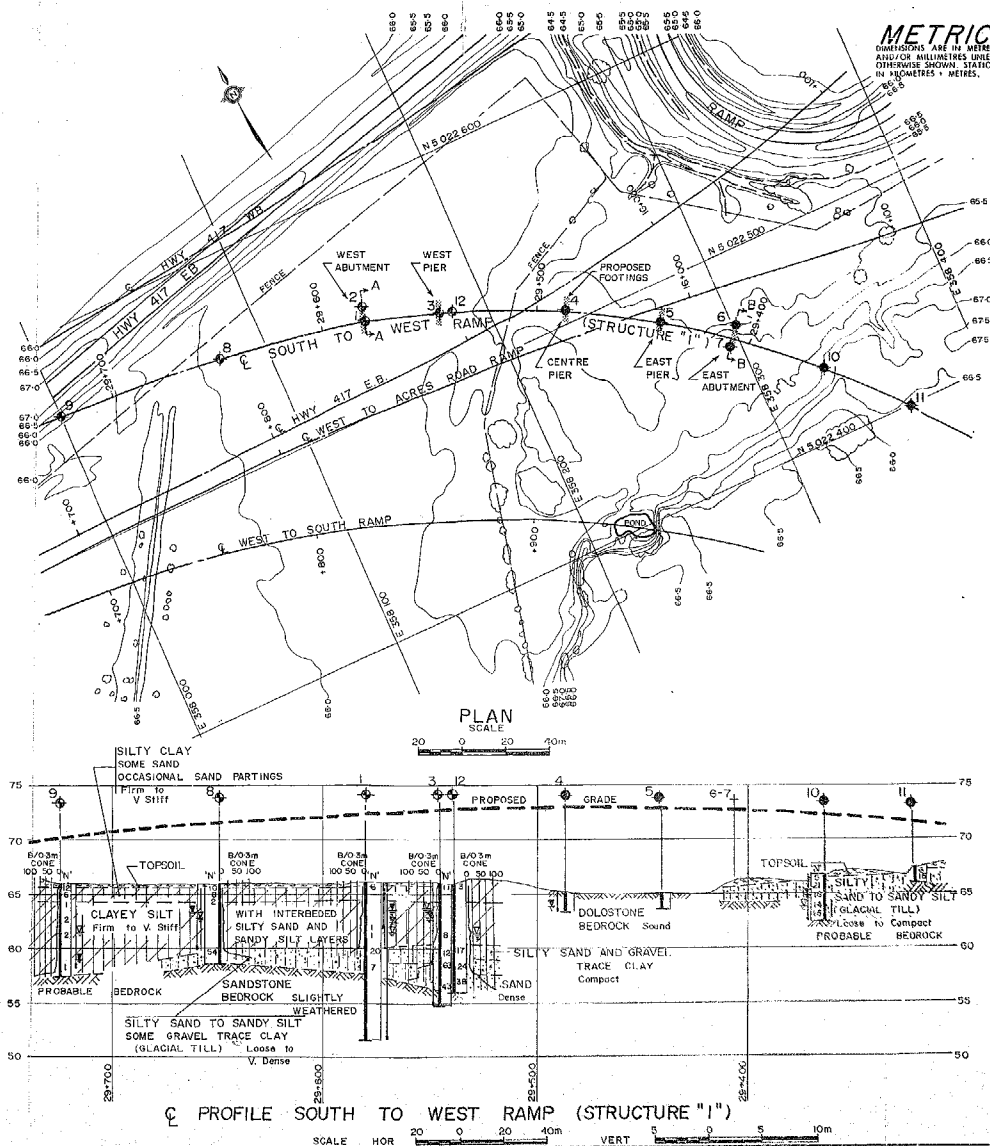
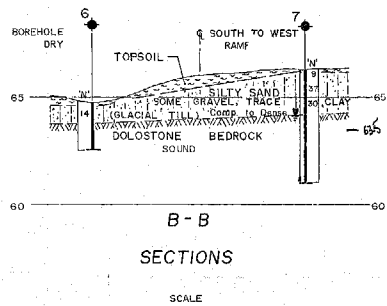
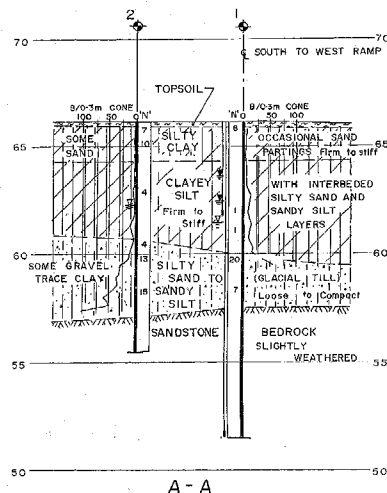
Ministry of
Transportation

ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR 'A' CORE

FIG No 8

W P 120-87-04

OVERSIZE DRAWING

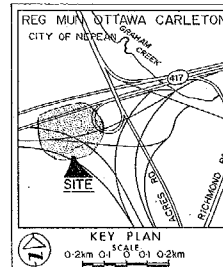


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATISTICS
IN METRES + METRES.

CONT No
WP No 120-87-04
STRUCTURE 'I'
(SOUTH TO WEST RAMP)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

TERRAPROBE LIMITED



- 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 (CT 88)
 - Piezometer
 - WL encountered on day of drilling

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	66.1	5 022 540	358 146
2	66.1	5 022 545	358 147
3	66.0	5 022 530	358 180
4	65.2	5 022 509	358 233
5	65.1	5 022 485	358 270
6	64.7	5 022 470	358 302
7	66.3	5 022 462	358 295
8	66.1	5 022 553	358 080
9	65.0	5 022 160	358 000
10	65.5	5 022 436	358 330
11	67.5	5 022 400	358 360
12	65.9	5 022 529	358 185

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

DATE	BY	DESCRIPTION
1988-09-11	BY	DESCRIPTION
1988-09-11	BY	DESCRIPTION
1988-09-11	BY	DESCRIPTION

Geos No 3165-147

HWY No 417	DIST B
SUBWD KJ CHECKED KJ DATE 1988-09-11	SITE 3-531
DRWNG JB CHECKED JJB	DWG 1208704-A