



## Memorandum

41K-33

GEOCREs No.

to:

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From: Soil Mechanics Section  
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Attention:

Date: July 14, 1975

Our File Ref.

In Reply to

JUL 17 1975

Subject:

### PRELIMINARY FOUNDATION INVESTIGATION REPORT

For

Feasibility Study of T.C. Hwy. No. 17  
(Proposed 4 Lanes)  
From Echo River to Bar River Road  
District No. 18 (Sault Ste. Marie)  
W.P. 903-72-01 00

We have carried out preliminary foundation investigation for the feasibility of new Highway 17 (proposed 4 lanes) from Echo River to Bar River Road. The following sites were investigated:

1. CPR Overhead
2. Hwy. 638 Interchange
3. Embankment Fills at approx. Sta. 150
4. Echo River Structure

The findings of the investigation were reported in the form of short reports (Parts I to IV) issued immediately upon the completion of the field work at each site.

Subsequently, meetings were held in the Regional Office to discuss the data submitted in the reports. It was concluded that the crossing of CPR and Hwy. 17 Lines A, B, C & D were not favourable from the foundations point of view. It was agreed that this Section would further investigate alignment 5. At this stage Mr. W.D. Neilipovitz suggested that Line 4 should also be investigated.

Findings of those additional investigations were reported in the Addendum to Part I.

This report contains all the information covered in the earlier reports, and also the recommendations regarding the most suitable alignment from the foundations points of view. Drawings 9037201-A and B contained in the Appendix show the location of boreholes and the stratigraphy. The previous reports (Part I to IV & Addendum to Part I) should be discarded as this report contains all relevant data.

No laboratory tests have been carried out (except on 2 samples at Hwy. 638) to determine the engineering properties of the subsoil. However, in our opinion, the data contained in this Report will be adequate for the preliminary design of the various structures.

It should be noted that the recommendations given in this Report are of a preliminary nature. A final foundation investigation will be necessary once the preliminary design is completed.

  
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# PRELIMINARY FOUNDATION INVESTIGATION REPORT

For

Feasibility Study of T.C. Hwy. No. 17 (Proposed 4 Lanes)  
From Echo River to Bar River Road  
District No. 18 (Sault Ste. Marie)  
W.P. 903-72-01

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## 1. INTRODUCTION

The Soil Mechanics Section was requested by Mr. W.L. Lees, Manager, Systems Design, Northwestern Region, Thunder Bay to carry out a preliminary foundation investigation for the proposed Hwy. No. 17 (4 lane) between Bar River Road and Echo River.

Subsequently, an investigation was carried out at the location of three proposed structures, and at one location of high fills. These sites are as follows, in a westerly direction:

1. CPR Overhead
2. Hwy. 638 Interchange
3. Embankment Fills at approx. Sta. 150
4. Echo River Structure

The findings of the investigation were reported in the form of short reports (Parts I to IV) issued at the completion of field work at each site. This report summarizes the subsoil conditions and preliminary comments regarding the foundation structures and the stability of embankments or cuts. The recommendations are based on the information obtained in the field and on the in situ tests. As a result, the recommendations contained herein are of a preliminary nature. A complete foundation investigation will be necessary once the alignment and geometrics are finalized.

The locations of boreholes and stratigraphical profile are shown on the attached drawing.

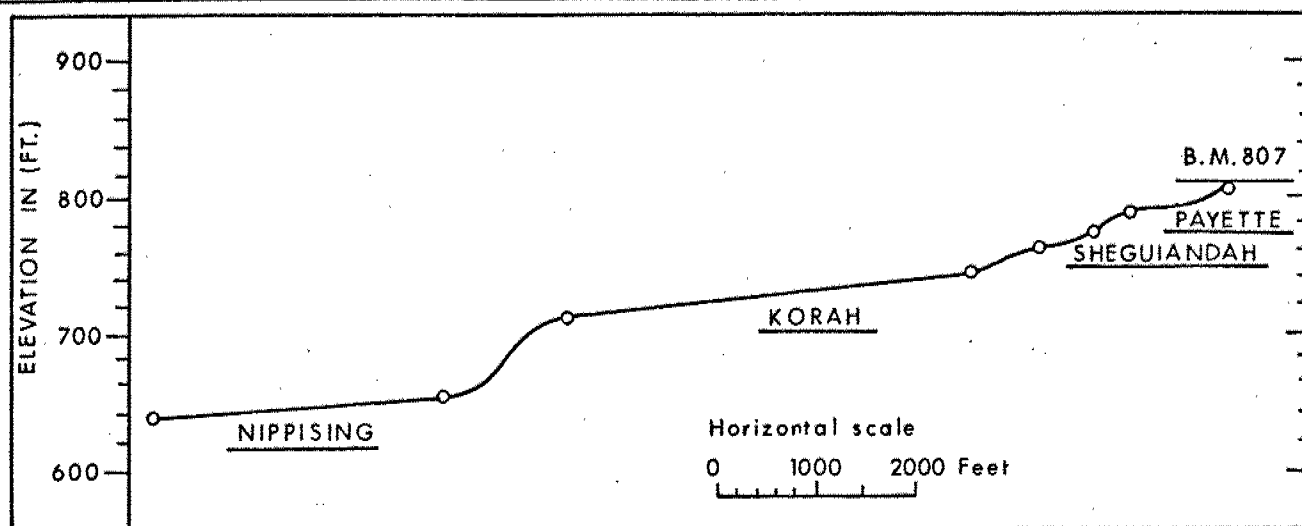
The stratigraphy in each borehole and the results of in situ tests are shown on the Record of Borehole Sheets attached herewith.

## 2. GEOLOGY, SUBSOIL AND ASSOCIATED ENGINEERING PROBLEMS

The geological history of the Great Lakes region is known to be quite complex. The lakes are located almost wholly within the Precambrian Shield. Further, they are nearly surrounded with highlands - i.e., the general pattern is that of a syncline with the Great Lakes occupying the trough. The edge of this shield is located approximately 5 miles north of Sault Ste. Marie.

In the geologic past the area was inundated by glacial Lake Algonquin, the main stage of which existed at about elevation 605. The lakeshore lay along the edge of a crystalline-rock upland, a few miles north of Sault Ste. Marie. In addition, there were a number of post-Algonquin sub-stages. During these sub-stages beaches were formed with the lake water outlets at successively lower levels. At least three such beaches have been identified in the Sault Ste. Marie area, namely, the Payette, Sheguiandah and the Korah systems; all these beaches are parallel to the original Algonquin beach. Following the Algonquin period, the general area was upwarped in a northerly direction; the maximum amount of uplifting occurs along a line located about 6 miles north of Sault Ste. Marie, here the Algonquin beach is known to be approximately 410 feet above the original glacial lake level, which was at elevation 605.

Following the aforementioned events, a low water stage, known as the Nipissing stage, existed. The deposits laid down in this, the most recent of the periods, crosses the parallel Algonquin beaches. This beach like the former beaches, was upwarped 25 to 50 feet above the present surface of Lakes Huron and Michigan. A typical profile in the Sault Ste. Marie area, showing the sequence and location of the various beaches, is given in the figure below.



The deposits, laid down during the various glacial lake stages, were composed of stratified silts and sands completely interbedded with strata of layered clay. These deposits, which are of lacustrine origin, are of variable thickness.

The overburden is underlain by fine-grained ferruginous sandstone bedrock. Geologic references indicate that this bedrock was formed during a transitional period between the Precambrian and Cambrian eras. It is of the Bruce Series of the Upper Keweenawan System.

The predominant soil type in this area is a lacustrine deposit of soft to stiff silty clay to clay. This stratum, occurs below a thin veneer of top soil or locally deposited silts and sands. The thickness of this stratum is variable. In general, the thickness of the clay deposit increases as the distance from the nearby hills increases and the distance from Lake George decreases. A number of boreholes were terminated in this stratum at depths in excess of 130 ft.

In some boreholes put down near the hills, at the location of the proposed CPR overhead, the clay deposit was fully penetrated. In these boreholes the clay stratum is underlain by a granular deposit which becomes coarser with depth.

Bedrock outcrops can be seen in the sides of the hills to the east.

As stated earlier the predominant subsoil in this area is the clay deposit found at or close to the ground surface. The design and performance

of engineering structures, fills and cuts is governed by this stratum.

The shear strength of this deposit, in general, increases with depth. In the upper portion, where it is more critical, the shear strength varies from 250 p.s.f. to 400 p.s.f. At depths greater than 50-60 ft., the shear strength is in the order of 1000 p.s.f. The material is cohesive and compressible. These unfavourable strength and compresssibility characteristics present the following engineering problems related to foundations of structures and stability and settlements of embankments:

### 1. Foundations

Spread footing type foundations in the original ground cannot be provided, because

- (a) The shear strength of the clay to silty clay deposit in the upper portion is only 250 to 400 p.s.f. This would allow a maximum bearing capacity of 500 to 800 p.s.f., which is impractical.
- (b) The clay to silty clay deposit is very compressible and deep. The footings will settle excessively in this material when subjected to loading.

### 2. Stability of Embankments

The presence of the compressible and cohesive clay to silty clay stratum requires that steps must be taken to ensure the overall stability of the fill sections as well as limit the settlements to a tolerable magnitude.

The critical condition for stability of an embankment on normally or slightly overconsolidated clays, as is the case with this clay stratum, generally occurs during or immediately after construction. This being the case, a total stress analysis ( $\phi = 0$ ) provides a suitable means of assessing the stability of the fill sections. In this method of analysis, stability is governed by the applied loads and the stress-strain and undrained shear strength properties of the foundation and fill material.

For preliminary purposes it is assumed in this report that a granular type fill material will be used to construct the embankments which will



have 2 horizontal to 1 vertical slopes. The soil properties for the fill are:

Bulk density	$\gamma = 125$ p.c.f.
Angle of shearing resistance	$\phi = 30^\circ$

The subsoil will be able to safely support a particular height of embankment which depends upon the undrained shear strength of the underlying soil. If the height of the embankment is higher than the safe height, then measures have to be taken against a rotational failure through the soft subsoil. These measures consist of:

- (a) providing one or more counterbalancing berms. The number and length of each berm depends upon the strength characteristics of the subsoil.
- (b) using lightweight fill material, such as slag from the local steel mill, for embankment construction. For computation purposes, it is assumed in this report that the bulk weight of lightweight fill is 90 p.c.f. and the angle of shearing resistance is  $30^\circ$ .
- (c) a combination of using lightweight fill and providing counterbalancing berms. The safe height of embankment (without berms) is considerably, increased, and the berm requirements are considerably reduced because of the lower unit weight of fill and consequently lower stresses induced in the underlying soil,
- (d) constructing the embankment slowly in stages. The procedure consists of building the embankment up to its safe height and leaving it to consolidate. During consolidation the subsoil gains some amount of strength. Additional height is added dependent upon the gain in strength, which is again left in place. This results in further consolidation of subsoil. This procedure is repeated many times. However, for this project this method of construction is not considered practical because of the time constraint. Besides the longer time of construction, this method requires elaborate instrumentation and close monitoring of behaviour and construction.

### 3. Settlement of Embankments

The underlying highly compressible cohesive stratum will undergo settlements due to consolidation, over a long term period, under the weight of the embankment. The amount of settlement depends upon the stresses induced in the subsoil, and the thickness of the compressible layer. The superimposed stresses, in turn depend upon the unit weight of the fill material, height of fill and the base width of the embankment.

Settlement calculations have been done using the results of tests conducted, in the past, on similar soil in this general area.

The settlement of the underlying compressible subsoil exerts negative frictional forces on end-bearing piles driven through it. Therefore, such piles have to be designed with a correspondingly lower load carrying capacity.

Furthermore, the abutments founded on long end-bearing piles driven through a deep, soft and compressible deposit have a tendency to undergo rotational movements.

In the following sections, each site is dealt with individually.

### 3. CPR OVERHEAD

#### 3.1 Introduction

The site is located about 2 miles south of Echo Bay. At this site the CPR tracks, are on a curve. The land adjacent to the tracks is flat, agricultural land. However, the land on the northeast gradually rises in the form of a hill to an ultimate height of 120 ft.

At this location, lines A and C merge together and lines B & D merge as another line and cross the tracks about 1300 ft. north of Maple Leaf Road and 1600 ft. east of the existing Hwy. At a later date two more alignments, one about 2500 ft. north (Line 4) and the other about 900 ft. southeast (Line 5) of the original proposal were investigated.

The field work consisted of three boreholes (#1, 2, and 9) along Lines A, B, C & D; two boreholes (#10 & 11) along Line 4; and nine boreholes (#12 to 19) along Line 5.

### 3.2 Subsoil Conditions

The subsoil conditions encountered along various Lines are as follows:

#### 3.2.1 Lines A, B, C, & D (Boreholes 1, 2 & 9)

<u>Borehole No. 1</u> <u>(G.L. 617 ft.)</u>	<u>Borehole No. 2</u> <u>(G.L. 619 ft.)</u>	<u>Borehole No. 9</u> <u>(G.L. 620 ft.)</u>	
0 - 3 ft.	0 - 3 ft.		Topsoil and sandy silt
		0 - 5 ft.	Clay, some sand
3 - 6 ft.	3 - 6 ft.	5 - 10 ft.	Clay to silty clay - desiccated zone
6 - 61 ft.	6 - 59 ft.	10 - 39 ft.	Clay to silty clay - soft to firm
		39 - 52 ft.	Sand and gravel
61 - 70 ft.	59 - 68 ft.		Silty fine sand to sandy silt
70 ft.	68 ft.	52 ft.	Refusal

The in situ vane tests indicate that the undrained shear strength of the clay to silty clay deposit gradually increases with depth and varies from 400 p.s.f. in the upper portion to about 1000 p.s.f. at the bottom of the deposit. The boreholes were terminated when refusal was met. No attempt was made to determine the type of deposit below this level.

It is estimated that groundwater level is about 7 ft. below ground surface.

#### 3.2.2 Line 4 (Boreholes 10 & 11)

The subsoil conditions encountered in both boreholes are briefly as follows:

<u>Borehole No. 10</u> <u>(Ground Elev. 614 ft.)</u>	<u>Borehole No. 11</u> <u>(Ground Elev. 615 ft.)</u>	
0 - 4.5 ft.	0 - 11 ft.	Silty sand to sand
4.5 - 11		Organic clay, very soft
	11 - 15.5	Clayey silt, soft to firm
11 - 140	15.5 - 138	Silty clay to clay, soft to stiff
140 -	138 -	Sand and gravel
143	140	End of Borehole

The in situ vane tests indicate that the undrained shear strength of the silty clay to clay stratum gradually increases with depth and varies from 400 p.s.f. in the upper position to about 1000 p.s.f. at a depth of about 100 ft.

### 3.2.3 Line 5 (Boreholes 12 to 19)

Boreholes 12 to 17 were carried out in the vicinity of the proposed crossing. Boreholes 18 and 19 were put down for investigating the stability of the embankment (400 to 1000 ft.) south of Maple Leaf Road.

Subsoil in Boreholes 12 to 17 consists of a 16.5 to 40 ft. thick deposit of silty sand to sand overlying a sand and gravel with cobbles stratum. The Boreholes were terminated at depths varying from 26.5 ft. to 47.0 ft., when refusal to augering was met.

In Borehole 14, which was put down in a ploughed field, the uppermost 9 ft. consisted of topsoil and firm clay overlying silty sand to sand. The relative density of the silty sand to sand deposit ranges from very loose to dense, but is in general very loose to loose. The relative density of the sand and gravel deposit is generally compact to very dense.

In Boreholes 18 and 19, the upper 19 to 37 ft. consists of soft to firm silty clay to clay, followed by loose to very loose silty sand to sand. The thickness of the silty clay to clay stratum increases in a southerly direction, i.e. away from the hill. The in situ vane tests indicate that the undrained shear strength of the silty clay to clay ranges from 400 to 600 p.s.f., generally in the order of 400 p.s.f.

### 3.3 Discussion and Recommendations

#### 3.3.1 Lines A, B, C & D

##### 3.3.1.1 Structure Foundations

It is recommended that the entire structure be supported on end-bearing steel piles driven to refusal. It is estimated that refusal will be achieved at approximate elevation 545, i.e. about 70 ft. below the ground level.

Perched abutments founded on spread footings placed in compacted granular fills are not recommended, because of excessive settlements of the fills. The footings placed in the fills will settle by the same amount as the fills themselves. The settlement aspect of the approach embankments is dealt with in subsection 3.3.1.2.

##### 3.3.1.2 Approach Embankments

###### (i) General

The proposed grade of Highway No. 17 is such that it will require embankments up to 30 ft. in height. These fills will be underlain by 60 ft. of soft to firm compressible clay to silty clay (layered) stratum.

###### (ii) Stability Considerations

Analyses have been carried out, in terms of total stresses, to determine the stability of the fill sections. The soil properties for the natural subsoil, assumed for computation purposes, and the results of analyses are as follows:

###### (A) Fill Material - Granular Type ( $\gamma = 125$ p.c.f.)

###### Subsoil

0 - 5 ft.	Cu = 1000 p.s.f.
5 - 18 ft.	Cu = 400 p.s.f.
18 - 48 ft.	Cu = 500 p.s.f.
48 - 58 ft.	Cu = 1000 p.s.f.

- fills less than 14 ft. in height may be constructed with 2:1 slopes
- for a fill height of 20 ft., a single mid-height berm of 50 ft. will be required
- for a fill height of 30 ft., double berms, each approximately 60 ft. in width, should be incorporated into the design. These berms should be located at equi-distances along the slope (i.e. at the third points).

A typical cross-section for single and double berms is shown in Fig. 1.

(B) Fill Material - Lightweight Aggregate ( $\gamma = 90$  p.c.f.)

- fills less than 20 ft. in height, may be constructed with 2:1 slopes
- fills 30 ft. in height would require a single mid-height berm 55 ft. in width

In order to minimize the embankment heights and consequently the berm requirements and associated settlements, consideration should be given to a longer, multi-span structure over this area.

(iii) Settlement Considerations

The consolidation settlement, estimated for various heights of fill, are summarized in the following table:

<u>Height of Fill</u>	<u>Fill Material</u>	<u>Berm Requirements</u>	<u>Estimated Settlement Consolidation</u>
			<u>Ft.</u>
30 ft.	Locally Available	Double - Ea. 60' in Width	3      1 - 2 yrs. 6 (Max.) - 25 yrs.
30 ft.	Lightweight	Single 50' in Width	2.5      1 - 2 yrs. 5 (Max.) - 25 yrs.
20 ft.	Locally Available	Single 50' in Width	2.5      1 - 2 yrs. 5 (Max.) - 25 yrs.
20 ft.	Lightweight	Nil	1.5      1 - 2 yrs. 3 (Max.) - 25 yrs.
14 ft.	Locally Available	Nil	1.5      1 - 2 yrs. 3      - 25 yrs.

It would, be advantageous to construct the embankments first and leave them in place for as long a period as possible, prior to constructing the structure.

### 3.3.2 Line 4

#### 3.3.2.1 Structure Foundations

Because of the soft and compressible subsoil at this location, spread footing type foundations in the original ground are not recommended. The entire structure may be supported on end-bearing steel piles driven to refusal. It is estimated that refusal will be achieved at about 150 ft. (vs. 70 ft. for Lines A, B, C & D) below the ground level.

The negative frictional forces exerted on the piles will be greater at Line 4 because of the greater depth of compressible stratum at this site. Therefore, the piles will have to be designed with a correspondingly lower load carrying capacity.

### 3.3.2.2 Approach Embankments

#### (i) Stability Considerations

The original investigation at the crossing of Hwy. 17 (Lines A, B, C & D) and the CPR tracks revealed that the soft to firm silty clay to clay deposit is the predominant soil type in this area. The thickness of this deposit at the crossing of Lines A, B, C & D is in the order of 60 ft., while at the crossing of Line 4, it is in the order of 140 ft. The shear strength pattern at both crossings are essentially similar. Therefore, the recommendations contained in Section 3.3.1.2 regarding the stability and settlement considerations of approach embankments and the structure foundations, are applicable to Line 4 crossing with the following modifications:

The recommendations with respect to the stability of embankments remain unaltered. In other words, the safe embankment height with 2:1 side slopes is about 14 ft., if locally available material ( $\gamma = 125$  p.c.f.) is used, and 20 ft. if lightweight material ( $\gamma = 90$  p.c.f.) is used for the fills. Embankments higher than the above heights will require counterbalancing berms as explained in the original Report. The length of berms will depend upon the height of embankment.

#### (ii) Settlement Considerations

The thickness of the compressible silty clay to clay deposit at the crossing of Hwy. 17, Line 4 and the CPR tracks, is about 140 ft., while at the crossing of Lines A, B, C & D, and the CPR tracks it is only 60 ft. As a result, under the same height of fill, the long term consolidation settlements along Line 4 will be even greater than the settlements along Lines A, B, C & D. It was estimated that along Lines A, B, C & D, a 30 ft. high



embankment constructed with locally available material, which will require double counterbalancing berms, each 60 ft. long, will settle in the order of 6 ft. It is estimated that a similar embankment (30 ft. high) along Line 4 will settle in the order of 7 ft.

It will be necessary to excavate the very soft organic clay, which was found on the west side of the CPR tracks (Borehole 10), to its full depth (11 ft.) within the plan limits of the approach embankment, and replace it with suitable material.

### 3.3.3 Line 5

#### 3.3.3.1 Structure Foundations

The silty sand to sand deposit is very loose to loose. Therefore, spread footing type foundations in the original ground are not recommended. It is recommended that the entire structure be supported on end-bearing steel piles driven in the sand and gravel stratum. It is estimated that the maximum allowable loads for the particular pile section chosen will be achieved at approximate elevation 570 ft., i.e. about 50 ft. below the ground level.

#### 3.3.3.2 Approach Embankment

At the proposed crossing of the CPR tracks and Hwy. 17, Line 5, the subsoil consists of a very loose to loose silty sand to sand stratum underlain by a compact to very dense sand and gravel with cobbles deposit. A cohesive stratum does not exist at this site. Therefore, no stability problems are anticipated for embankments constructed with 2:1 slopes. Because the underlying subsoil is of a non-cohesive nature, the settlements under the embankments will be elastic and will occur instantaneously as the construction proceeds.

A soft to firm silty clay to clay deposit was found south of the Maple Leaf Road. This stratum is cohesive and compressible in nature. An embankment constructed in this area

will undergo long term consolidation settlements. The thickness of this stratum increases in a southerly direction. However, it is anticipated that the height of embankment decreases in a southerly direction. Because of the above reasons, the settlements along Line 5 will be much smaller than along Lines A, B, C & D.

### Conclusions

A comparison of subsoil conditions along Lines A, B, C & D, 4 and 5, indicates that the cohesive stratum does not exist in the vicinity of the proposed crossing of the CPR tracks and Hwy. 17, Line 5. The problems associated with the approach embankments, namely, long counterbalancing berms and several feet of long term consolidation settlements, which are present along Lines A, B, C, D and 4, are not present at the proposed crossing of Line 5. Therefore, Line 5 is preferable to all other Lines in this area, from a soil mechanics point of view.

## 4. HWY. 638 INTERCHANGE (Lines A, B, C & D)

### 4.1 Introduction

The site is located just east of the village of Echo Bay and about 1 mile east of the present intersection of Hwys. 17 and 638. The land in this area is flat, low lying and on the south side of Hwy. 638 it is covered with bush.

The field work at this site consisted of two boreholes (B.H. 5 & 6) and cone tests. Borehole No. 5 was put down near the present alignment of Hwy. 638. Borehole No. 6 was put down about 700 ft. south of Hwy. 638, in order to investigate a possible realignment of Hwy. 638, if the subsoil conditions are favourable, (Borehole Location Plan).

### 4.2 Subsoil Conditions

A deep deposit of silty clay to clay is the predominant soil type at this site. Both boreholes were terminated in this layer at depths of 85 and 90 ft. Dynamic Cone Penetration tests were carried

out commencing from the bottom of the boreholes, down to a depth of 130 ft. below ground surface. These tests did not indicate any change in soil type to this depth.

The subsoil conditions encountered in both boreholes are, briefly, as follows:

<u>Borehole No. 5</u> <u>(Ground Elev. 584 ft.)</u>	<u>Borehole No. 6</u> <u>(Ground Elev. 589 ft.)</u>	
0 - 1 ft.	0 - 1 ft.	Topsoil
1 - 4 ft.	1 - 3 ft.	Sandy silt with organics
4 - 20 ft.	3 - 32 ft.	Clay to silty clay, soft
20 - 27 ft.		Clayey silt to silt, firm
27 - 38 ft.	32 - 48 ft.	Silt to sandy silt, loose to compact
38 - 130 ft.	48 - 130 ft.	Clay to silty clay (stratified) with silt layers, firm to stiff

The in situ vane tests indicate that the upper soft silty clay to clay stratum has an undrained shear strength of 250 to 300 p.s.f. and, that the lower firm to stiff stratum has an undrained shear strength of 900 to 1200 p.s.f. About 500-1000 ft. west of the proposed crossing, the embankment for Hwy. 638 failed when it was constructed to a height of 4 ft. above the prevailing ground level. This indicates that at some places the subsoil is very soft within 10 ft. of the ground surface. The presence of these very soft layers should be investigated during the final investigation. It will be necessary to remove this very soft material, as this is incapable of supporting even 5 ft. high fills.

The Atterberg Limit test carried out on this deposit gave the following results:

	<u>Liquid Limit</u>	<u>Plastic Limit</u>	<u>Natural Moisture Content</u>
Upper Stratum	37 - 83	18 - 25	40 - 97
Lower Stratum	69	25	63

Grain size analyses carried out on two samples of silt to sandy silt stratum gave the following distribution:

Sand	5 - 18 %
Silt	78 - 91 %
Clay	4 %

The groundwater level is estimated to be about 0 to 6 ft. below the ground surface.

#### 4.3 Discussion and Recommendations

##### 4.3.1 General

It is proposed to construct an interchange at either of the crossings of New Highway 17 (Lines A, B, C or D) and Highway 638.

At this location Lines B, C and D come very close together, and Line A is about 200 ft. east of them. Therefore, the comments contained herein are equally applicable to all four lines.

Two possible locations for the proposed grade separation were investigated.

1. at the present alignment of Hwy. 638 (Borehole 5);
2. south (500 - 1000 ft.) of Hwy. 638 (Borehole No. 6).

It was believed that the subsoil conditions may be more favourable at this place.

A comparison of information from Boreholes 5 & 6 indicates that the subsoil conditions do not improve south of Hwy. 638. The differences between the two boreholes are not very significant. However, the thickness of the soft clay to silty clay layer is only 20 ft. in Borehole 5, while it is 32 ft. in Borehole 6. Therefore, Borehole 5 location is more preferable than Borehole 6 location for the proposed crossing. However, it will be necessary to carry out a more detailed investigation to confirm the advantage of one site over the other. The comments contained in the following text are equally applicable to both alternate crossing sites.

The recommendations for the structure foundations and the approach embankments are given below:

#### 4.3.2 Structure Foundations

Spread footing type foundations in the original ground are not recommended because of the low shear strength (250 p.s.f.) and very compressible nature of the underlying silty clay to clay stratum.

Therefore, the entire structure should be supported on friction timber piles. For No. 14, treated timber piles driven at least 40 ft. in the original ground, an allowable load of 12 tons per pile may be assumed for estimate purposes.

No end-bearing stratum was encountered to a depth of 130 ft. However, if further and deeper boring reveals an end-bearing stratum at a reasonable depth, then the entire structure may be supported on steel piles driven to refusal.

#### 4.3.3 Approach Embankments

##### 4.3.3.1 General

It is proposed to construct an interchange to carry Hwy. 638 over new Hwy. 17. In the initial stages the crossing will be a grade crossing. Because the interchange is a long-term proposal, the ultimate grade of Hwy. 638 is not known at this time. However, it will require embankments about 20 - 25 ft. in height. In the initial stage of a grade crossing, the maximum height of embankment for Hwy. 17 is 5 ft. (Line C).

##### 4.3.3.2 Stability Considerations

A very preliminary analysis in terms of total stress, to determine the stability of the fill sections, indicates that:

###### A. For Granular Fill ( $\gamma = 125$ p.c.f.)

1. Fills up to 8 ft. with 2:1 slopes will be stable.
2. Fills in excess of 8 ft., but less than 16 ft. in height will require counterbalancing mid-height berms.

3. The mid-height berm will be 8 ft. long for every 1 ft. height in excess of the safe height of 8 ft.
4. Fills in excess of 16 ft. in height will require multiple berms.

B. For Lightweight Fill ( $\gamma = 90$  p.c.f.)

1. Fills up to 10 ft. with 2:1 slopes will be stable.
2. Fills in excess of 10 ft., but less than 20 ft. in height will require counterbalancing mid-height berms.
3. The mid-height berm will be 6 ft. long for every 1 ft. height in excess of the safe height of 10 ft.
4. Fills in excess of 20 ft. in height will require multiple berms.

#### 4.3.3.3 Settlement Considerations

The underlying cohesive subsoil is compressible and will undergo settlements due to consolidation over a long-term period; under the weight of the approach embankments. Settlements have been estimated, using the test results on soil in this general area, and are as follows:

<u>Height of Fill</u>	<u>Estimated Settlements</u>
8 ft. (earth fill)	8 - 10 ins.    1 - 2 yrs.
10 ft. (lightweight fill)	18 - 24 ins.    25 yrs.
16 ft. (earth fill)	18 - 24 ins.    1 - 2 yrs.
20 ft. (lightweight fill)	36 - 48 ins.    25 yrs.

The settlements under earth fills greater than 16 ft. in height (or 20 ft. in height for lightweight fills) will be excessive, therefore, a multi-span structure will be more desirable than embankments, in excess of 16 ft. in height, if constructed with earth fill, and more than 20 ft. in height if constructed with lightweight fill.

It would be advantageous to construct the embankments first and leave them in place for as long a period as possible prior to constructing the structure.

5. EMBANKMENT AT APPROX. STA. 150 (Lines A, B, C, D & D')

5.1 Introduction

A field investigation was carried out to evaluate the stability of the proposed embankments at this location. The embankments are required to cross a hill between this site and the Echo River about 1 mile to the north.

The site is located about 1.5 miles northeast of the village of Echo Bay, and west of Echo Lake Road. At the west limits in the vicinity of Line D, the land is flat and low lying, and gradually rises towards the Echo Lake Road. The area between Line D and Echo Lake Road is heavily treed. To the north of this location, the ground rises in the form of a hill to an ultimate height of 200 ft. above surrounding ground.

The field work at this site consisted of two boreholes (Boreholes No. 7 and 8). Borehole No. 7 was put down on Line D, while Borehole No. 8 was put down about 150 ft. east of the Echo Lake Road; i.e. about 500 ft. west of Line C (See B.H. Location Plan). It was not possible to put down a borehole between Lines A and C because the area is heavily treed.

5.2 Subsoil Conditions

The stratigraphy in the two boreholes and the results of in situ tests are shown on the Record of Borehole No's. 7 and 8 sheets, which are enclosed herewith. The subsoil conditions encountered in both boreholes are, briefly, as follows:

Borehole No. 7

(G.L. 588)

0 - 1

1 - 3

3 -

Borehole No. 8

(G.L. 600)

0 - 3

3 - 29

29 -

Organics

silty sand/sandy silt  
trace of organics

clay to silty clay

silty sand to sand,  
compact

Borehole No. 7 was terminated in the clay to silty clay deposit at a depth of 23 ft., and Borehole No. 8 was terminated in the silty sand to sand layer at a depth of 35 ft. below ground level.

In situ vane tests indicate that in Borehole No. 7, the average undrained shear strength of the clay to silty clay stratum varies from 300 to 400 p.s.f., and in Borehole No. 8 it varies from 600 to 800 p.s.f. However, Borehole No. 8 is about 500 ft. away from Line C and 1000 ft. from Line A. It is believed that throughout the major portion of the length where an embankment is to be constructed, the subsoil condition will be similar to Borehole No. 7; however, in the close proximity of the hill, the subsoil conditions encountered in Borehole No. 8 may be more applicable.

### 5.3 Discussion and Recommendations

It is proposed to construct an embankment along the Line A, B, C, D or D' south of the hill which lies between Hwy. 638 and Echo River. Lines A, B and C traverse the hill, Line D travels along the side of the hill, and Line D' travels along the toe of the hill. The following table shows the maximum height of embankment for the five alternate alignments:

Line A	Maximum height of embankment	35 ft.
Line B	" " " "	20 ft.
Line C	" " " "	14 ft.
Line D	" " " "	10 ft.
Line D'	" " " "	5 ft.



As mentioned above, the shear strength of the clay to silty clay stratum varies from 300 to 400 p.s.f. The underlying subsoil can safely support an embankment up to 10 ft. in height with 2:1 side slopes built with granular material ( $\phi = 30^\circ$ , &  $\gamma = 125$  p.c.f.). Embankments greater than 10 ft. in height will require mid-height berms to ensure their stability. For every 1 ft. height in excess of the safe height of 10 ft., the length of berm will be 8 ft. Embankments greater than 18 ft. in height will require multiple berms.

If lightweight aggregate ( $\gamma = 90$  p.c.f.) is used for fill material, then the safe height of fill with 2:1 side slopes will be 14 ft. Beyond this height, stabilizing berms will be required. The mid-height berm will be 6 ft. long for every 1 ft. height in excess of the safe height of 14 ft. Fills in excess of 24 ft. in height will require multiple berms.

The underlying cohesive subsoil is compressible and will undergo settlements due to consolidation, over a long term period, under the weight of the embankment. Using the test results in this general area, the following settlements are estimated:-

<u>Height of Fill</u>	<u>Estimated Settlements</u>	
5 ft. (granular fill)	4 - 6 ins.	1 - 2 yrs.
	8 - 12 ins.	25 yrs.
10 ft. (granular fill)	8 - 10 ins.	1 - 2 yrs.
14 ft. (lightweight fill)	18 - 24 ins.	25 yrs.
18 ft. (granular fill)	18 - 24 ins.	1 - 2 yrs.
24 ft. (lightweight fill)	36 - 48 ins.	25 yrs.

Line D has a maximum height of embankment of 10 ft. and Line D' has a maximum height of embankment of 5 ft. and both can be constructed using granular fill and without stabilizing berms. From the stability and settlement considerations, Line D' is preferable over Lines A, B, C and D in this area.

## 6. ECHO RIVER STRUCTURE (Lines A, B, C, D & D')

### 6.1 Introduction

The site is located about 2 miles northeast of the town of <sup>Garden</sup> Echo Bay on the boundary of the township of MacDonald and the Golden River Indian Reserve. The Echo River originates from Echo Lake, about 3 miles upstream and terminates into Echo Bay about 1/2 mile downstream. At this place, the Echo River flows in an East to West direction, and is approximately 100 ft. wide. The old crossing consisted of a timber bridge. The timber bridge was damaged when a car went off it. It was then replaced with the present Bailey Bridge. It is reported that the timber bridge was supported in the middle by means of a pier resting on a rockfill, and that it had experienced considerable settlements.

The field work at this site consisted of two boreholes (BH No. 3 & 4) accompanied by dynamic cone penetration tests. Borehole No. 3 was put down on Lines D and D' which merge at this location, and Borehole No. 4 on Lines A, B and C which come very close together at this Site. (See plan of Boreholes).

### 6.2 Subsoil Conditions

The stratigraphy in the two boreholes and the results of in situ tests are shown on the Record of Borehole No's 3 and 4 Sheets attached herewith. The subsoil conditions in both boreholes are similar and consist of a deep deposit of soft to stiff clay to silty clay layer. The only difference between the two boreholes is in the upper 3 ft. immediately below ground surface. In borehole No. 4, the top 1 ft. consists of top soil followed by 2 ft. of sandy silt. In borehole No. 3 this is absent. Both boreholes were terminated in the clay to silty clay deposit at a depth of 132 ft. No other stratum throughout this depth was intersected, and no end-bearing stratum was encountered.

The in situ vane tests indicate that the undrained shear strength of the clay to silty clay deposit gradually increases with depth and varies from 400 p.s.f. in the upper portion to about 1400 p.s.f. at a depth of 100 ft. In the upper 60 ft. the shear strength increases from 400 p.s.f. to 800 p.s.f.

### 6.3 Discussion and Recommendations

#### 6.3.1 General

It is proposed to construct a structure at either of the crossing of new Highway No. 17 (Lines A, B, C, D or D') and the Echo River.

At this location, Lines A, B and C come down from the hill situated just to the south of the Echo River, and cross the river about 200 ft. west (downstream) of the existing bridge. The proposed grade along these lines is approximately 605 ft. while the ground level on either side of the river is about 595 ft. This means that Line A, B, or C will require an embankment about 10 ft. higher than the prevailing ground surface.

Lines D and D' travel along the toe of the hill which is situated South of the River, and consequently, its grade at the crossing is lower than the grade of Line A, B or C. The proposed grade of Line D or D' is about 592 ft., while the ground level on the south side is 587 ft. and on the north side is 590 ft. Therefore, Line D or D' will require embankments with a maximum height of 5 ft. above the prevailing ground surface.

Our recommendations for the structure foundations and the approach embankments are given in the following subsection.

#### 6.3.2 Structure Foundations

Due to the soft and compressible nature of subsoil, spread footing type foundations in the original ground are not recommended at this site.

Therefore, the entire structure should be supported on friction timber piles. For No. 14, treated timber piles driven at least 40 ft. in the original ground, an allowable load of 15 tons per pile may be assumed for estimate purposes.

### 6.3.3 Approach Embankments

#### 6.3.3.1 General

The proposed grade of Highway No. 17 is such that it will require embankments up to 10 ft. in height above the prevailing ground surface along Lines A, B and C, and up to 5 ft. in height along Line D or D'. These fills will be underlain by a deep, compressible clay to silty clay stratum.

#### 6.3.3.2 Stability Considerations

The proposed grade along Lines A, B and C is 605 ft., while the prevailing ground surface is at elevation 595 ft. The proposed grade along Line D or D' is about 592 ft. while the prevailing ground surface is at elevation 587 ft. (south) to 590 ft. (north). The maximum depth of water at the location of the existing bridge varies from 10 to 15 ft. The water level in the river at the time of investigation was at elevation 580 ft.

From the above, it is evident that along Lines A, B and C, the slopes in the transverse direction will be 10 ft. in height, but in the longitudinal direction will be 35 - 40 ft. above the river bed. Along Line D or D' the slopes in the transverse direction will be 5 ft. in height, but in the longitudinal direction will be at 22 - 27 ft. above the river bed.

Approximate analyses have been carried out in terms of total stress, to determine the stability of the fill sections. The soil properties for the natural subsoil, assumed for computation purposes, are as follows:

### Fill Material (Granular type)

#### Subsoil

0 - 25 ft.	Cu = 400 p.s.f.
25 - 60 ft.	Cu = 600 p.s.f.
60 - 100 ft.	Cu = 1200 p.s.f.

The results of the stability analyses, for an acceptable factor of safety of 1.3, are summarized below:

- Fills up to 13 ft. in height are safe.
- Forward slopes greater than 13 ft. in height (measured from river bed) will require mid-height berm.
- The mid-height berm will be 7 ft. long for every 1 ft. height in excess of the safe height of 13 ft.
- Forward slopes greater than 24 ft. in height will require multiple berms.

#### 6.3.3.3 Settlement Considerations

The underlying highly compressible, cohesive stratum will undergo settlements due to consolidation, over a long-term period, under the weight of the approach embankments. Settlements have been estimated, using the results of the tests conducted in the past, on soil in this general area, and are as follows:

<u>Height of Fill</u>	<u>Estimated Settlements</u>	
5 ft. (Line D or D')	4 - 6 inches	4 yrs.
	8 - 12 inches	25 yrs.
10 ft. (Line A, B or C)	9 - 12 inches	4 yrs.
	18 - 24 inches	25 yrs.

It would be advantageous to construct the embankments first and leave them in place for as long a period as possible, prior to constructing the structure.

## 6.4 Conclusions

1. Line A, B or C has a grade such that the height of slope in the longitudinal direction is in excess of 35 feet (above river bed). This will require long multiple berms in forward directions. This will result in a much longer structure. The height of slopes in the longitudinal direction for Line D or D' is 22 - 27 ft. This will require relatively short berms, depending upon the actual height, resulting in a shorter structure compared to Lines A, B or C.

2. The proposed grade of Line A, B or C is about 10 ft. above the surrounding ground level, while the proposed grade of Line D or D' is only 5 ft. (at its maximum) above the ground level. As mentioned in sub-section 6.3.3.3, the settlements under the fills for Line D or D' will be half as much as the settlements under the fills for Line A, B or C. This will considerably minimize the maintenance of the approaches.

For the aforementioned reasons, Line D or D' is preferable at this site from the foundation point-of-view.

## 7. RECOMMENDED ALIGNMENT

As mentioned earlier, in the vicinity of the CPR crossing Line 5 has overwhelming advantage over all other lines. Along this line the structure foundations i.e. piles will be relatively short, no long counterbalancing berms will be required and no settlement problems will occur. As a result the structure will be much smaller and will require little or no maintenance.

At the location of the proposed Hwy. 638, no particular line offers any advantage over others, as they are within 200 ft. of each other, and the subsoil conditions are relatively uniform. Moreover, there is no improvement in subsoil conditions to the south of the existing Hwy. 638, therefore, realignment of Hwy. 638 would not be of any benefit.

Between the CPR crossing and Hwy. 638 the embankment along Line 5 should be kept as low as possible.

Near Station 150, Line D' is preferable over all other lines, because it requires only 5 ft. fill height at its maximum. Therefore, no stability problems are presented and settlement and subsequent maintenance problems are minimized.

At the Echo River Line D' is the most suitable, because the maximum height of forward slopes is minimum at this place. The river is 10 - 15 ft. deep, therefore it presents stability problems in the forward direction, which will require berms and result in a longer and more expensive structure.

North of Hwy. 638 Alignment D' should be followed because of its low profile.

In general, wherever embankments are required the grade of Hwy. 17 should be kept as low as possible.

#### 8. MISCELLANEOUS

The field work for this project was carried out under the supervision of Mr. H. Shah, Project Engineer and Mr. C. McKercher, Student Technician, between the period of Feb. 24 to April 29, 1975.

The drilling equipment used was owned and operated by Master Soil Investigation Ltd. and Atcost Drilling Co.

This report was prepared by Mr. A. Prakash, Senior Engineer and reviewed by Mr. M. Devata, Supervising Engineer.

AP/rjc

July 1975

*A. Prakash*

A. PRAKASH  
Senior Engineer

*M. Devata*

M. DEVATA  
Supervising Engineer



## APPENDIX



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

W.P. 903-72-01 LOCATION CO-ORDS. 884,600N 984,530 E ORIGINATED BY HS  
DIST. 18 HWY. 17 BORING DATE Feb. 24 -26, 1975 COMPILED BY MM  
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger & Cone Test CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		SHEAR STRENGTH					WATER CONTENT %				
							20 40 60 80 100					$w_p$ $w$ $w_L$				
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
						400 800 1200 1600 2000										
617.±	Ground Level															
0.0	Topsoil															
614.0	Sandy Silt															
3.0	Desiccated or Frozen		1	SS	2	610										
			2	TW	PH											
			3	SS	1	600										
	Silty Clay		4	SS	1											
	to		5	SS	1	590										
	Clay		6	SS	1											
			7	TW	PH	580										
			8	SS	1											
			9	SS	1	570										
			10	TW	PH	560										
556.0			11	SS	22											
61.0	Silty fine sand to sandy silt.		12	SS	29	550										
547.2	Compact to Dense		13	SS	70											
69.8	End of Borehole Refusal					540										

20  
15 5 % STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 2

W.P. 903-72-01

LOCATION CO-ORDS. 884,770 N 984,680 E

ORIGINATED BY HS

DIST. 18 HWY. 17

BORING DATE February 26-28, 1975

COMPILED BY MM

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger and Cone Test

CHECKED BY *Lo*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
619.1	Ground Level															
0.0	Topsoil															
616.0	Sandy Silt															
3.0	Desiccated or Frozen Firm		1	SS	4											
			2	SS	1											
			3	SS	1											
	Soft		4	TW	PH											
	Silty Clay to		5	SS	1											
	Clay		6	SS	1											
			7	SS	1											
	Firm		8	TW	PH											
			9	SS	2											
			10	SS	3											
560.0																
59.0	Silty fine sand to sandy silt.		11	SS	34											
	Compact to Dense															
551.3			12	SS	30											
67.7	End of Borehole Refusal															

## RECORD OF BOREHOLE NO 3

W.P. 903-72-01

LOCATION CO - ORDS. 902,260 N 987,800 E

ORIGINATED BY HS

DIST. 18 HWY. 17

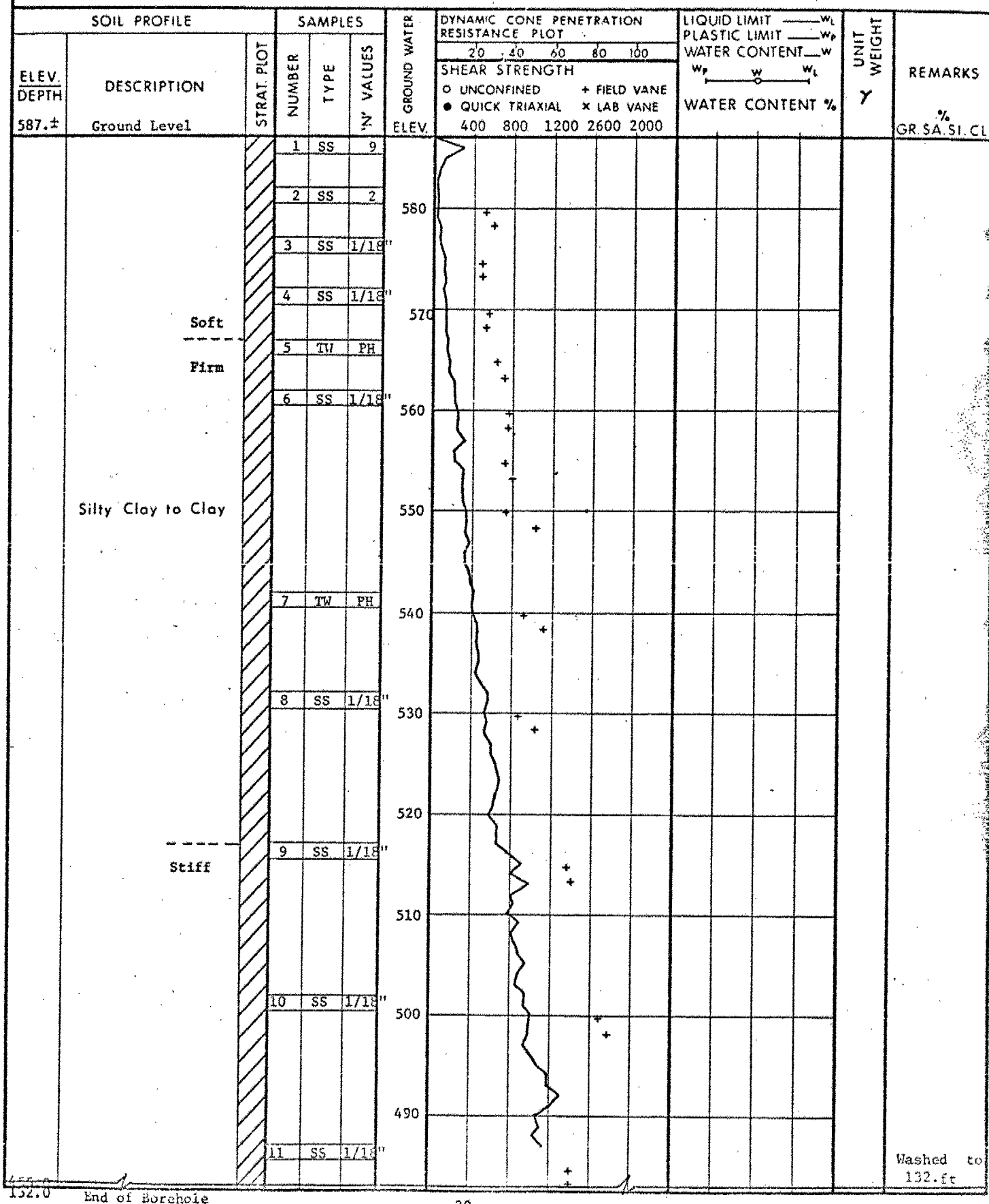
BORING DATE March 1-3, 1975

COMPILED BY MM

DATUM Geodetic

BOREHOLE TYPE Hollow Type Auger &amp; Cone Test

CHECKED BY



CHECKED BY \_\_\_\_\_

20  
15  $\phi$  5 % STRAIN AT FAILURE  
10

## ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

## RECORD OF BOREHOLE No 5

W.P. 903-72-01

LOCATION CO-ORDS. 894,085N 988,030 E

ORIGINATED BY HS

DIST. 18 HWY. 17

BORING DATE March 6-7, 1975

COMPILED BY AP

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger and Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N° VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
584.0	Ground Level															
583.0	Topsoil															
1.0	Sandy silt with organics.															
579.0																
4.0	silty clay. to Clay		1	SS	1/18"	580										M/C 97% Org. 1.8%
	Soft		2	SS	1/18"											
			3	SS	1/18"	570										
564.0	Clayey silt to silt.		4	TW	PH											
20.0	Firm		5	SS	2	560										
557.0	Silt to sandy silt.		6	SS	10											0 5 91 4
27.0	Loose to Compact		7	SS	23	550										0 18 78 4
546.0			8	SS	1/18"											
38.0	silty clay to clay (stratified)		9	SS	1/18"	540										
	with silt layers.		10	TW	PH											
	Firm to Stiff		11	SS	3	530										
						520										
						510										
						500										
494.0																
90.0	End of Borehole															

## Cone Test

90-100 Ft. 9-9-9-10-12-12-13-13-13-14  
 100-110 Ft. 16-21-22-22-25-24-23-24-24-24  
 110-120 Ft. 51-50-50-49-52-42-39-37-37-34  
 120-130 Ft. 40-40-36-36-37-39-37-35-39-40

# RECORD OF BOREHOLE NO 6

W.P. 903-72-01

LOCATION CO-ORDS. 893,295 N 987,740 E

ORIGINATED BY HS

DIST. 18

HWY. 17

BORING DATE March 8-10, 1975

COMPILED BY AP

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger & Cone Test

CHECKED BY


SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$		UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$		
589 ±	Ground Level														
588.	Topsoil														
586.	Sandy silt with orgs.														
3.0	silty clay to clay  Soft		1	AS	-										
			2	AS	-										
			3	SS	1/18"	580									
			4	TW	PH										
			5	AS	-	570									
			6	SS	1/18"										
			7	SS	1/18"	560									
557.0	Silt to sandy silt  Loose to Compact		8	SS	13										
32.0			9	SS	13	550									
			10	SS	1										
541.0	silty clay to clay (stratified)  with silt layers  Firm to Stiff		11	SS	1/18"	540									
48.0			12	SS	1/18"										
						530									
			13	SS	5										
						520									
			14	SS	2	510									
504.	End of Borehole					500									

## Cone Test

85 - 90 Ft. 3-4-6-8-9  
 90-100 Ft. 11-12-11-13-13-15-15-16-21-25  
 100-110 Ft. 25-26-28-28-27-25-26-27-30-36  
 110-120 ft. 47-46-45-45-41-39-37-36-39-43  
 120-130 Ft. 45-40-41-44-55-58-53-51-51-51

# RECORD OF BOREHOLE NO 7

W.P. 903-72-01 LOCATION CO-ORDS. 897,835 N 988,245 E ORIGINATED BY HS  
 DIST. 18 HWY. 17 BORING DATE March 11, 1975 COMPILED BY CNcK  
 DATUM Geodetic BOREHOLE TYPE Auger and sample CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	SHEAR STRENGTH ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE 400   800   1200   1600   2000				
588.0	Ground Level															
587.0	Organics															
585.0	fine sandy silt, trace of organics & clay.		1	AS	-											
3.0	silty clay. to clay  Soft					580	+									
							+									
			2	AS	-		+									
			3	AS	-		+									
565.0			4	AS	-		+									
23.0	End of Borehole					560										

W.P. 903-72-01

LOCATION CO-ORDS. 897,600N 989,555 E

ORIGINATED BY HS

DIST. 18 HWY. 17

BORING DATE March 11, 1975

COMPILED BY CMcK

DATUM Geodetic

BOREHOLE TYPE Auger and sample

CHECKED BY \_\_\_\_\_

15  $\frac{20}{10}$  5 % STRAIN AT FAILURE



## RECORD OF BOREHOLE NO 9

W.P. 903-72-01

LOCATION CO-ORDS. 884,970 N 984,970 E

ORIGINATED BY C. McK

DIST. 18 HWY. 17

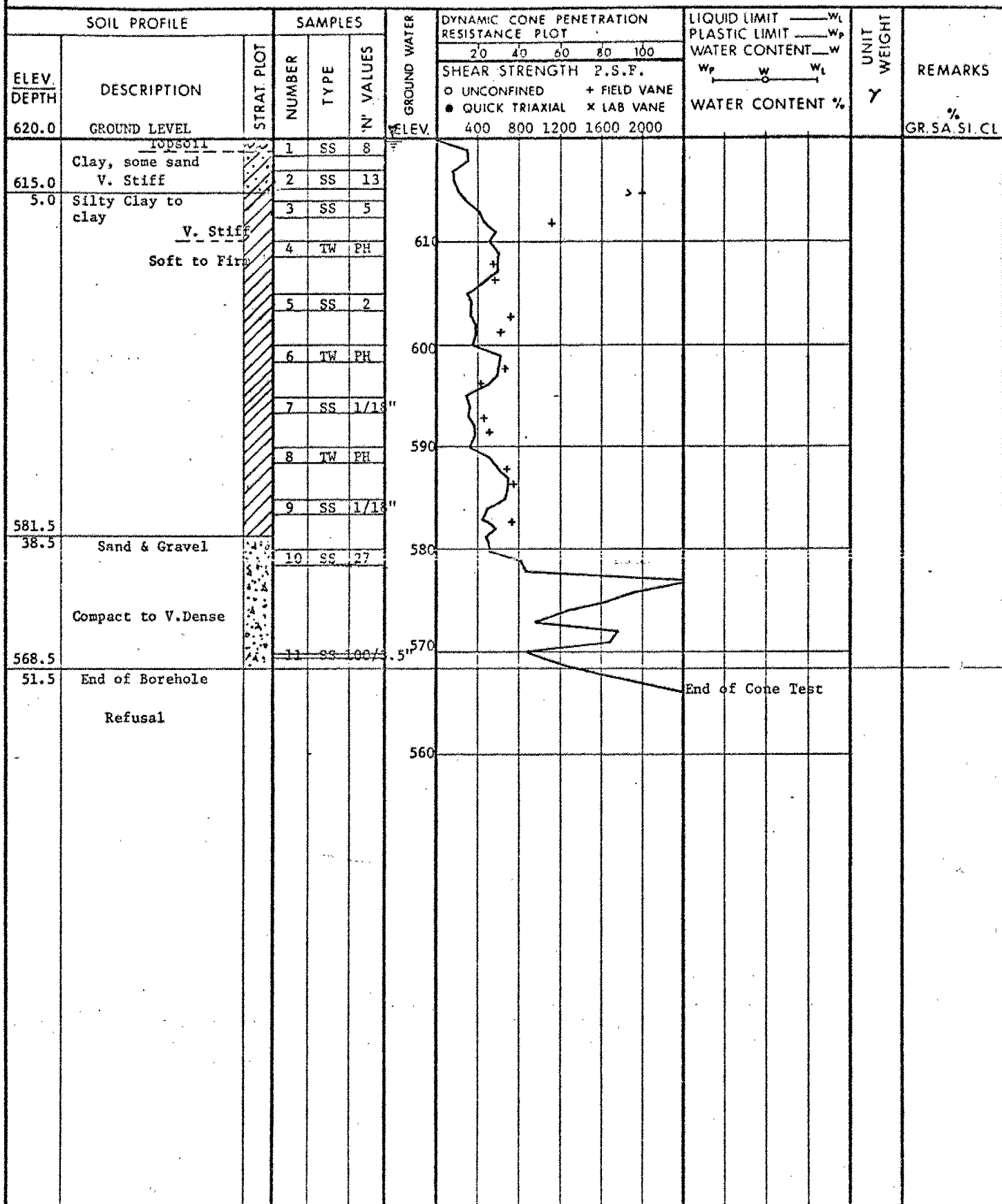
BORING DATE MARCH 22, 1975

COMPILED BY C. McK.

DATUM GEODETIC

BOREHOLE TYPE AUGER AND SAMPLE WITH CME 55 MACHINE

CHECKED BY



RECORD OF BOREHOLE NO 10

W.P. 903-72-01

LOCATION CO-ORDS. 886,550N 984,020E

ORIGINATED BY H.S.

DIST. 18 HWY. 17

BORING DATE APRIL 23-24, 1975

COMPILED BY H.S.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM - 2 3/4"

CHECKED BY H.S.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
614.0	GROUND LEVEL															
0.0	topsoil															
609.5	Silty Sand Very Loose		1	SS	2	610										
4.5	Organic Clay, Occ. silty sand layers		2	SS	3											
603.0	Very Soft		3	SS	10											
11.0	Silty Clay to Clay — Stiff Soft		4	SS	1/18"	600										
			5	TW	PM											
			6	SS	1/18"	590										
			7	SS	1/18"											
			8	SS	2/18"	580										
			9	TW	PM											
			10	SS	2	570										
			11	SS	1	560										
			12	SS	2	550										
			13	SS	4	540										
514.0						530										
100.0						520										
	continued....															



CHECKED BY 2/5

15 <sup>20</sup> 5 % STRAIN AT FAILURE



RECORD OF BOREHOLE No 12

W.P. 903-72-01 LOCATION CO-ORDS. 883,640 N 985,145 E ORIGINATED BY H.S.  
 DIST. 18 HWY. 17 BORING DATE APRIL 26, 1975 COMPILED BY H.S.  
 DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM - 2 3/4" CHECKED BY H.S.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$W_P$	$W$	$W_L$		
617.0	GROUND LEVEL															
0.0	Silty Sand to Sand  Very loose to loose		1	SS	10											
			2	SS	4	610										
			3	SS	2											
			4	SS	4											
			5	SS	2											
			6	SS	6	600										
596.0			7	SS	11											
21.0	Sand & Gravel compact v. dense  with cobbles  and few boulders		8	SS	71	590										
			9	SS	100/	6"										
			10	SS	76/	3"										
			11	SS	200/	3"										
570.0						570										
47.0	End of Borehole Refusal to Augering probable boulder					560										

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE NO 13

W.P. 903-72-01

LOCATION CO-ORDS. 883,400N 985,135 E

ORIGINATED BY HS

DIST. 18 HWY. 17

BORING DATE APRIL 28, 1975

COMPILED BY H.S.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

CHECKED BY H.S.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W			UNIT WEIGHT γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>		
615.0	GROUND LEVEL															
0.0	Topsoil		1	SS	10											
	Silty Sand to Sand		2	SS	2/18	610										
	Very Loose to loose		3	SS	4											
			4	SS	4	600										
			5	SS	4											
590.5																
24.5	Sand & Gravel		6	SS	12	590										
	Compact with cobbles		7	SS	96											
579.5			8	SS	180	580										
35.5	End of Borehole Refusal to Augering Probable Boulder															
						570										

RECORD OF BOREHOLE NO 14

W.P. 903-72-01 LOCATION CO-ORDS. 883,520N 985,240 E ORIGINATED BY H.S.  
 DIST. 18 HWY. 17 BORING DATE APRIL 28, 1975 COMPILED BY H.S.  
 DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4" CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$		UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$W_P$	$W$		
616.0	GROUND LEVEL														
0.0	Topsoil														
613.0			1	SS	7										
3.0	Clay, Trace of Sand Firm		2	SS	2	610									
607.0															
9.0	Silty Sand to Sand  Very Loose		3	SS	2										
			4	SS	4	600									
592.0															
24.0	Sand and Gravel with V. Loose cobble V. Dense		5	SS	2	590									
583.5			6	SS	69										
32.5	End of Borehole Refusal to Augering Probable Boulder					580									



W.P. 903-72-01

LOCATION CO-ORDS. 883,565 N 985,040 E

ORIGINATED BY H.S.

DIST. 18 HWY. 17

BORING DATE APRIL 28, 1975

COMPILED BY H.S.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

CHECKED BY NS

15 <sup>20</sup> 5 % STRAIN AT FAILURE

W/P 903-72-01

LOCATION CO-ORDS. 883,860N 985,145 E

ORIGINATED BY H.S.

DIST. 18 HWY. 17

BORING DATE APRIL 28-29, 1975

COMPILED BY H.S.

DATUM      GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2-3/4"

CHECKED BY N.T.

15  $\phi$  5 20 % STRAIN AT FAILURE

RECORD OF BOREHOLE NO 17

W.P. 903-72-01

LOCATION CO-ORDS. 884,095N 985,120E

ORIGINATED BY H.S.

DIST. 18 HWY. 17

BORING DATE APRIL 29, 1975

COMPILED BY HS

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

CHECKED BY M.J.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS  % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
623.0	GROUND LEVEL															
0.0	Sand, Some gravel		1	SS	31	620										
			2	SS	17											
613.5	Compact to Dense		3	SS	34											
9.5	Silty Sand to Sand, Some Gravel Loose to Compact		4	SS	15	610										
			5	SS	10											
			6	SS	12	600										
			7	SS	6											
			8	SS	10	590										
593.0	Cobbles															
40.0	End of Borehole Refusal to Auger probable boulder					580										

OFFICE REPORT ON SOIL EXPLORATION

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 18

W.P. 903-72-01

LOCATION CO-ORDS. 882,915N 984,910 E

ORIGINATED BY H.S.

DIST 18 HWY. 17

BORING DATE APRIL 29, 1975

COMPILED BY H.S.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

CHECKED BY H.S.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
615.0	GROUND LEVEL															
0.0	Topsoil															
612.0			1	SS	2	610	+S=3.5									
3.0	Silty Clay to Clay Soft Firm		2	SS	1		+S=2.4									
			3	TW	PM		+S=2.6									
			4	SS	2	600	+S=7.5									
							+S=6.2									
596.0							+S=5.7									
19.0	Silty Sand to Sand Very Loose		5	SS	4	590										
			6	SS	4											
			7	SS	2											
581.8			8	SS	140/2"											
33.2	End of Borehole Refusal to Augering Probable Boulder					580										

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE NO 19

W.P. 903-72-01

LOCATION CO-ORDS. 882,470 N 984,665 E

ORIGINATED BY H.S.

DIST. 18 HWY. 17


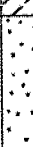
BORING DATE APRIL 29, 1975

COMPILED BY H.S.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

CHECKED BY *M. S.*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$		UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100					$W_P$ $W$ $W_L$			
							SHEAR STRENGTH P.S.F.					WATER CONTENT %			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
						400 800 1200 1600 2000									% GR. SA. SI. CL.
615.0	GROUND LEVEL														
0.0	<u>topsoil</u> Silty Clay to Clay		1	SS	2	610		+S=13.5							
			2	SS	1/18"			+S=13.3 +S=3.7 +S=4.0 +S=2.4							
			3	SS	1/18"	600		+S=6.9 +S=4.5							
			4	SS	1/18"	"		+S=3.3 +S=4.0							
	Soft Firm		5	SS	2/18"	590		+S=8.5 +S=5.1							W.L. not established
578.0			6	SS	3	580									
37.0	Sand, some gravel														
	Loose		7	SS	6	570									
568.5															
46.5	End of Borehole					560									

OFFICE REPORT ON SOIL EXPLORATION

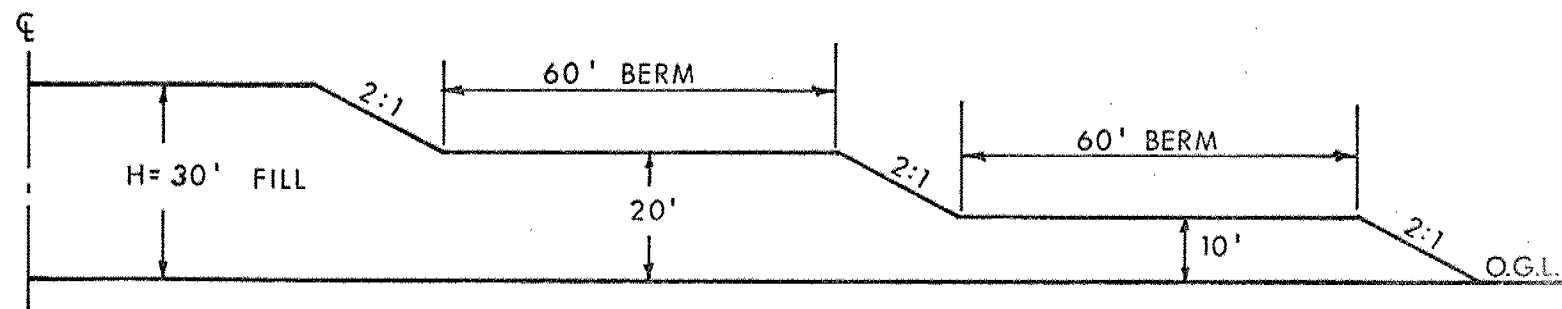
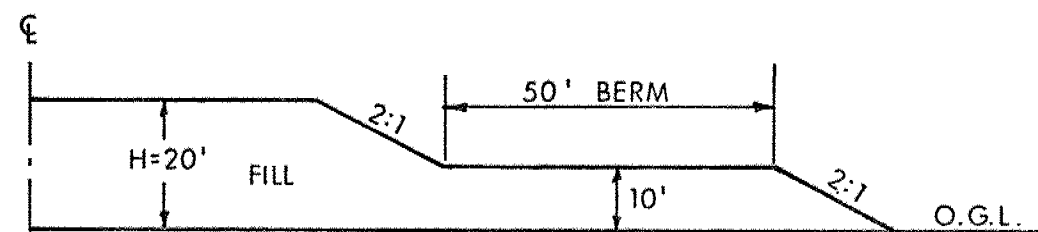
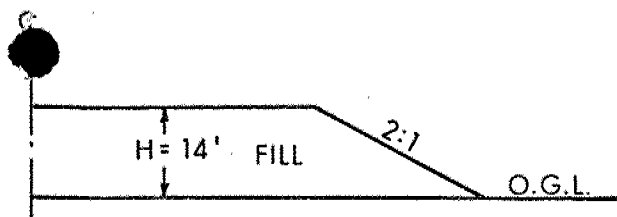


FIG. 1

W.P. 903-72-01

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION RESISTANCE

'N' STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

# ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

## SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
$S_r$	DEGREE OF SATURATION
$w_L$	LIQUID LIMIT
$w_p$	PLASTIC LIMIT
$I_p$	PLASTICITY INDEX
$w_s$	SHRINKAGE LIMIT
$I_L$	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
$I_c$	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
$e_{max}$	VOID RATIO IN LOOSEST STATE
$e_{min}$	VOID RATIO IN DENSEST STATE
$I_D$	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY $D_r$ IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
$m_v$	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
$c_v$	COEFFICIENT OF CONSOLIDATION
$C_c$	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
$T_v$	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

## GENERAL

$\pi$	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

## STRESS AND STRAIN

u	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

## EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

## FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
$k_s$	MODULUS OF SUBGRADE REACTION

## SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



Coulter

705

248 2011

prop  
util.

chief

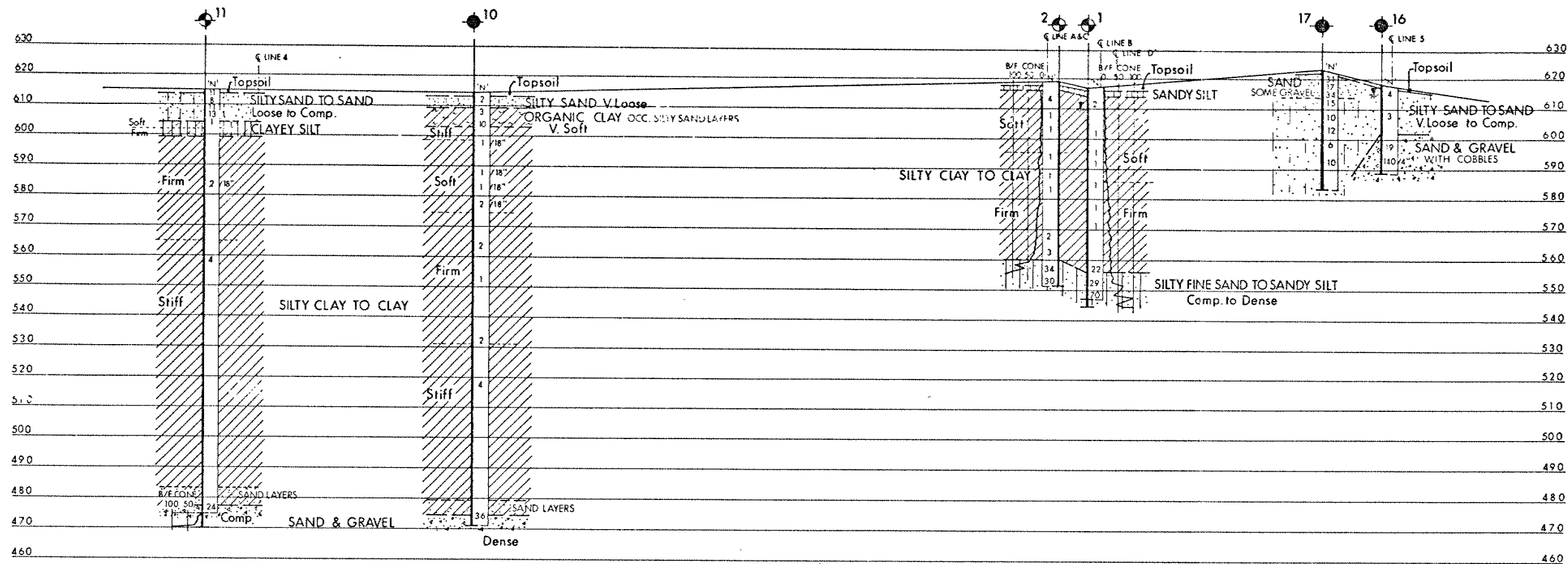
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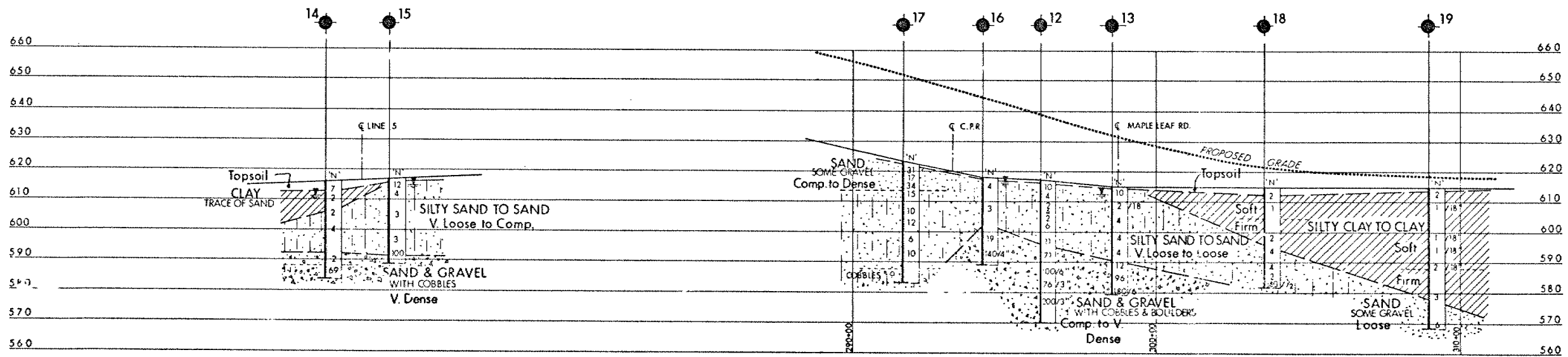
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# OVERSIZE DRAWING

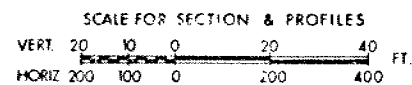


PROFILE ALONG C.P.R.



SECTION A-A

PROFILE ALONG PART OF LINE 5



SEE DRAWING 9037201-A

KEY PLAN

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Resistance Test  
B/F CONE - Blows/Ft. Cone Test (350 ft. lbs. energy/blow)
- ⊕ Bore Hole & Cone Test
- ⊕ Water Levels established at time of field investigation, Feb. & Apr. 1975
- ⊕ Water Levels not established in Bore Holes 2, 10, 11, 12, 17, 18 & 19

NO.	ELEVATION		

NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

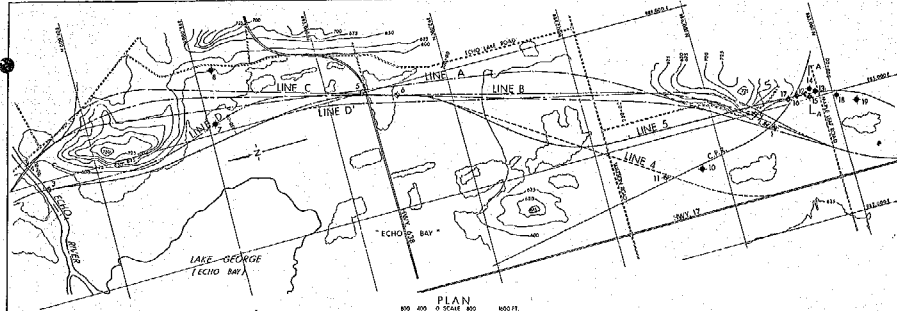
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

PRELIMINARY INVESTIGATION  
FEASIBILITY STUDY  
**ECHO RIVER to BAR RIVER ROAD**  
HIGHWAY NO 17 (PROPOSED 4 LANES) DIST NO 18  
DIST. OF ALGOMA  
TWP. MACDONALD 10' CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD A P	CHECKED	W.P. NO	903-72-01	DRAWING NO	9037201-B
DRAWN SO	CHECKED	A.C. NO		DATE	10 JULY 1975
DATE	10 JULY 1975	S.T. NO		BRIDGE DRAWING NO	
APPROVED		CONT NO			

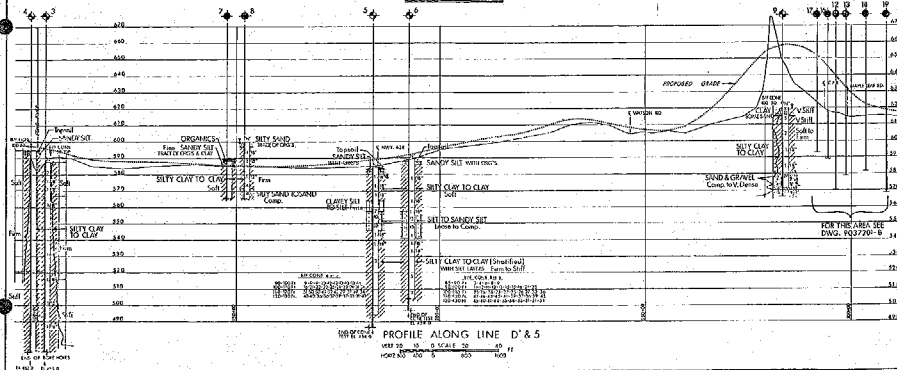
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GEOCRETS No.



**LEGEND**

- ◆ Bare Hole
- ◆ Chopped Core Penetration Provenance Test (COP) (Borehole Core Test (BCT) equivalent)
- ◆ Bare Hole & Core Test
- ◆ Water Levels established at time of field investigation, March 1976
- ◆ Water Levels established in Bare Holes 3, 4 & 8

NO.	ELEVATION	CO-ORDINATES
		NORTH EAST
1	617.5	681.574
2	615.6	681.750
3	620.7	681.850
4	620.7	681.850
5	620.7	681.850
6	620.7	681.850
7	620.7	681.850
8	620.7	681.850
9	620.7	681.850
10	620.7	681.850
11	620.7	681.850
12	620.7	681.850
13	620.7	681.850
14	620.7	681.850
15	620.7	681.850
16	620.7	681.850
17	620.7	681.850
18	620.7	681.850
19	620.7	681.850



**NOTE**

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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO  
 (INCORPORATING MINISTRY OF HIGHWAYS - ONTARIO) - 73, RENOVES, SECT. 7

FEASIBILITY STUDY

**ECHO RIVER TO BAR RIVER ROAD**

HIGHWAY NO. 17 (PROPOSED & LINES), DIST. NO. 18

CO. DIST. OF ALGOMA

TOW. JACONDA

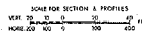
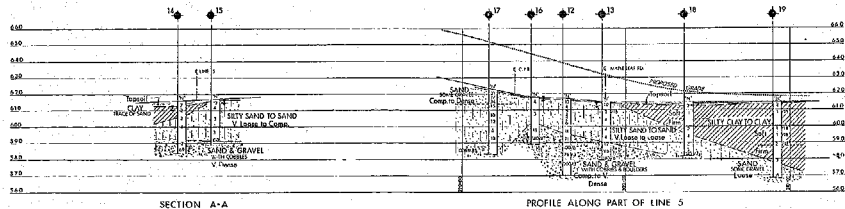
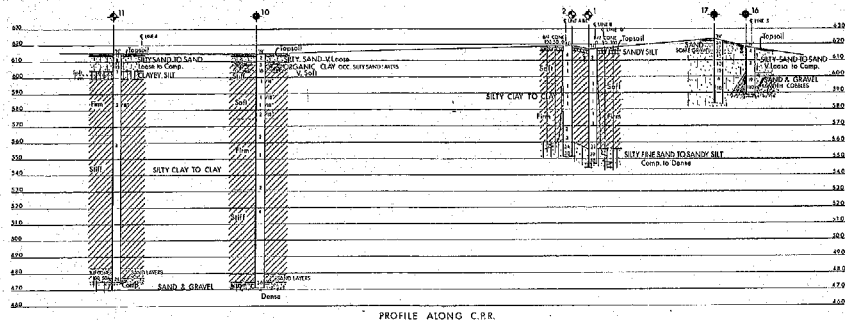
**BORE HOLE LOCATIONS & SOIL STRATA**

Location: ECHO RIVER ROAD, DIST. NO. 18, CO. DIST. OF ALGOMA

Sheet: 5-D (COVERED) AT NO. 18-22-61

DATE: 3 JULY 1976 SITE NO. 9037201-A

APP'D: [Signature] ROAD NO. [Blank]



PROFILE ALONG PART OF LINE 5

SEE DRAWING 9037201-A



### KEY PLAN

### LEGEND

- ◆ Race Hole
- ◆ During Crop Penetration Resistance Test  
SIF CORP. Fourty Case Test 12001 No remarkable
- ◆ Soil Hole & Core Test
- ◆ Water Levels established on line  
of field investigation Feb. & Apr. 1973
- ◆ Water Levels set established in Deep  
Holes 2, 10, 11, 12, 19, 18 & 19

[illegible]

- NOTE -

The boundaries between and within have been established only at some local locations. Between some holes the boundaries are assumed from geological evidence.

001	002	003	004	005	006	007	008	009	010	011	012	013	014	015	016	017	018	019	020	021	022	023	024	025	026	027	028	029	030	031	032	033	034	035	036	037	038	039	040	041	042	043	044	045	046	047	048	049	050	051	052	053	054	055	056	057	058	059	060	061	062	063	064	065	066	067	068	069	070	071	072	073	074	075	076	077	078	079	080	081	082	083	084	085	086	087	088	089	090	091	092	093	094	095	096	097	098	099	100
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MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO  
ENGINEERING, SURVEYING, GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

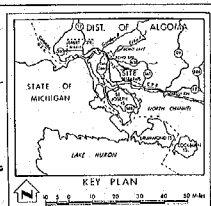
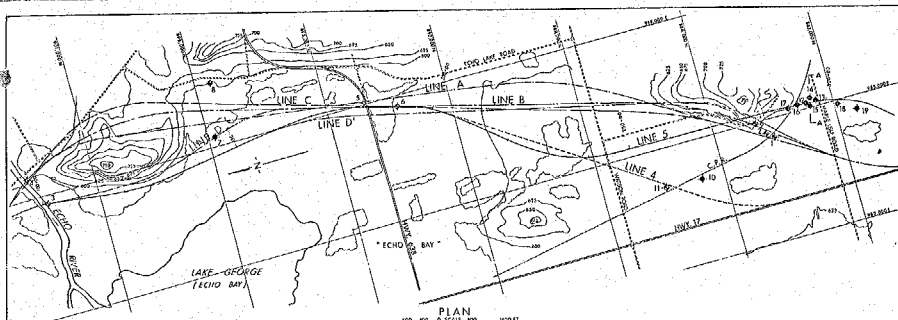
# ECHO RIVER to BAR RIVER ROAD

HIGHWAY NO. 17 (PROPOSED 4 LANES) DIST NO. 18  
 DIST. OF ALGOMA  
 TWP. MACDONALD LOT 101 CON.

BORE HOLE LOCATIONS & SOIL STRATA			
SHEET NO. 1	CHECKED	DATE 10-22-81	DESIGNED BY

DRAWN BY	DATE	9037201-B
10 JULY 1975	SCALE	MOORE DRAWING NO

an Project	Cover No.	
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# LEGEND

Base line

Exposure Core Penetration Resistance Test

Base hole & Core Test

Water levels established at time of field investigation, March 1972

Water levels established at Base Station, S.C.A.B.

NO. ELEVATION

CD - CRESTWATER

NO. 1

CD - CRESTWATER

NO. 2

CD - CRESTWATER

NO. 3

CD - CRESTWATER

NO. 4

CD - CRESTWATER

NO. 5

CD - CRESTWATER

NO. 6

CD - CRESTWATER

NO. 7

CD - CRESTWATER

NO. 8

CD - CRESTWATER

NO. 9

CD - CRESTWATER

NO. 10

CD - CRESTWATER

NO. 11

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NO. 12

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NO. 13

CD - CRESTWATER

NO. 14

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NO. 15

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NO. 16

CD - CRESTWATER

NO. 17

CD - CRESTWATER

NO. 18

CD - CRESTWATER

NO. 19

CD - CRESTWATER

NO. 20

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NO. 21

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NO. 22

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NO. 23

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NO. 26

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NO. 31

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NO. 32

CD - CRESTWATER

NO. 33

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NO. 34

CD - CRESTWATER

NO. 35

CD - CRESTWATER

NO. 36

CD - CRESTWATER

NO. 37

CD - CRESTWATER

NO. 38

CD - CRESTWATER

NO. 39

CD - CRESTWATER

NO. 40

CD - CRESTWATER

NO. 41

CD - CRESTWATER

NO. 42

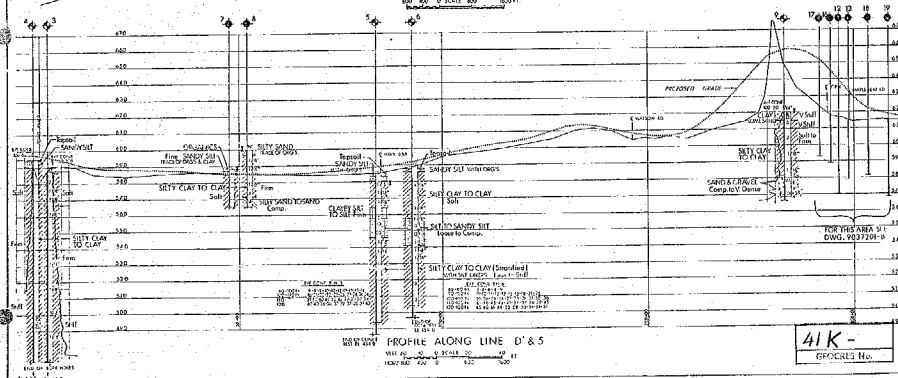
CD - CRESTWATER

NO. 43

CD - CRESTWATER

NO. 44

CD - CRESTWATER



**NOTE**

The boundaries between soil series have been established only at Base hole locations. Between these holes the boundaries are deduced from geologic evidence.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
FEDERAL HIGHWAYS DIVISION  
REGULATORY STUDY  
**ECHO RIVER TO BAR RIVER ROAD**  
HIGHWAY NO. 17 (PROPOSED 4 LANES) DIST NO. 18  
DIST. OF ALCOA  
W. MACDONALD  
NOT COM  
**BORE HOLE LOCATIONS & SOIL STRATA**  
Survey A & B (See also 903-72-51)  
Drawn S.O. (See also 903-7201-A)  
Date: 2 JANU 1972  
Scale: 1" = 100' HORIZ.  
Vertical: 1" = 10'

41K-  
GPORES H.O.