

## MEMORANDUM

To: Mr. A. M. Toye,  
Bridge Engineer,  
Bridge Division.

FROM: Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. F.I. Hewson  
Consultant Liason Engineer.

DATE: October 9, 1964.

OUR FILE REF.

IN REPLY TO

## SUBJECT:

## FOUNDATION INVESTIGATION REPORT

For

Proposed Carlton St. Tunnel under  
the Welland Canal at St. Catherines,  
Ontario.

W.P. 444-64

W.J. 64-F-53 - 1

Enclosed please find our detailed report on the subsoil conditions existing at the above mentioned proposed tunnel location.

It should be noted that slight revisions have been made to the preliminary borelog sheets #1-#3 which were given to Mr. N.D. Lea of General Engineering Co. Ltd. on August 26th.

We believe that the factual information and recommendations contained in the report will be adequate for immediate design purposes, however certain facets of the investigation dealing with ground water conditions and effective shear strength of the cohesive layers are still under study in the field and in the laboratory and will be fully reported at a later date.

Should any additional information be required or should any points in the report require clarification, please feel free to call our office.

AGS/PB  
Attach.

cc: Messrs. A.M. Toye (2)  
H.A. Tregaskes  
H.D. McMillan

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PRINCIPAL FOUNDATION ENGINEER

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# FOUNDATION INVESTIGATION REPORT

FOR

Proposed Carlton St. Tunnel under  
the Welland Canal at St. Catharines,  
Ontario.

W.P. 444-64

W.J. 64-F-53

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## 1. INTRODUCTION:

A preliminary foundation investigation, at the site of the proposed Carlton St. Tunnel, consisting of three sampled boreholes was requested by N. D. Lea, General Engineering Co. Ltd. in a letter to A. Rutka, Materials and Research Engineer, dated June 24 1964. Following this request an investigation was commenced by this Section on July 6 and was completed by August 26 at which time the resulting information was made available to Mr. Lea in the form of detailed borelog sheets.

During the preliminary work it became apparent that a more detailed programme was necessary and due to the urgency of the tunnel project it was decided to complete the preliminary and detailed programmes in one operation. A memo from F. I. Hewson, Consultant Liason Engineer dated August 6, requested that the Foundation Section proceed accordingly.

The final form of the detailed soils programme was decided upon at a meeting between the Foundation Section and N. D. Lea held on August 25, 1964. Subsequent meetings were held during the course of the fieldwork to review the work progress.

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This report contains the results of both field and laboratory tests carried out during the course of the investigation together with a detailed description of subsoil conditions existing at the site.

## 2. DESCRIPTION OF SITE:

The site is located in the west part of St. Catherines some 1100' south of the existing bascule type bridge which carries Carlton St. over the Welland Canal. The proposed  $Q_L$  of the tunnel intersects the  $Q_L$  of Lock #2 at Sta. 2N+16-Seaway Drainage. At this location the Welland Canal is some 700' wide and about 35' deep, the mean water level during the navigation season being el. 335.5. During boring operations for B.H.s #3 and #7 it was observed that the water level in the canal fluctuated approximately  $\pm 1.5$  feet about the mean level in periods of less than 30 minutes. The canal is contained by earth dykes which are approximately 20' higher than the surrounding terrain the latter being generally flat with an average ground surface elevation of approximately 320.0.

A geological report for this project has been prepared by Professor Deane of Toronto University and should be referred to if geological information is required.

## 3. FIELD INVESTIGATION PROCEDURE:

The fieldwork consisted of thirteen groups of boreholes, each group consisting of from one to six separate borings the grand total being forty seven. The pattern of the various groups



3. FIELD INVESTIGATION PROCEDURE: (cont'd)...

was arranged so that the minimum distance between adjacent holes was about four feet. At each of Boreholes #1 - #4, three separate holes were drilled, one being for the purpose of recovering continuous samples in the clay layers and the other two being for the purpose of carrying out field vane tests in the clay layers at intervals in elevation of nine inches. Boreholes #5, #7, #11 and #13 consisted of a single hole only in which samples were recovered at various intervals and field vane tests were carried out where applicable at elevations 12" below the various sample depths. Boreholes #6, #8, #9, #10 and #12 each consisted of a group of holes the deepest of which was used for sampling and vane testing the remainder of each group being drilled for piezometer installation only.

Boreholes #1 and #2 were drilled partly by means of a continuous flight auger 4-inch diameter and partly by means of conventional diamond drilling equipment adapted for soil sampling purposes. All other boreholes were drilled using the diamond drill only. In all instances where the diamond drill was used the borehole was advanced by means of wash-boring methods using NX casing down to the bottom of the clay layers and BX casing beyond this depth. Boreholes #3 and #7 were carried out in the canal in which case the drilling machine was mounted on a raft.

Undisturbed samples in the cohesive layers were obtained by means of 2-inch I.D. shelly tubes fitted with a piston sampler which were pushed into the soil either hydraulically

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3. FIELD INVESTIGATION PROCEDURE: (cont'd) ...

by the drilling machine or by hand. 'Disturbed' samples in all layers were recovered by means of a standard 2-inch O.D. split spoon sampler driven into the soil with a 140 lb. hammer falling freely a distance of 30 inches. AX+ rock core samples were obtained in B.H.s #2, #5 and #1.

Field vane tests were carried out in the cohesive layers using a standard D.H.O. field vane having dimensions such that the shear strength equals twenty times the applied torque at failure. The vane tip was advanced eighteen inches into the undisturbed soil at which point the vane rods were clamped to a vertical bearing mounted on the top of the casing. This permits free rotational movement in the horizontal plane but prevents downward movement of the vane during the test. The torque was then applied and the value at failure recorded. The vane was then rotated six complete turns and a remoulded test was carried out in the same fashion. In order to determine the effect of rod friction in the vane tests, a number of tests were carried out using the same apparatus with the vane blades removed. Immediately after a friction test, a conventional vane test was carried out in order that a comparison might be made.

A number of piezometers were installed at five different locations numbered B.H.s 6P, 8P, 9P, 10P and 12P. At each location a Peaker Type piezometer connected to two 1/2" by 3/8" plastic tubes was installed at the bottom of the hole which was utilised for sampling purposes, and a bentonite

3. FIELD INVESTIGATION PROCEDURE: (cont'd) ...

seal was made some ten feet higher than the piezometer. The remainder of the hole was filled with sand. At each of B.H.s 6P and 10P four Geonor brass piezometers connected to single 3/8" by 1/4" plastic tubes were installed at various elevations between the sampling hole bottom and the ground surface. At each of B.H.s 8P, 9P and 12P five Geonor piezometers were installed in similar fashion. All Geonor piezometers were placed in separate borings which were drilled to within ten feet of the intended piezometer elevation, attached to 'A' rods or 'E' rods then driven the remaining ten feet by a hammer. The elevations of the various piezometers are shown on the pertinent log sheets contained in the Appendix of this report.

The locations and elevations of all boreholes were surveyed in the field by D.H.O. personnel from Engineering Surveys Section. Elevations are referred to Geodetic Bench Mark #563F located on Bridge #3. The Geodetic elevation on this land mark is 340.214 but the St. Lawrence Seaway Authority use a value 340.97, which has also been used during this project.

For the sake of simplicity all groups of boreholes are referred to as single boreholes numbered 1-13 incl. the centre of each group being given as the borehole location. The information from every hole in each group is therefore plotted on the single borelog sheet corresponding to a particular group. The borelog sheets are contained in the Appendix at this report.

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4. SOIL TYPES AND SOIL CONDITIONS:

4.1) GENERAL:

Subsoil at the site, apart from the fill material in the canal dykes, consists of about five different deposits or types of deposit overlying shale bedrock. Generally speaking, conditions are fairly uniform over the whole site as regards soil type though somewhat variable as regards the depths of the various strata. The boundaries of the different deposits as determined in the boreholes are shown on the accompanying borelog sheets and the estimated stratigraphical profile contained in Drawing #64-F-53A is based on this information. From ground level downward the various soil types are as follows:

4.2) FILL MATERIAL:

This material was observed in B.H.s #1, #8P, #9P and 12P which were drilled through the artificial dykes constructed at the edges of the Welland Canal. The soil consists of brown coloured silty clay with traces of sand and fine gravel and is generally well compacted with a consistency ranging randomly from stiff to very stiff. On the east side of the canal the depth was observed to be 22.5 feet whilst on the west side the observed depth was 29 feet. Based on the results of field and laboratory tests the physical properties of the material are summarised as follows:-

Bulk Density	.....	122 P.C.F.
Natural Moisture Content	...	14%-27%
Liquid Limit	.....	31%-53%

cont'd /7...

4.2) FILL MATERIAL: (cont'd)...

Plastic Limit ..... 16%-24%

Unconfined shear strength .. 1500-3900 P.S.F.

4.3) CLAYEY SILT WITH SAND AND OCCASIONAL GRAVEL:

(Upper Glacial Till)

This material extends from original ground level at M borehole locations for depths ranging from 25 to 39 feet and consists of a heterogeneous mixture of clayey silt, sand and gravel in the following average proportions:- clay 27%, silt 52%, sand 18%, gravel 3%. A wide range of 'N' values, 12 - 120 blows per foot, indicates some variation in consistency which is estimated to range from hard in the upper portion of the deposit to stiff in the lower portions. The texture of the material clearly shows the deposit to be of glacial origin with the predominant constituent being a clayey silt. A plot of plasticity index versus liquid limit shows the majority of the points confined to the CL portion of the chart. Natural moisture contents are generally close to the plastic limit. Generally speaking the lower ten feet of the deposit is somewhat softer than the rest and it was possible to obtain some shelby tube samples in this zone. Unconfined compression tests carried out on these samples gave undrained shear strength values ranging from about 800 to 1700 p.s.f. The worst location appears to be at B.H. #3 where the average was about 1000 p.s.f. Field vane tests on the other hand, were almost always greater than 2000 p.s.f. in this zone. No strength tests were carried out on material from the upper portion of the

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4.3) CLAYEY SILT WITH SAND AND OCCASIONAL GRAVEL: (cont'd)...  
(Upper Glacial Till)

deposit because of the hardness of the layer. Consolidation tests were carried out on samples obtained from B.H. #2 and #4 and indicated an overconsolidation pressure at about 1.5 tons in excess of the present overburden pressure. The effective overburden pressures quoted in this report have been computed from existing original ground level for convenience. It is believed that the overconsolidation pressure of the upper 20 feet of this layer is quite high. No attempt was made to estimate this during the present investigation.

Typical stress strain curves from unconfined compression tests, typical grain size distribution curves and a plot of plasticity index versus liquid limit are contained in the Appendix of this report on Figures 5, 3 and 1 respectively.

Physical properties of the material in the deposit are summarised as follows:-

Bulk Density	.....	130-140 p.c.f.
Natural Moisture Content	.....	12%-23%
Liquid Limit	.....	17%-37%
Plastic Limit	.....	11%-23%
Unconfined shear strength (bottom 10')	...	800-1700 p.s.f.

4.4) SILTY CLAY: (BROWN)

This deposit underlies the clayey silt stratum and extends for depths ranging from  $3\frac{1}{2}$  feet in B.H. #6P to 15 feet in B.H. #9P. It was observed in all boreholes and could be identified particularly by its brown to grey-brown colour and its

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4.4) SILTY CLAY (BROWN): (cont'd)...

generally high plasticity. The material contains numerous tiny pockets of red and grey silt and clay. These pockets were observed to be in a much dryer state than the parent material and were uniformly dispersed throughout the whole layer. Some horizontal layering in the form of faint red bands and colour boundaries was observed indicating evidence of stratification. A plot of plasticity index versus liquid limit on Figure 1 shows the points to be concentrated about the CI-CH boundary on the chart. The latter is contained in the Appendix of the report. The liquidity index averages about 0.5 for the whole deposit. A number of unconfined and unconsolidated-undrained triaxial tests were carried out on samples from this layer and gave values of undrained shear strength ranging from about 400 to 1400, p.s.f. These values were in general lower than the field vane tests carried out at the corresponding sample elevations. Results of these tests are discussed in greater detail in the section headed 'Discussion'. A number of typical stress strain curves obtained from the compression tests are shown on Fig. 6. The overall consistency is estimated to range from firm to stiff.

Consolidation tests which were carried out on samples recovered from B.H. #1 indicate the stratum to be overconsolidated by about 3000 p.s.f. in excess of the existing overburden pressure. These tests are plotted on Fig. 4 in the Appendix of the report.

Physical properties of the material in the deposit as determined from laboratory and field tests are summarised as follows:-

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4.4) SILTY CLAY (BROWN): (cont'd)...

Bulk Density	.....	106-122 p.c.f.
Natural Moisture Content	.....	28%-52%
Liquid Limit	.....	40%-55%
Plastic Limit	.....	21%-27%
Unconfined Shear Strength	.....	400-1400 p.s.f.
Undrained Triaxial Shear Strength	....	600-1100 p.s.f.
Field Vane Shear Strength	.....	500-1800 p.s.f.

Typical grain size distribution curves are shown in Fig. 4 of the Appendix.

4.5) SILTY CLAY WITH LAYERS OF CLAYEY SILT AND SILT:

(GREY)

This deposit underlies the brown silty clay stratum and extends for depths ranging from 7 feet in B.H. #3 to 14 feet in B.H. #1. It was observed in all boreholes and consisted of grey silty clay containing layers of varying thickness up to about 3 inches of red and grey clayey silt and grey silt. The layers were sensibly parallel and in the approximate horizontal plane. A plot of plasticity index versus liquid limit of the material from the cohesive layers shows a wide spread of the points along the 'A' line between the CI-CH and ML-CL boundaries. This plot is shown on Fig. 1 of the Appendix. The liquidity index ranges from about 0.5 to 1.0 the higher values generally representing the material of lower plasticity.

A number of unconfined and unconsolidated undrained triaxial tests were carried out on samples from this layer which showed

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4.5) SILTY CLAY WITH LAYERS OF CLAYEY SILT AND SILT

(GPEY): ( cont'd ) ...

an extremely wide scatter of results for the undrained shear strength. The range was approximately 300-1100 p.s.f. for both types of test the average value being 700 p.s.f. for each type of test. Typical stress strain curves for these tests are shown in Fig. 7 of the Appendix. Field vane tests were in general much higher than the laboratory tests. These results are discussed further in the section headed 'Discussion'. The overall consistency of the deposit is estimated to range from firm to stiff.

Consolidation tests carried out on samples from B.H. #2 indicated an overconsolidation pressure of about 1.5 t.s.f. throughout the stratum in excess of the existing overburden pressure. Results of these tests are plotted on Fig. 11 contained in the Appendix.

Physical properties of the material in the deposit as determined from laboratory and field tests are summarised as follows:-

Bulk Density .....	113-124 p.c.f.
Natural Moisture Content .....	20%-43%
Liquid Limit .....	22%-45%
Plastic Limit .....	14%-28
Unconfined Shear Strength .....	300-1100 p.s.f.
Undrained Triaxial Shear Strength .....	400-1000 p.s.f.
Field Vane Shear Strength .....	650-1800 p.s.f.

Typical grain size distribution curves are shown on Fig. 4 of the Appendix.

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4.6) CLAYEY SILT, SILT AND SAND (LAYERED):

This material underlies the grey silty clay at all borehole locations and extends for depths ranging from 10 feet in B.H. #5 to 30 feet in B.H. #11. The overall deposit consists of a number of layers of varying thickness of material ranging in grain size from gravel and sand to clayey silt, with the predominant constituent being silt. Results of mechanical analysis are summarised in the accompanying borehole sheet. Definite boundaries between layers which were observed during sampling operations are shown on the borelog sheets contained in the Appendix. The 'N' values obtained from Standard Penetration tests ranged from 22 to more than 100 blows per foot. The relative density of the stratum is estimated to range from dense at the surface to very dense at the lower boundary. The permeability is estimated to range from high in the granular coarse grained layers to relatively low in the fine grained cohesive layers.

Physical properties of the material in the deposit as determined from field and laboratory tests are summarised as follows:

Bulk Density	.....	111-130 p.c.f.
Natural Moisture Content	.....	7%-46%
Liquid Limit (Cohesive layers)..		18%-55%
Plastic Limit (Cohesive layers).		12%-28%
Relative Density ('N' Values) ..		22-100 blows /ft.

During boring operations it was observed that at a number of locations natural gas started to emerge from the boreholes immediately this layer was intersected. The quantity appears to fluctuate from time to time with no definite observable

4.6) CLAYEY SILT, SILT AND SAND (LAYERED): (cont'd)...

pattern. It is believed that the gas may be present both in solution of the pore water and as free bodies in the pore spaces. Further information on this phenomenon will be reported when the results of our study on piezometer water levels are completed. The boreholes at which gas was definitely observed are as follows:- B.M. #2, 4, 6, 8, 9, 10, 11, and 12.

4.7) CLAYEY SILT SAND AND GRAVEL (LOWER GLACIAL TILL):

This deposit is of glacial origin and consists of a red to brown coloured mixture of clayey silt, sand and occasional gravel. The material is in a very dense state. 'N' values being in the order of 60 > 100 blows per foot. The observed depth of the deposit ranged from 12 feet in B.H. #5 to 21 feet in B.H. #2.

Physical properties are summarised as follows:-

Natural Moisture Content	8%-21%
Plastic Limit	14%-17%
Liquid Limit	18%-23%

Results of mechanical analyses are summarized in the accompanying borelog sheets.

4.8) BEDROCK - RED SHALE:

Bedrock was observed at depths of 10<sup>4</sup> feet 95 feet and 90 feet in B.H.s 2.5, and 11 respectively and consisted of a reddish coloured shale. The upper contact of the sound bedrock was somewhat difficult to determine due to the presence of weathered material which resembled the overlying till material very closely.

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4.9) GROUNDWATER CONDITIONS:

A detailed investigation of piezometric water levels is at present underway in the field. At the present time the information available is incomplete and inconclusive. An additional report will be prepared covering this aspect of the subsoil conditions when our field investigation is completed.

5. DISCUSSION:

5.1) GENERAL:

It is proposed to construct a tunnel to replace the existing bascule type bridge at the intersection of Carlton St. and the Welland Canal at St. Catharines, Ontario. The tunnel will be constructed by the 'open cut' method and since the overall design is greatly affected by the geometry of the temporary slopes during construction and the permanent slopes for the tunnel approaches, the shear strength of the cohesive layers is of paramount importance. This aspect, together with the compressibility of the cohesive layers which must be taken into account in estimating the pressures exerted on the tunnel sections is discussed in some detail below:-

5.2) SHEAR STRENGTH OF THE CLAY LAYERS:

The stability of slopes immediately following construction is governed by the undrained shear strength and in the present case the most critical material is contained within the layers referred to as the brown silty clay and the grey silty clay. A wide range of scatter of undrained shear strength values has been obtained even for the same type of

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5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

strength test and a fairly wide divergence occurs between field and laboratory tests. At this point it will be convenient to refer to the summary of the results of all undrained shear strength tests carried out in the two strata referred to as the brown clay and the grey clay.

Table I shows the average undrained shear strength computed numerically from a total of 145 field vane tests, 100 unconfined compression tests and 26 triaxial compression tests for each of B.H.s 1-12 together with the minimum values. The grand averages for the whole site are also given. In the table the brown clay and the grey clay layers are considered separately. In every case the field vanes averages are the higher. It can be observed however, that much closer agreement exists between the field vane averages and the compression test averages for the brown clay than for the grey clay. Since the grey clay consists of a number of stratified layers of silty clay, clayey silt and silt it is believed that the effects of disturbance caused by sampling in the field and by handling thereafter would be much greater than in the case of the brown clay which is a much more homogeneous material. With regard to the field vane tests it is believed that the presence of the silt layers in the grey clay deposit would tend to increase the measured value of the shear strength. In addition, a number of vane rod friction tests carried out during the course of the field work indicated that between the shear strength range 1000 - 1500 p.s.f. rod friction accounted for about five to 10 percent of the measured torque

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B.H.	FIELD VANE AVERAGE MINIMUM		UNCONFINED AVERAGE MINIMUM		TRIAXIAL AVERAGE MINIMUM		FIELD VANE AVERAGE MINIMUM		UNCONFINED AVERAGE MINIMUM		TRIAXIAL AVERAGE MINIMUM	
1	1100	480	906	765	610	610	1147	640	722	535	834	546
2	1054	720	856	729	972	842	1108	760	491	287	600	525
3	1509	1120	775	575			1422	1040	834	635		
4	1091	960	1083	961	865	804	1065	640	765	655		
5	1280	1280	972	844			1413	1280	685	525	413	413
6			764	764			880	720	877	813		
7	1240	1120	1175	959			1108	960	874	741	732	690
8	1040	800	830	694			1013	880	678	455	742	564
9	970	720	849	663			1350	960	601	601		
10	1520	1520	387	387			980	800	534	370		
11	1040	960	1114	1010			1040	960	536	536	713	713
12	1360	1200	801	490			1350	1000	871	836	620	582
1-12	*1155/65 tests		*915/40 tests		*896/9 tests		*1160/80 tests		*707/60 tests		*701/17 tests	
1-4	1152/43 tests		887/22 tests		896/9 tests		1154/57 tests		687/38 tests		733/7 tests	

\* Averages for all boreholes.

TABLE I - UNDRAINED SHEAR STRENGTH P.S.F.

5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

at failure. When all of the foregoing is taken into consideration it can be deduced that, whereas in the case of the brown clay the compression test results are probably fairly representative of the actual undrained shear strength, in the case of the grey clay they are somewhat lower than the true value.

For design purposes it is recommended that a value of undrained shear strength for the brown clay of 900 p.s.f. should be used.

For the grey clay, it is known that the laboratory strengths are too low due to greater sample distance. However, the anisotropic nature of the clay has not been determined. Previous experience has shown that for a layered material, the normal testing method employed, with the direction of the major principal stress at Failure perpendicular to the bedding planes of the soil, yields the maximum strength with respect to different orientation of the major principal stress. These two factors have a compensating effect and it is therefore suggested that the undrained strength used in design should be 700 p.s.f. which is the average of the laboratory tests. The actual strength can only be determined when the test section is carried out.

In order to examine the long term stability of approach cuts it is necessary to determine the effective shear strength parameters of the till and clay strata. It is also necessary to estimate the pore water conditions both during and after excavation of the cuts. In order to determine the parameters

5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

of the cohesive strata a series of undrained triaxial compression tests with pore pressure measurements and drained tests were carried out. The results of these tests are plotted on Figs. 8-10 of the Appendix of the report. Two multi-stage undrained tests with pore pressure measurements carried out on samples from the till stratum gave values of  $\phi'$  of 26° and 31° with the values of effective cohesion  $C'$  being 110 p.s.f. and zero respectively. Drained tests carried out on samples from the brown clay stratum gave a value for  $\phi'$  of 18° and a cohesion intercept of 300 p.s.f. Further tests are now underway and the results will be presented as an addendum to this report.

5.3) REMOULDED STRENGTH OF THE CLAY LAYERS:

A number of tests were carried out to try to ascertain the remoulded shear strength of the clay layers and the effect of time on this factor.

In the field, remoulded vane tests were carried out immediately after every natural vane test and in a few instances, additional tests at varying times after the natural test were also carried out. The results of the field tests indicated an immediate average remoulded value of about 25% of the undisturbed value and showed large regains in strength within a few hours.

In the laboratory unconfined compression tests were carried out on a number of samples to determine both the undisturbed undrained shear strength and the remoulded shear strength of varying times up to 8 days after remoulding.



5.3) REMOULDED STRENGTH OF THE CLAY LAYERS: (cont'd)..

The tests indicated that little or no increase in strength occurs beyond the value of the immediate remoulded strength and that this value is about 15% of the undisturbed value.

It is believed therefore, that the regain of the field vane test can be largely attributed to consolidation effects.

5.4) COMPRESSIBILITY OF THE COHESIVE LAYERS:

To study the compressibility and swelling characteristics of the cohesive deposits and thus to determine the possible movements of the tunnel units during construction and subsequent performance a number of consolidation tests were carried out. These are summarised in the Appendix of this report. The tests show that in general the clay layers and the lower 10 feet of the upper till layers are overconsolidated by about 1.5 t.s.f. in excess of overburden pressure except at the dyke locations where the overburden pressure has been artificially increased by about 2000 p.s.f. due to the weight of the fill.

From the general shape of the pressure-void ratio curves it is apparant that the samples indicate some degree of disturbance. It is therefore not possible to compute exact values for the compression index  $C_c$ . For computation purposes it is considered best to use the actual plot of void ratio versus the logarithm of pressure as shown on the figures.

To simulate the possible movements due to swelling and rebound the consolidation tests were subjected to various load cycles. In general, the cycles were commenced from overburden

5.4) COMPRESSIBILITY OF THE COHESIVE LAYERS: (cont'd)..

pressure as computed from the borehole results. The pressure was then reduced to 0.25 t.s.f. and the sample reloaded to a pressure in excess of overburden pressure. The resulting rebound curve was generally quite uniform in shape both for the lower section of the upper till layers and the clay layers for which the average values of the recompression indices  $C_{cr}$  are estimated to be 0.01 and 0.06 respectively. These values can be used to estimate movement due to swelling and rebound as the values of the swelling indices  $C_s$  and the values for  $C_{cr}$  will be essentially similar.

6. SUMMARY:

The results of a foundation investigation to determine the subsoil conditions existing at the site of the proposed Carlton St. Tunnel in St. Catharines, Ontario, are reported. It was found that the site is underlain by deposits of hard to stiff glacial till, firm to stiff clays, stratified layers of clayey silt, silt and sand, and a further layer of glacial till overlying shale bedrock. Depth to bedrock ranged from 90 to 104 feet.

The most critical material is contained within the clay layers and a detailed programme of field and laboratory testing has been carried out to determine values of undrained shear strength to be used for design purposes. This and other aspects are discussed in some detail in the main body of the report.

A study of piezometric water levels in the various strata is at present underway in the field. The results will be reported as soon as the study is complete.

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7. MISCELLANEOUS:

The boring programme was commenced on July and is still underway at the time of writing this report. Equipment being used on the site is owned and operated by Dominion Soil Investigations Ltd. The fieldwork was supervised directly by Project Fdn. Engineers Mr. P. Payer and Mr. P. McGlone. The preparation of this report together with the general supervision of the fieldwork was carried out by Mr. K. Selby, Senior Fdn. Engineer. The report was reviewed by Mr. K. Y. Lo, Supervising Foundation Engineer.

October 9, 1964.

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - 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>'N' BLOWS / FT.</u>	<u>c LB. / SQ FT</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W.	THINWALL OPEN
W.S	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

# ABBREVIATIONS 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
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
$S_i$	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
$K_o$	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

APPENDIX I.

DEPARTMENT OF HIGHWAYS - CHAIRMAN  
MATERIALS & RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 1

## FOUNDATION SECTION

JOE 64-P-53 LOCATION Sta. 37463 145' Rt. OR ORIGINATED BY K.S.  
W.P. 444-6-4 & 5 BORING DATE July 6, 1964. COMPILED BY H.S.  
DATHIN G.S.C. BOREHOLE TYPE Augured to 50'-hashboring NEX Casing to 95' CHECKED BY P.Mc.

SOIL PROFILE			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— PL PLASTIC LIMIT ——— P WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLCH	SAMPLES NUMBER	TYPE	THICKNESS / FOOT	ELEV. SCALE	500 1000 1500 2000 2500	20 40 60				
338.9	Groundlevel											
0.0												
			1	SS	16							
						330						
	Silty clay with traces of sand and fine gravel.		2	SS	20							
	Very stiff to stiff											
	Brown Coloured.		3	SS	16							
						320						
	(Fill Material)		4	SS	21							
316.4												
22.5			5	SS	79							
	Clayey Silt with Sand and occasional gravel.					110						
	Hard to stiff.		6	SS	68							
	Brown Coloured.											
	(Glacial Till)		7	SS	30							
						300						
			8	SS	22							
			1B	TW	PM							
			2B	TW	PM							
			3B	TW	PM & driven							
288.7			4B	TW	PM	290						
50.0			10	SS	11							
			11	TW	PM							
	Silty clay.		12	TW	PM							
			13	TW	PM							
	Firm to stiff.		14	TW	PM							
			15	TW	PM	280						
	Brown Coloured.		16	TW	PM							
277.9			17	TW	PM							
61.0			18	TW	PM							
	Silty clay with layers of clayey		19	TW	PM							
	silt and silt.		20	TW	PM							
	Firm to stiff		21	TW	PM							
	Grey Coloured.		22	TW	PM	270						
			23	TW	PM							
			24	TW	PM							
			25	TW	PM							
263.9			26	TW	PM							
75.0	Silt with traces of fine sand and gravel.		27	TW	PM							
261.4	Y. dense-grey.		28	SS	54							
77.5	Fine to medium sand with occasional gravel.					260						
257.9	Y. dense-grey.		1A	SS	40							
79.0												
	Sandy silt to silty sand with occasional gravel and traces of clay.		2A	SS	88 for 11"	250						
	Very dense											
	Grey Coloured.		3A	SS	95							
242.4			4A	SS	172							
96.5	End of borehole.					240						

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH DIVISION		RECORD OF BOREHOLE NO. 2		FOUNDATION SECTION	
JOB <u>64-P-53</u>		LOCATION <u>Sta. 39+75.41' Rt.</u>		ORIGINATED BY <u>K.S.</u>	
W.P. <u>444-66 &amp; 5</u>		BORING DATE <u>July 7, 1964</u>		COMPILED BY <u>H.S.</u>	
DATUM <u>G.S.C.</u>		BOREHOLE <u>Augured to 25' - Washboring NX Casing to 103.5'</u>		CHECKED BY <u>P.H.</u>	

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLCT	SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS - FOOT					Liquidity Limit PLASTIC LIMIT - W.P. WATER CONTENT %			BULK DENSITY P.C.F.	REMARKS
			NUMBER	TYPE		500	1000	1500	2000	2500	20	40	60		
318.6	Groundlevel														
	Clayey silt with sand and occasional gravel.		1	SS 29	310										
	Hard to stiff.		2	SS 35											
	Brown Coloured.		3	SS 25											
	(Glacial Till)				300										
			1A	TN PM											
			2A	TN PM											
			3A	TN PM											
			6	TN PM											
			3C	TN PM	290										
			4C	TN PM											
287.1			5C	TN PM											
31.5	Silty clay.		6C	TN PM											
	Firm to stiff.		7C	TN PM											
	Brown Coloured.		8C	TN PM											
			9C	TN PM	280										
278.6			10C	TN PM											
43.0	Silty clay with layers of clayey silt and silt.		11C	TN PM											
	Firm.		12C	TN PM											
	Grey Coloured.		13C	TN PM											
			14C	TN PM											
			15C	TN PM											
			16C	TN PM	270										
			17C	TN PM											
			18C	TN PM											
			19C	TN PM											
266.1			20C	TN PM											
52.5	Silt with traces of fine sand and gravel.		1B	SS 70											
263.1	V. dense-grey.														
55.5	Fine to medium sand with occasional gravel.		2B	SS 90	260										
260.1	V. dense-grey.														
58.5	Clayey silt with layers of silt and sand.		3B	SS 45											
	V. dense-grey.		4B	SS 50	250										
			5B	SS 163											
			6B	SS 50	240										
235.6			7B	SS >100											
83.0	Clayey silt, sand and gravel.				230										
	Hard														
	(Glacial Till)														
	(Red Coloured)														
			9BA	SS Not Recorded	220										
214.6															
104.0	Bedrock				210										
	Red Shale														
202.6															
115.0	End of borehole.														

Sa-20%  
 SI-51%  
 CI-29%

Gr-1%  
 Sa-1%  
 SI-1%  
 CI-1%

Gr-10%  
 Sa-59%  
 SI-24%  
 CI-4%



DEPARTMENT OF ENERGY - SECURITY  
MATERIALS & RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOE 64-F-53

LOCATION Sta. 34+51 127' Int.

ORIGINATED BY K.S.

W P 444-64 & 5

BOOKING DATE July 28, 1964.

COMPILED BY H.S.

DETUR G.S.C.

Washboring - NX Casing.

CHECKED BY           P. Mc          

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT - W.L. PLASTIC LIMIT - P.L.		WATER CONTENT - %		SOIL DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLCT NUMBER	TYPE	BLOWS / FOOT	WE	WL	WP	WC		
335.5	Mean H.L.									
	Water									
306.0	Groundlevel									
	Clayey silt with sand and occasional gravel.	1B	SS 23							
	Hard to stiff.	2B	TW 33/6"	300						
	(Glacial Till).	3B	TW 24/6"							
	Brown Coloured.	4B	TW 14/6"							
		5B	TW 20/6"							
		6B	TW PM							
		8	TW PM	290						
		9	TW PM							
		10	TW PM							
		11	TW PM							
		12	TW PM							
		13	TW PM							
		14	SS 22	280						
		15	TW PM							
276.0		16	TW PM							
30.0	Silty clay.	17	TW PM							
	Firm to stiff.	18	TW PM							
	Brown Coloured.	19	TW PM							
271.2		20	TW PM	270						
34.8	Silty clay with layers of clayey silt and silt.	21	TW PM							
	Firm to stiff.	22	TW PM							
		23	TW PM							
264.2	Grey Coloured.	24	TW PM							
41.8		25	TW PM							
	Silt with traces of fine sand and gravel.	26	TW PM							
	V. dense-grey.	27	SS 29	260						
		2A	SS 53							
255.0										
51.5	Fine to med. sand with occasional gravel.	3A	SS 163	250						
	V. dense-grey.									
250.0										
58.0	Clayey silt with layers of silt and sand.	4A	SS 10							
	Compact to dense.	5A	TW PM							
	Gray Coloured.	6A	TW 20							
		7A	SS 22	240						
236.7		8A	SS 39							
69.3	End of borehole.			230						



## FOUNDATION SECTION

ELEV DEPT	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER TYPE BLANK / FOOT	ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT PLASTIC LIMIT WATER CONTENT % W <sub>p</sub> W <sub>L</sub> W <sub>t</sub>	WATER CONTENT % 20 40 60	BULK DENSITY P R	REMARKS
316.7 0.0	Groundlevel							
		1 SS 43						
		2 SS 69	310					
	Clayey silt with sand and gravel.	3 TW 37						No recovery Used spoon.
		4 TW 37						Silt-42% Clay-27% Grav-19% Sand-12%
	Hard to stiff.	5 SS 40	300					
	(Glacial Till)	6 TW PH						
	Brown coloured.	7 TW PH						
		8 TW PA	290					No recovery Used spoon.
		9A SS 17						
		9 TW 34						
		10 SS 11						
		11 TW PH						
		12 TW 44						
		13 TW 39	280					
278.2		14 SS 19						
38.5	Silty clay. Firm to stiff.	15 TW PH						Silt-46% Sand-32% Clay-16% Grav-6%
274.2	Brown coloured.							
42.5	Silty clay with layers of clayey silt and silt. Firm to stiff.	16 TW PH						
		17 TW PH	270					
	Grey Coloured.	18 TW						
64.2								
52.5		19 SS 105						Silt & Clay-86% Sand-13% Grav-1%
	Clayey silt and silt Layered Very Dense Grey Coloured.	20 SS 113	260					
		21 SS 86	250					
		22 SS 86	240					
		23 SS 111	230					Silt-59% Clay-36% Sand-5%
333.7		24 SS 107						
83.0								
	Clayey silt. Sand and gravel.	25 SS 141	230					
		26 SS 111						
	Hard (Glacial Till)	for 10" 27 SS 150 for 1"						
221.7	Brown to Red Bedrock	28 RC -						
95'0"	Red Shale	29 KC -	220					
216.7	End of borehole.							





DEPARTMENT OF MINES, ONTARIO  
MATERIALS & RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 8P

FOUNDATION SECTION

JOB 64-F-53 LOCATION Sta. 37+92 E ORIGINATED BY K.S.  
 W.P. 444-64 & 5 BORING DATE August 25, 1964. COMPILED BY B.M.G.  
 DATUM E.S.C. BOREHOLE TYPE Washboring using NX & BX casing. CHECKED BY P.Mc.

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W <sub>c</sub>		BULK DENSITY P.C.F.	REMARKS							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER TYPE	BLOWS / FOOT	ELEV SCALE	SHEAR STRENGTH P.S.F. Oedometer Compr. • Triaxial Compr + Field Vane Test				WATER CONTENT % 20 40 60						
338.48	Groundlevel					500	1000	1500	2000	2500						
0.0																
	Silty clay with traces of sand and fine gravel.		1A	SS	14											
	Very stiff to stiff.		2A	SS	33	330										
	Brown Coloured		3A	SS	21											
	(Fill Material)		4A	SS	20	320										
315.5			5A	SS	115											
23.0	Clayey silt with sand and occasional gravel.		6A	SS	56	310										
	Hard to stiff.		7A	SS	27											
	(Glacial Till)		8A	TW	PM	300										
	Brown Coloured.		9A	TW												
			10A	TW	PM	290										
288.8			11A	TW	PM											
49.7	Silty clay.		12A	TW	PM											
	Firm to stiff.		13A	TW	PM	280										
	Brown Coloured.		14A	TW	PM											
274.9			15A	TW	PM											
63.6	Silty clay with layers of clayey silt and silt.		16A	TW	PM	270										
	Firm to stiff.		17A	TW	PM											
	Gray Coloured.		18A	SS	9											
264.7			19A	TW	PM											
75.5	Clayey silt, silt and sand.		20A	SS	30											
	- Layered -		21A	SS	40											
	Dense to V. dense		22A	SS	78	260										
	Gray Coloured.		23A	SS	116											
256.48			24A	SS	102											
82'-0"	End of borehole.															

Silt-57%  
Clay-32%  
Sand-11%  
Piez (F)  
Tip El 325.4

Piez (E)  
Tip El 316.5

Piez (D)  
Tip El 311.3

Silt-51%  
Clay-36%  
Sand-13%

Piez (C)  
Tip El 294.5

No recovery  
Used spoon

115

113

Piez (B)  
274.8  
Tip El.

Silt-48%  
Clay-39%  
Sand-7%  
Grav-6%

Silt & Clay-  
88%  
Sand-10%  
Grav-2%

Sand-61%  
Silt-35%  
Grav-4%  
Piez (A)  
Tip El 256.5

## DEPARTMENT OF HIGHWAYS, MATERIALS &amp; RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 9P

FOUNDATION SECTION

JOB 64-P-53 LOCATION Sta. 37+29 g ORIGINATED BY K.S.  
 W.P. 444-64 & 5 BORING DATE September 2, 1964. COMPILED BY B.M.G.  
 DATUM G.S.C. BOREHOLE TYPE Washboring using NK Casing. CHECKED BY P.Mc

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— W <sub>L</sub>		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE	BLWS./FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. + Field Vane Test	PLASTIC LIMIT ——— W <sub>P</sub> WATER CONTENT ——— W <sub>0</sub> ——— W <sub>L</sub>		
336.3	Waterlevel					O Unconf. Compr. Test 500 1000 1500 2000 2500	WATER CONTENT %		
	Water				330				
321.1	Groundlevel								
0.0	Silty Clay		1A SS 11		320				
	- Stiff -								
316.1	Fill Material		2A SS 55						Piez (E) Tip El. 314.6
5.0	Clayey silt with sand and occasional gravel.		3A SS 58		310				
	hard to stiff.								Piez (D) Tip El. 308.1
	Glacial Till		4A SS 51						
	Brown Coloured.		5A SS 29		300				
			6A SS 13						Gr-1% Sa-19% Si-52% Cl-28% Piez (C) Tip El. 290.1
290.3	Silty Clay.		7A TW PM		290				
30.8	Firm to Stiff.		8A TW PM						
	Brown Coloured.		9A TW PM		280				
275.1									
46.0	Silty clay with layers of clayey silt and silt.		10A TW PM		270				Piez (B) Tip El. 273.6
	Firm to stiff.		11A TW PM						
	Grey Coloured.		12A TW PM						Gr-3% Sa-17% Si-61 83%
264.1	Silt		13A SS 24						
261.1	Very dense		14A SS 38						
60.0	Sand and gravel.		15A SS 48		260				Gr-6% Sa-34% Si-61 60% Piez (A) Tip El. 258.1
257.6	Very dense		16A SS 148						
63.6	End of borehole.								

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 10P

FOUNDATION SECTION

JOB 64-F-53 LOCATION Sta. 39+00 E ORIGINATED BY K.S.  
 W.P. 444-64 & 5 BORING DATE August 19, 1964. COMPILED BY H.S.  
 DATUM G.S.C. BOREHOLE TYPE Washboring using NX and BX Casing. CHECKED BY P.Me.

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	PLASTIC LIMIT — WP		
							WATER CONTENT — W		
							WP — W — WL		
							WATER CONTENT %		
							20 40 60		
319.5	Groundlevel								
0.0									
			1A	SS	45				
	Clayey Silt with sand and occasional gravel.					310			Piez (E) Tip El 311.
			2A	SS	35				
	Hard to Stiff.								
	(Glacial Till)		3A	SS	29				Gr 2% Si 49% Sa 14.2% Piez (D) Tip El 305.0
	Brown Coloured)					300			
			4A	SS	18				
			5A	TW	PM				137.8
			6A	TW	PM				142
						290			
286.5			7A	TW	PM				142
33.0									
	Silty Clay.		8A	SS	13				Piez (C) Tip El 285.0
	Firm to Stiff.								
			9A	TW	PM				106
	Brown Coloured.					280			
277.3									
42.2	Silty clay with layers of clayey silt and silt.		10A	TW	PM				115
	Firm to Stiff								Piez (B) Tip El 275.0
	Grey Coloured.		11A	TW	PM				142
						270			
266.5			12A	TW	PM				124
53.0	Silt								
			13A	SS	60				
263.0	Very dense		14A	SS	57				
56.5	Sand and gravel								
261.0	Very dense		15A	SS	155				
58.5	Clayey silt & silt		16A	SS	113 for	260			Gr 2% Si 66% Sa 12% Cl 20%
258.5	Very dense		17A	SS	131 "B2"				Piez (A) Tip El. 258.2
61.0	End of borehole.								



DEPARTMENT OF HEALTH AND HUMAN SERVICES  
MATTHEW S. R. DEBBES, M.D., M.P.H., Director

## RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOE 64-P-53

LOCATION Sta. 22+14.78' Lt.

ORIGINATED BY E.S.

WFO 444-64 & 5

BOEING CO: August 13, 1964.

ORIGINAL BY A.S.  
CORRECTED BY B.M.D.

SYSTEM G.S.C.

### Washboring using NY and BY Casinos

COMPILED BY **B. M. S.**

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION

## RECORD OF BOREHOLE NO. 12P

FOUNDATION SECTION

JOB 64-F-53

LOCATION Sta. 29+10 66' Rt.

ORIGINATED BY K.S.

W.P. 444-64 & 5

BORING DATE September 10, 1964.

COMPILED BY B.M.G.

DATUM G.S.C.

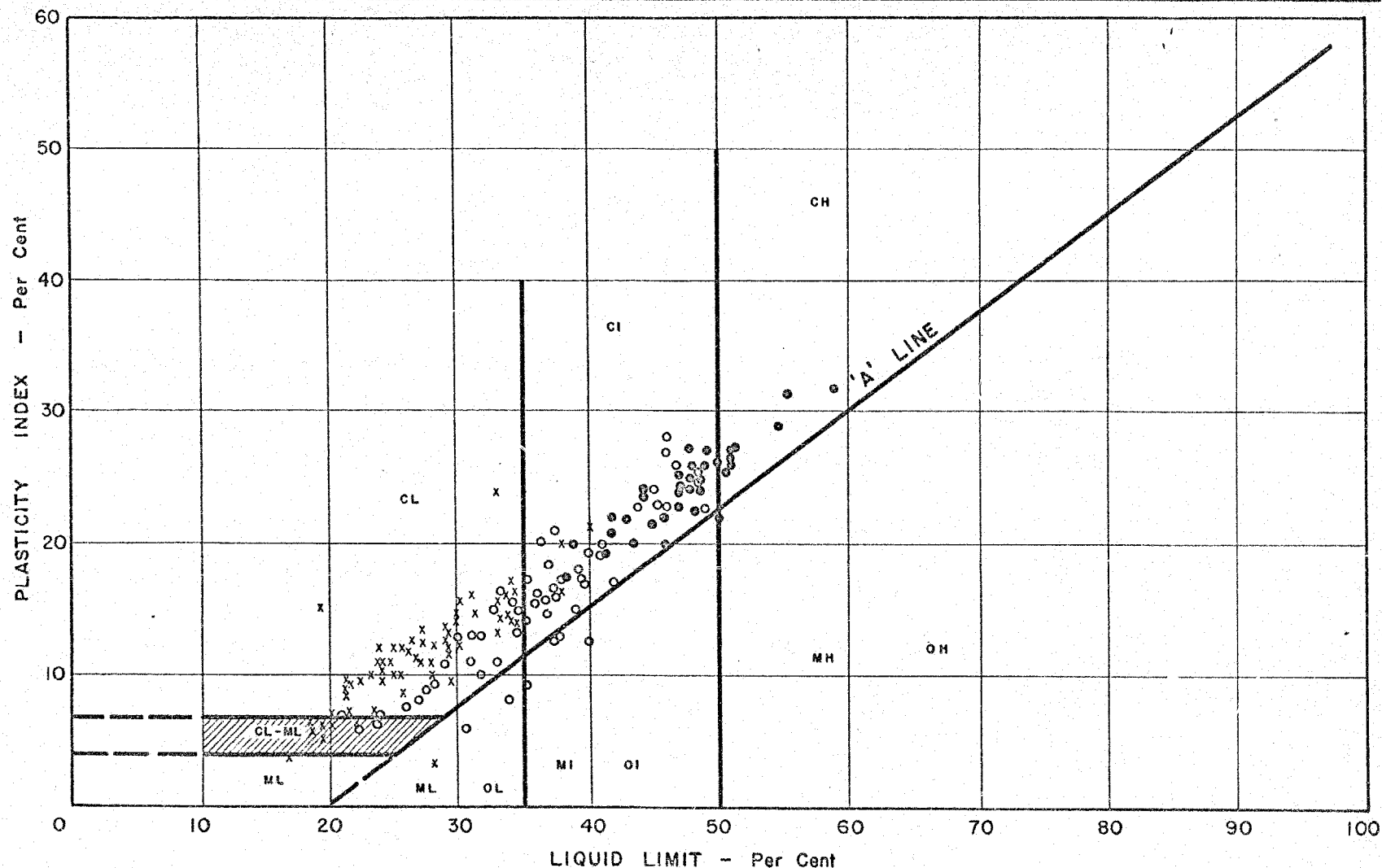
BOREHOLE TYPE: Washboring using NX Casing.

CHECKED BY P.Mc.

[illegible]

JOB 64-F-53 LOCATION Sta. 39400, 1180' Rt. ORIGINATED BY H.S.  
 W.P. 64-64 & 5 BORING DATE Sept. 21, 1964. COMPILED BY H.S.  
 DATUM G.S.C. BOREHOLE TYPE Neighboring - Neighboring & EX Casing. CHECKED BY P.H.

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ———— W <sub>L</sub> PLASTIC LIMIT ———— W <sub>P</sub> WATER CONTENT ———— W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	WCSN / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. C Unconf. Compr. Test + Field Vane Test 500 1000 1500 2000 2500		
322.6	Groundlevel								
0.0									
						320			
			1	SS	39			○ —	
			2	SS	29			○ —	
	Clayey silt with sand and occasional gravel.		3	SS	43			○ —	
						310			
			4	SS	35			○ —	
	Hard to stiff.		5	SS	22			○ —	
	Glacial Till		6	TW	PM				
						300			
	Brown Coloured.		7	TW	PM				
			8	TW	PM				
			9	TW	PM				
			10	SS	23			○ —	
			11	TW	PM				
285.8			12	TW	PM				
36.8									124
	Silty Clay.		13	TW	PM			○ —	
	Firm to stiff.		14	TW	PM				
	Brown Coloured.		15	TW	PM			○ —	118
			16	TW	PM			○ —	108
473.1									
49.5	Silty Clay with layers of clayey silt and silt.		17	TW	PM			○ —	116
	Firm to stiff.		18	TW	PM			○ —	119
265.6	Grey Coloured.								
57.0	Clayey silt to sandy silt.		19	TW	PM			○ —	
	- Layered -		20	SS	35				
	Dense to v. dense.		21	SS	52				
			22	SS	32				
			23	SS	42				
255.9	Grey Coloured.		24	SS	60for5"				
66.7	Sand		25	SS	70for7½"				
252.6	Very dense.		26	SS	56for4"				
18.0						250			
	Clayey silt to sandy silt with occasional gravels.		27	SS	70for6"			○ —	
	- Layered -		28	SS	85for6"				
						240			
	Very dense.		29	SS	40for3"				
	Grey Coloured.								
			30	SS	100for3"				
						230			
			31	SS	100for5"				
			32	SS	105				
221.1						220			
101.5	End of borehole.								



NOTES

- X GLACIAL TILL
- FIRM BROWN CLAY
- FIRM GREY CLAY
- (BORE HOLES 1 - 11)

FIG. 1

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION  
PLASTICITY CHART

Job No. 64-F-53 W.P. No. 444-64  
Location CARLTON ST. - ST. CATHARINES

# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt

Sand

Gravel

Fine

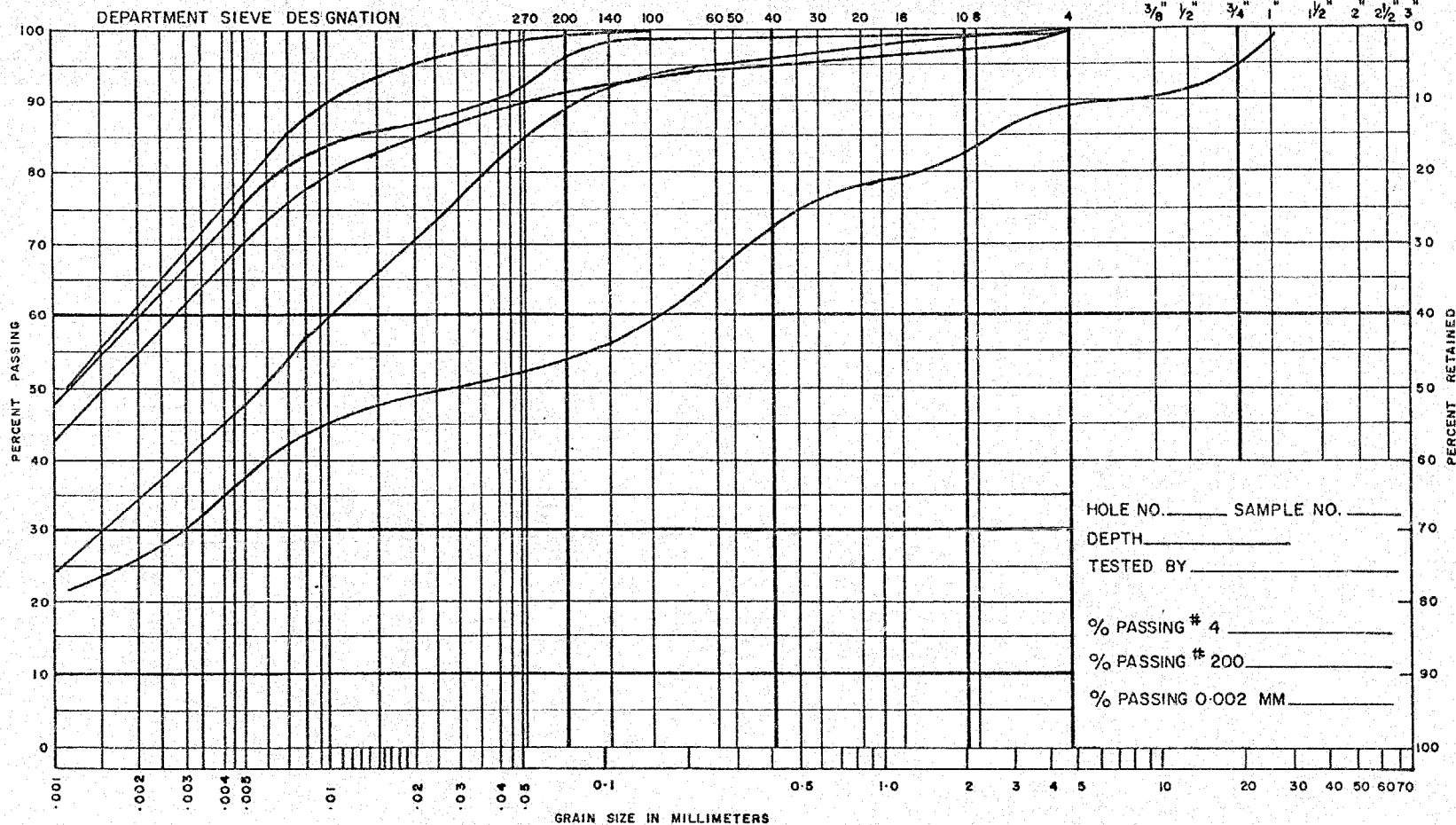
Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION



HOLE NO. \_\_\_\_\_ SAMPLE NO. \_\_\_\_\_

DEPTH \_\_\_\_\_

TESTED BY \_\_\_\_\_

% PASSING # 4 \_\_\_\_\_

% PASSING # 200 \_\_\_\_\_

% PASSING 0.002 MM \_\_\_\_\_

NOTES TYPICAL CURVES

FILL MAT'L. FROM BORE HOLES NO. 1, 8P & 12 P

DEPARTMENT OF HIGHWAYS — ONTARIO

MATERIALS & TESTING DIVISION

GRAIN SIZE DISTRIBUTION

JOB NO. 64-F-53

W. P. NO. 444-64

LOCATION CARLTON STREET

ST. CATHARINES

FIG. 2

# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt

Sand

Gravel

Fine

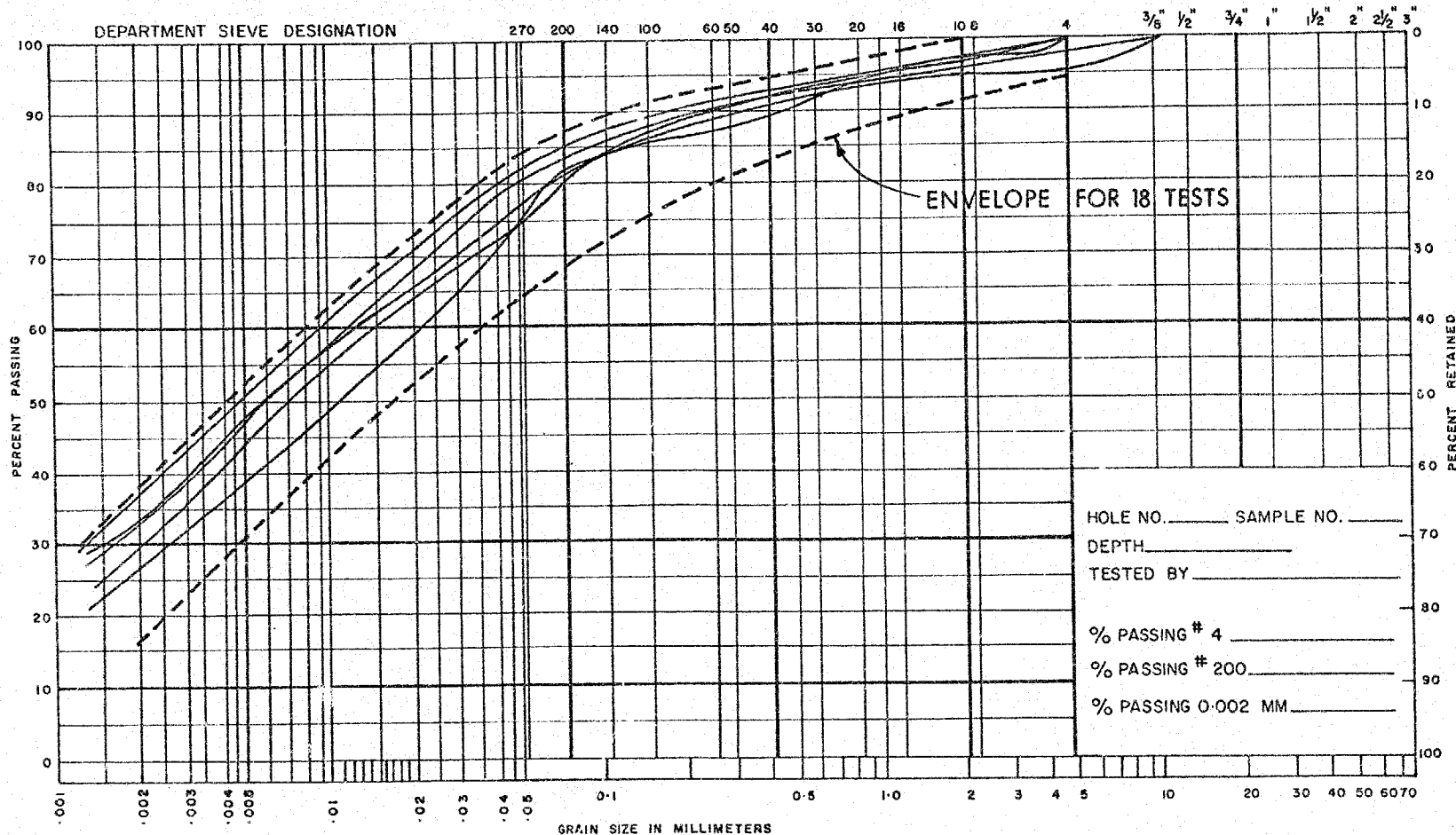
Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION



NOTES TYPICAL CURVES

TILL FROM BORE HOLES NO. 1, 2, 3, 4 & 7

DEPARTMENT OF HIGHWAYS — ONTARIO  
MATERIALS & TESTING DIVISION

GRAIN SIZE DISTRIBUTION

JOB NO. 64 - F - 53 W.P. NO. 444 - 64

LOCATION CARLTON STREET ST. CATHARINES

FIG. 3

## UNIFIED SOIL CLASSIFICATION SYSTEM

Clay &amp; Silt

Sand

Gravel

Fine

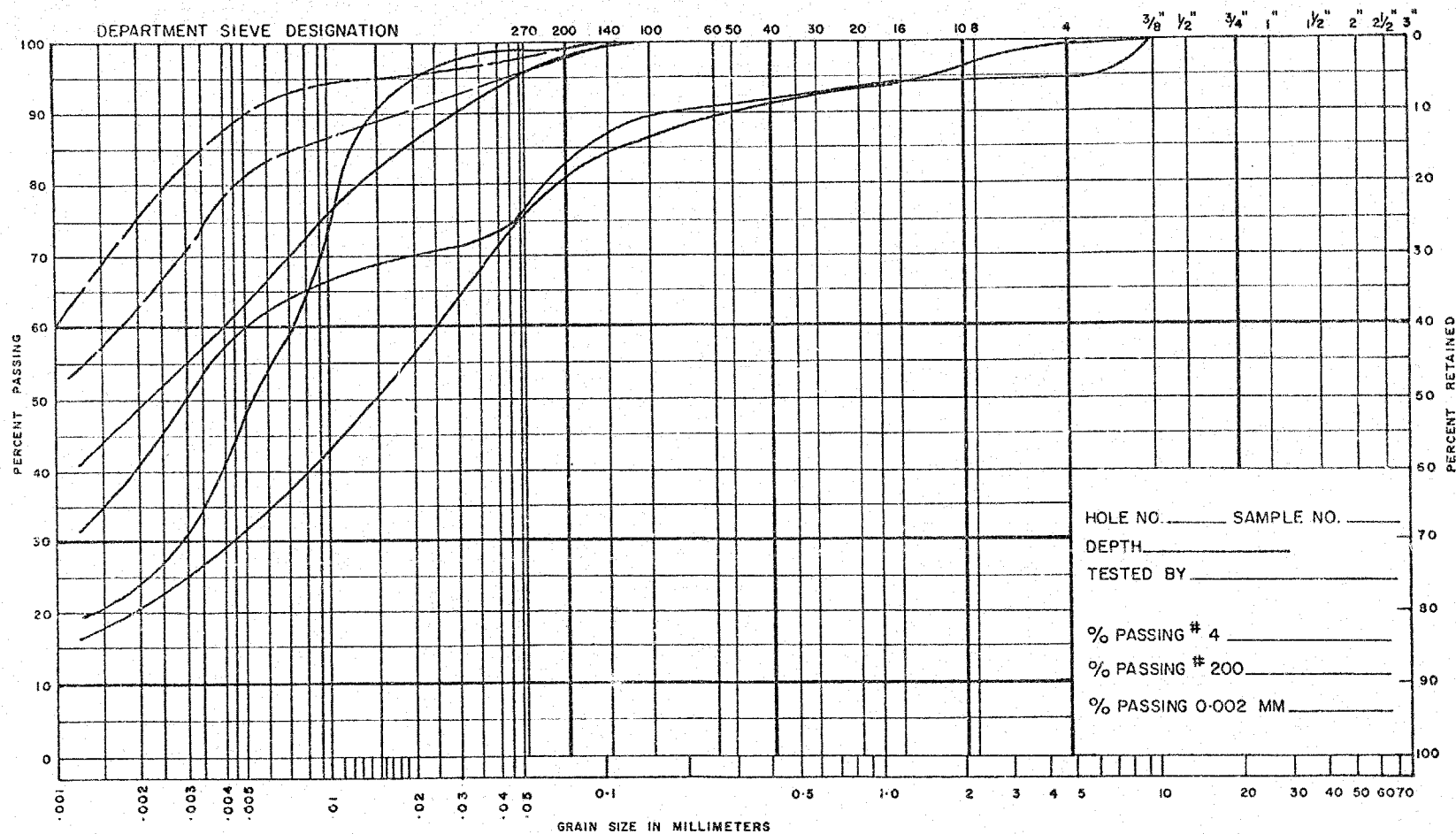
Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION



## NOTES TYPICAL CURVES

- \_\_\_\_ GREY CLAY FROM BORE HOLES NO. 2C & 8P  
\_\_\_\_ BROWN CLAY FROM BORE HOLES NO. 1 & 4

DEPARTMENT OF HIGHWAYS — ONTARIO  
MATERIALS & TESTING DIVISION

## GRAIN SIZE DISTRIBUTION

JOB NO. 64 - F - 53 W.P. NO. 444 - 64  
LOCATION CARLTON STREET ST. CATHARINES

FIG. 4

# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt

Sand

Gravel

Fine

Medium

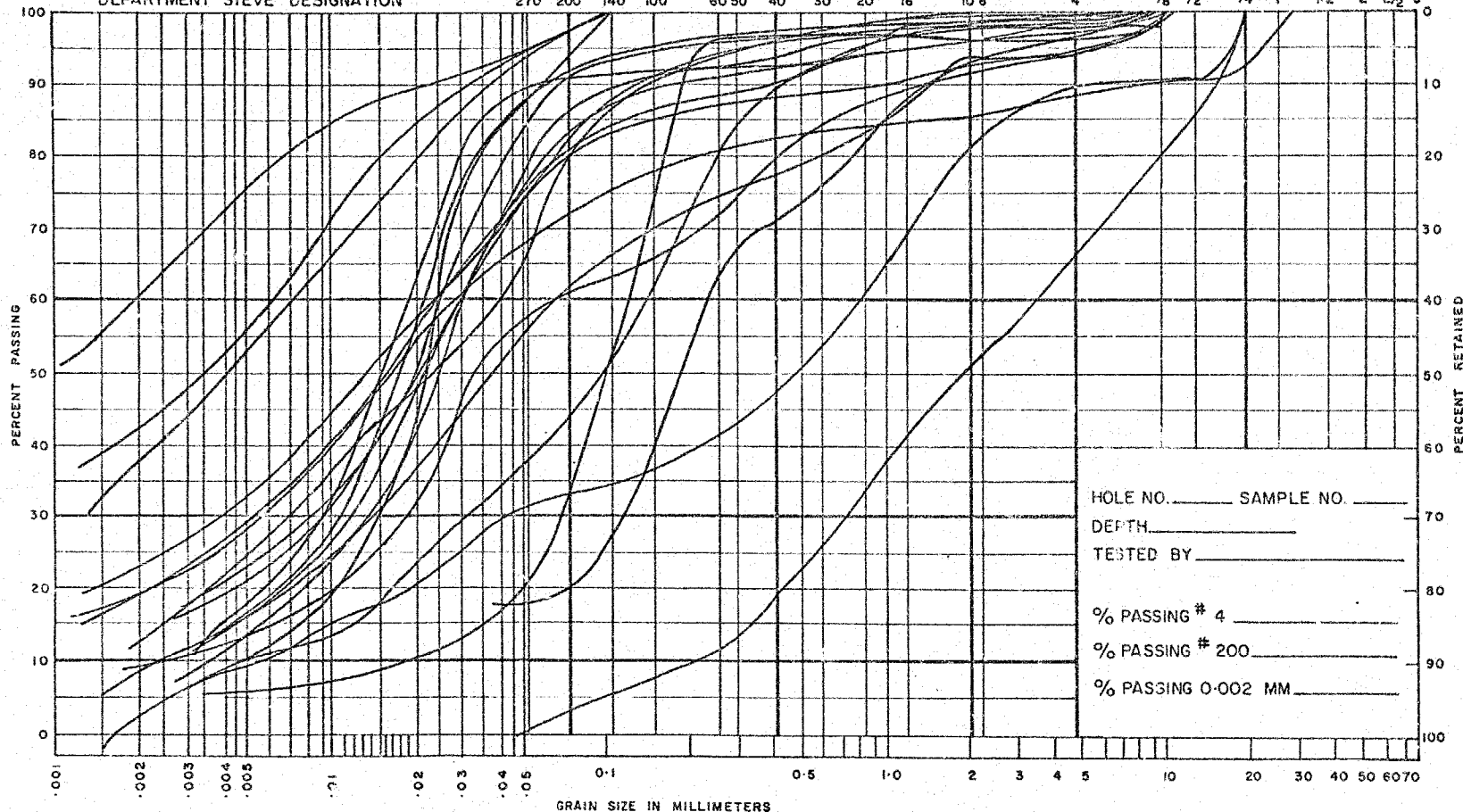
Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270 200 140 100 60 50 40 30 20 16 10 8 4 3/8" 1/2" 3/4" 1" 1 1/2" 2" 2 1/2" 3"



TYPICAL CURVES

NOTES CLAYEY SILT, SILT & SAND DEPOSIT  
FROM ALL BOREHOLES

DEPARTMENT OF HIGHWAYS — ONTARIO  
MATERIALS & TESTING DIVISION

GRAIN SIZE DISTRIBUTION

JOB NO. 64-F-53 W.P. NO. 144-64 & 65  
LOCATION CARLTON ST. ST. CATHARINES



# UNCONFINED COMPRESSION TEST, TYPICAL STRESS STRAIN CURVES

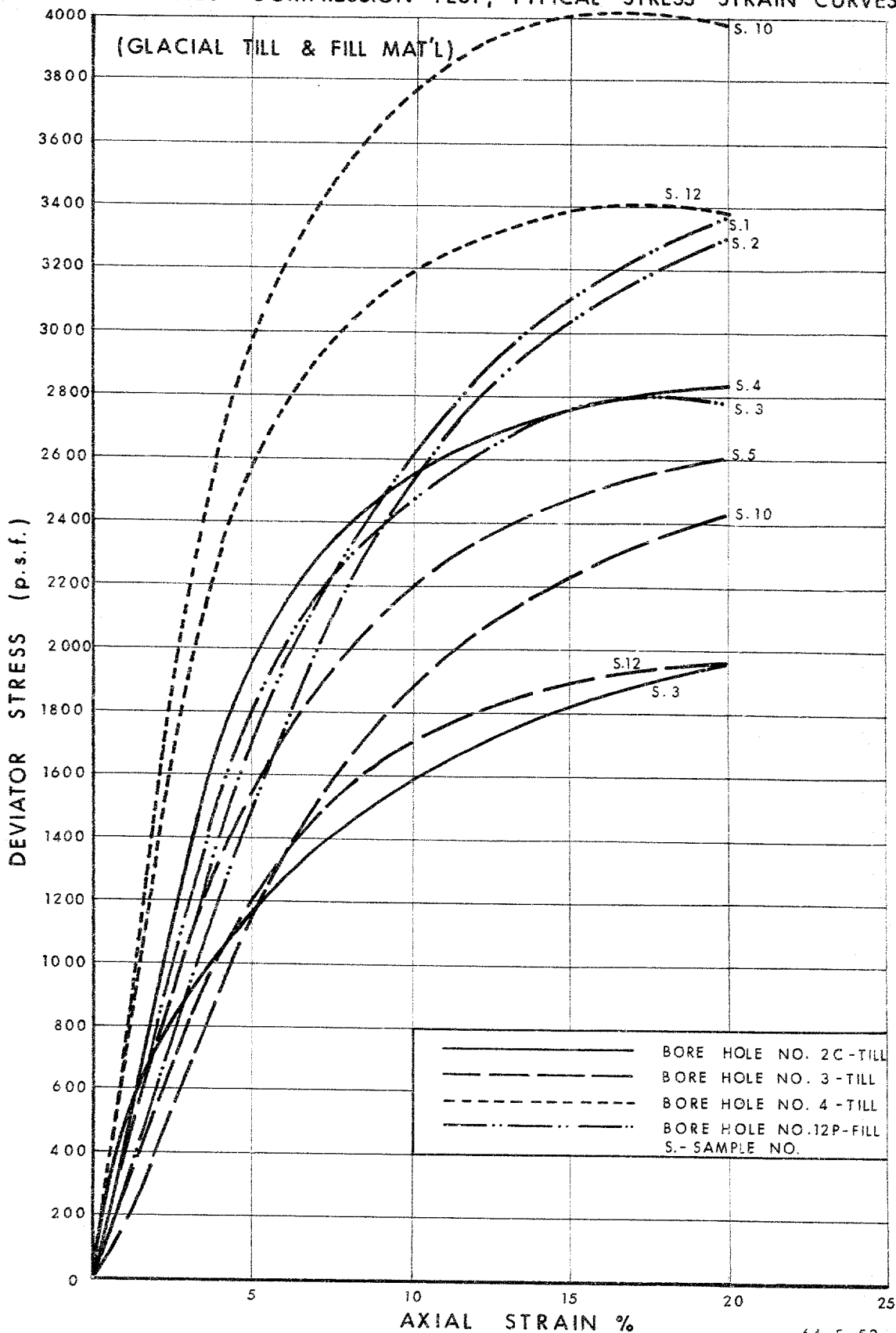
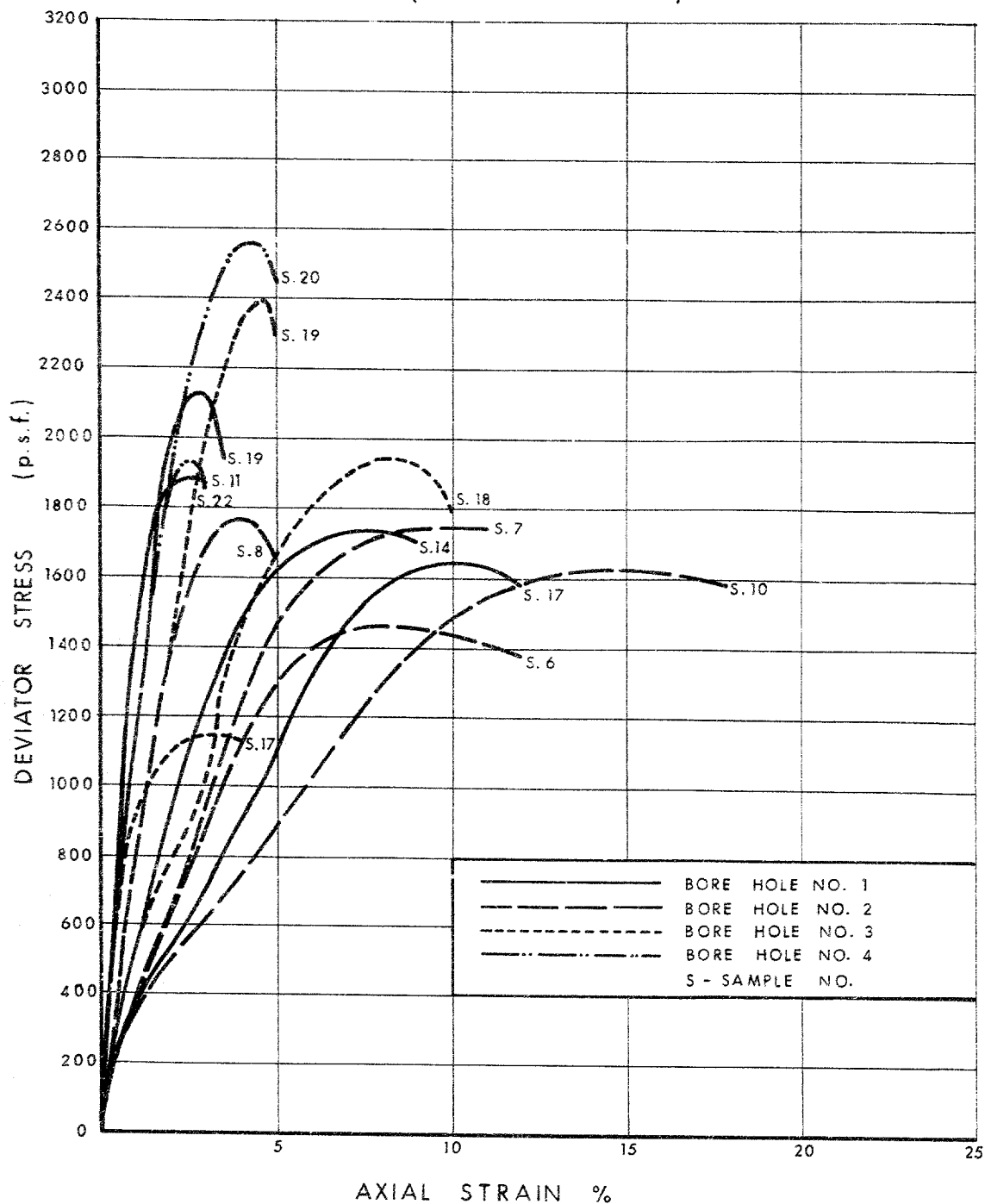


FIG. 5

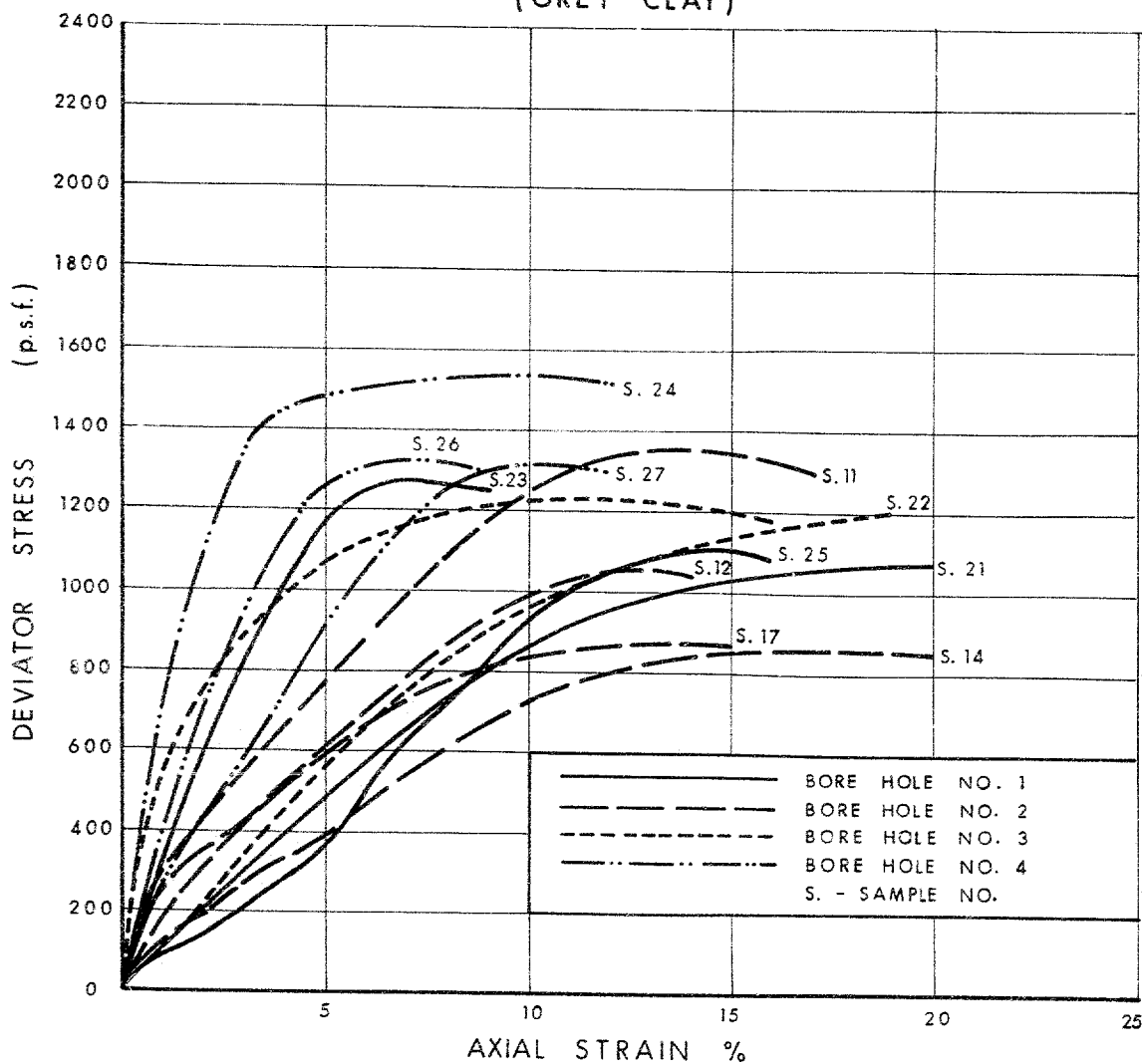
# UNCONFINED COMPRESSION TEST, TYPICAL STRESS STRAIN CURVES (FIRM BROWN CLAY)



JOB NO. 64 - F - 53

FIG. 6

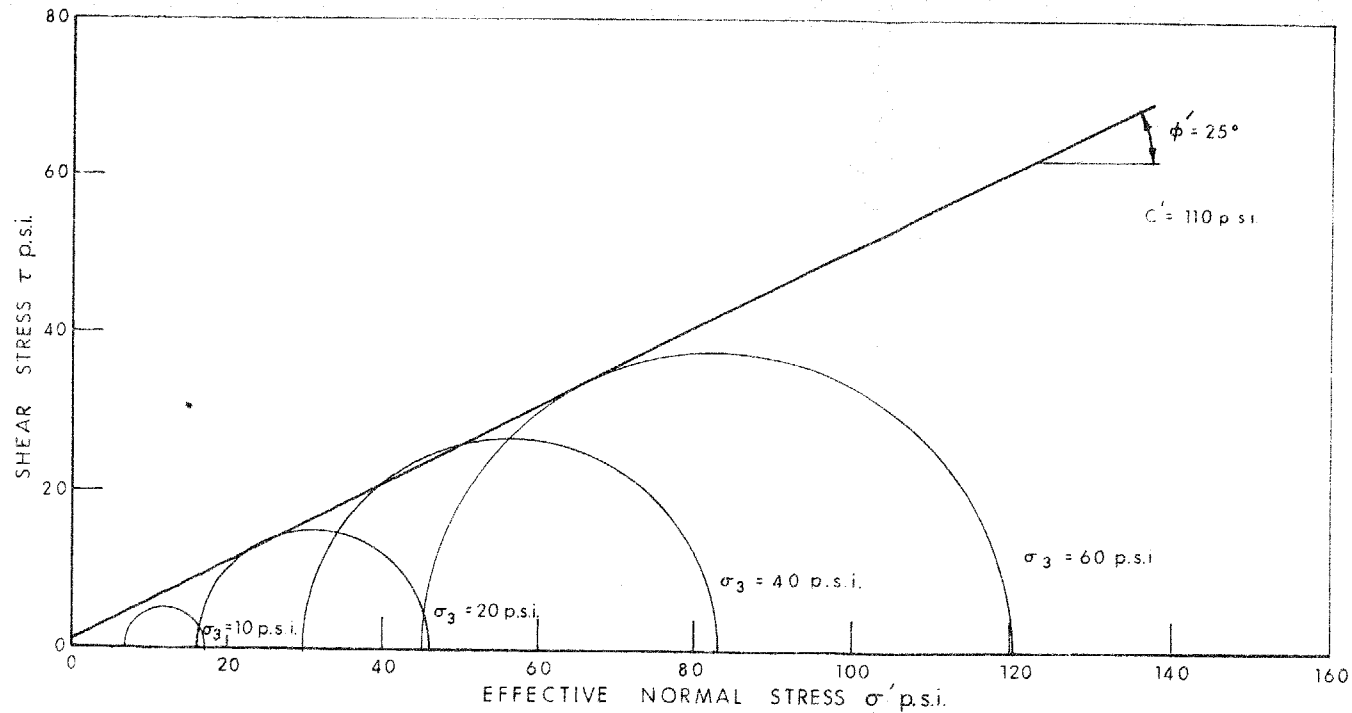
# UNCONFINED COMPRESSION TEST, TYPICAL STRESS STRAIN CURVES (GREY CLAY)



JOB NO. 64 - F - 53

FIG. 7

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS



BORE HOLE NO. ....7  
 SAMPLE NO. ....6  
 DEPTH .....19'-4"  
 BULK DENSITY .....134.5 p  
 LIQUID LIMIT .....27.3 %  
 PLASTIC LIMIT .....17.0 %  
 INITIAL MOISTURE CONTENT...16.2 %  
 FINAL MOISTURE CONTENT...14.1 %

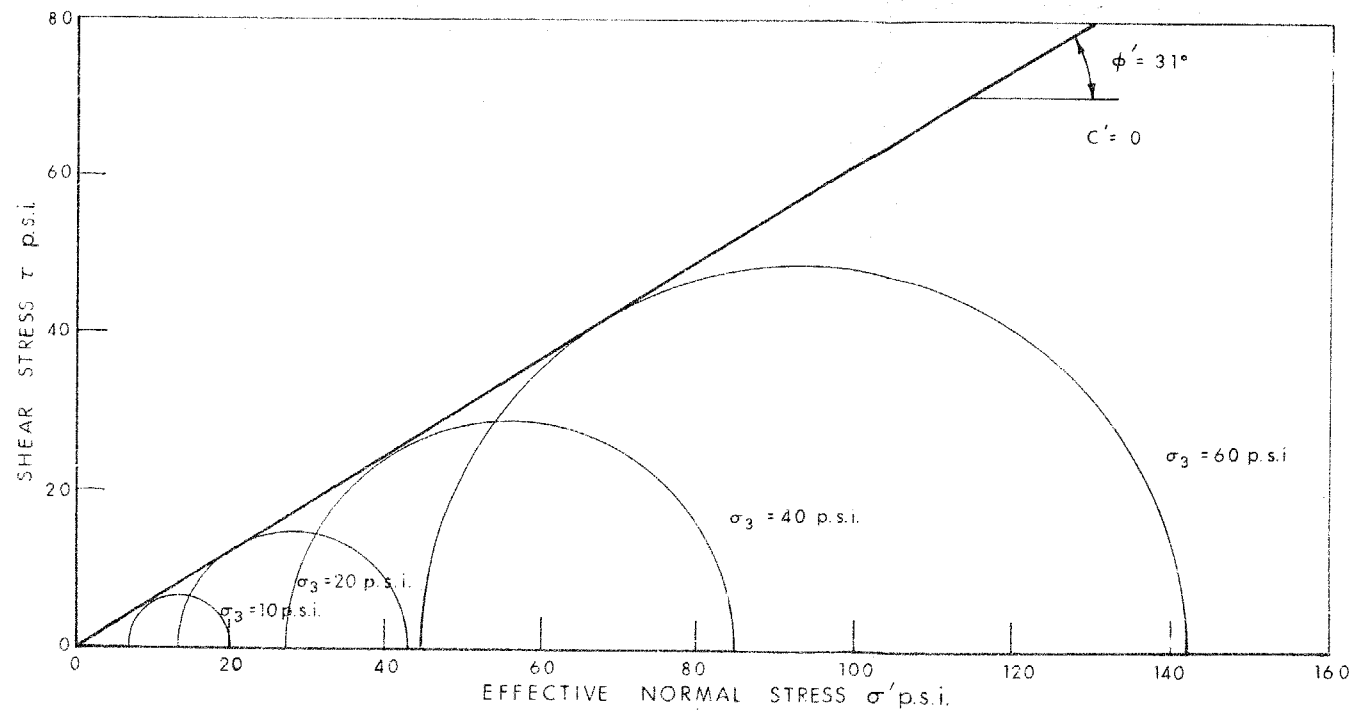
NOTE: (1)  $\sigma_3$  MAINTAINED CONSTANT  
 WITH  $\sigma_1$  INCREASING.

JOB - 64 - F - 53

FIG. 8

ORIGINAL REVISOR: 12-5

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS



NOTE:  $\sigma_3$  MAINTAINED CONSTANT  
WITH  $\sigma_1$  INCREASING

BORE HOLE NO. .... 7

SAMPLE NO. .... 7

DEPTH ..... 21'-4"

BULK DENSITY ..... 141.0 p.c.f.

LIQUID LIMIT ..... 23.7 %

PLASTIC LIMIT ..... 17.0 %

INITIAL MOISTURE CONTENT ..... 13.4 %

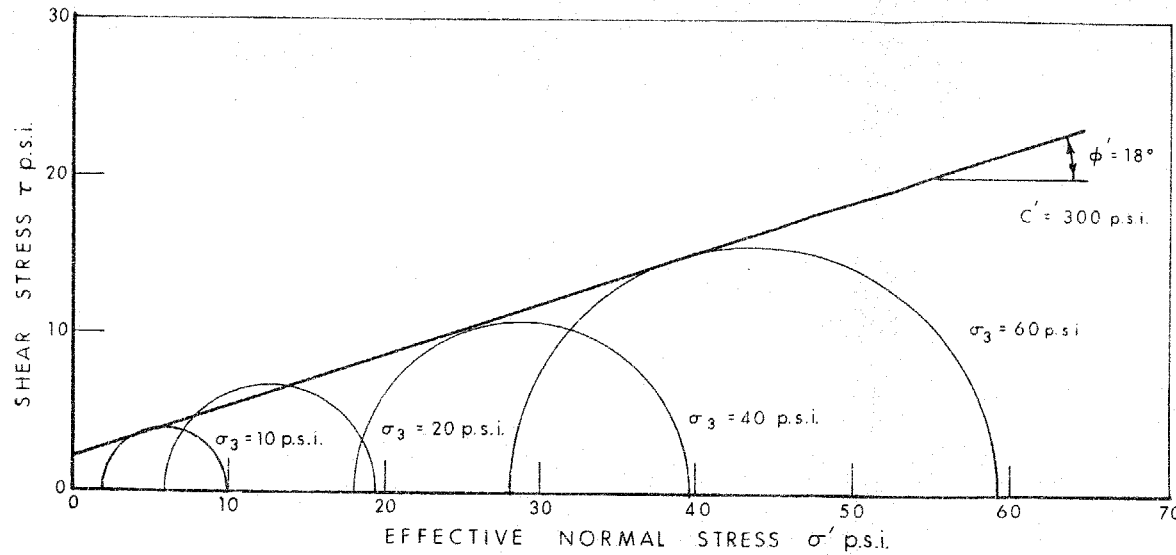
FINAL MOISTURE CONTENT ..... 11.4 %

JOB - 64-F-53

FIG. 9

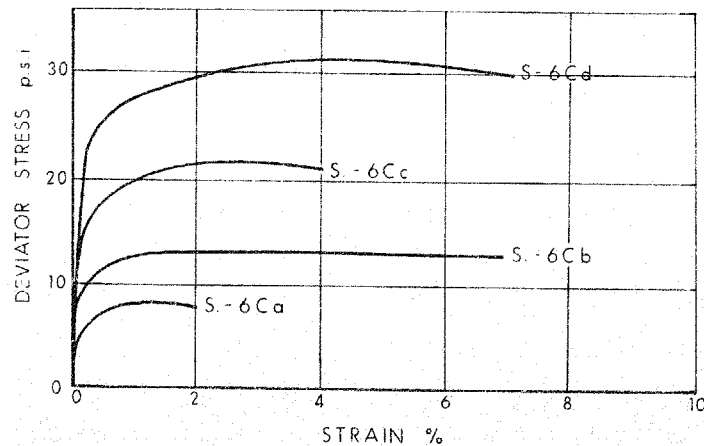
ORIGINAL REVISED (12)

# CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS



BORE HOLE NO. 1

SAMPLE NO'S .....	6Ca	6Cb	6Cc	6Cd
DEPTH .....	52'-4"	52'-8"	53'-0"	53'-4"
BULK DENSITY ( $\rho_s f$ ) .....	117	115	118	113
LIQUID LIMIT .....	49%	49%	49%	49%
PLASTIC LIMIT .....	23%	23%	23%	23%
INITIAL MOISTURE CONTENT .....	38.1%	40.4%	36.7%	44.0%
FINAL MOISTURE CONTENT .....	39.0%	41.2%	35.8%	38.6%

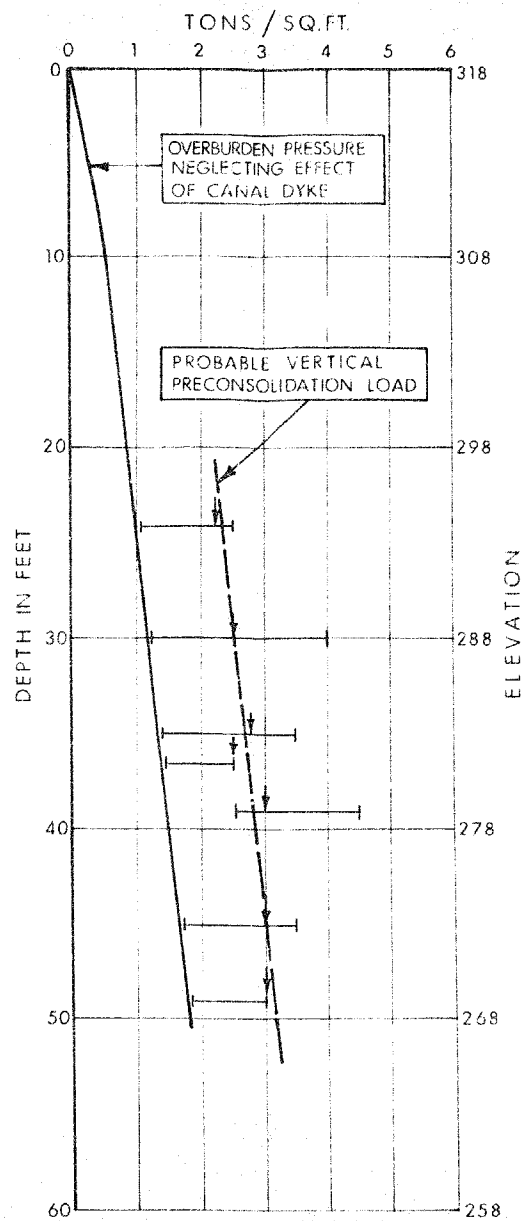
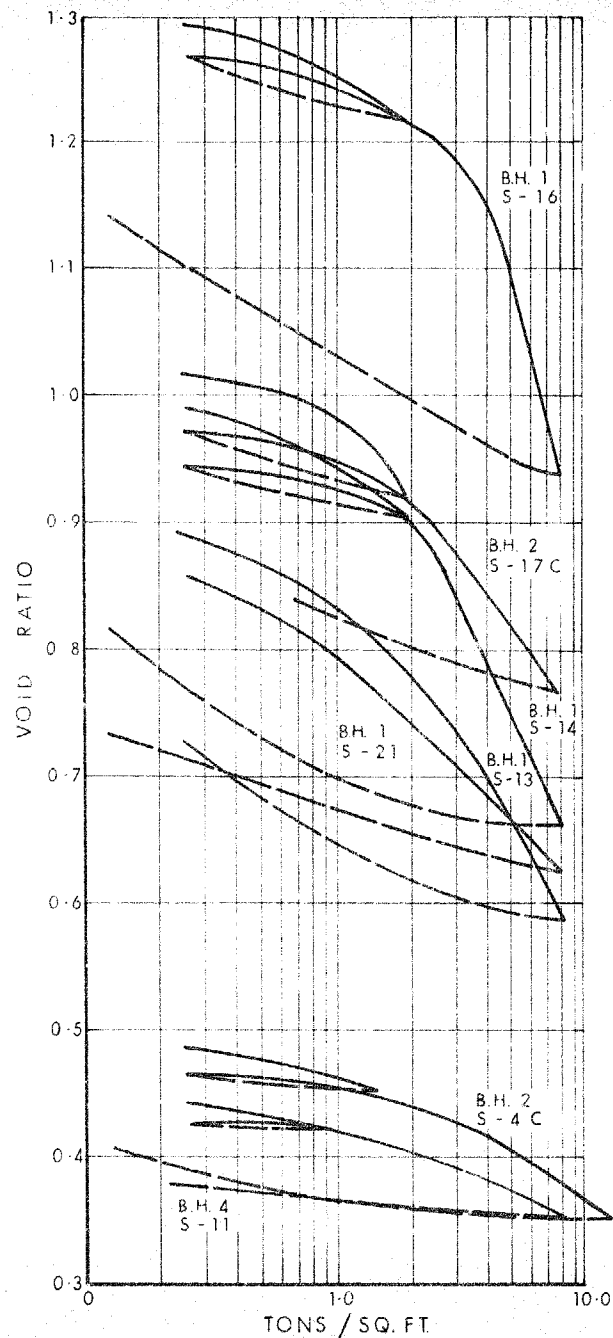


- NOTE (1)  $\sigma_1$  MAINTAINED CONSTANT WITH  $\sigma_3$  DECREASING.
- (2) SHEAR STRENGTH DEFINED AS THE MAXIMUM PRINCIPAL STRESS DIFFERENCE  $\sigma_1 - \sigma_3$

JOB - 64 - F - 53

FIG. 10

ORIGINAL REVISED (14a)



B.H. NO.	SAMPLE NO.	ELEVATION FEET	DEPTH FEET	$e_0$	$P_0$ T.S.F.	$W_L$ %	$W_p$ %	$W_L$ %
1	S-13	283.0	35.0	0.96	1.30	50.8	26.7	37.2
1	S-14	282.0	36.0	1.05	1.35	44.1	23.6	39.1
1	S-16	279.0	39.0	1.31	1.40	44.6	23.2	48.6
1	S-21	273.0	45.0	0.91	1.60	30.5	24.2	34.1
2	S-4C	288.0	30.0	0.49	1.15	25.3	15.7	18.1
2	S-17C	269.0	49.0	1.10	1.71	39.9	27.5	40.5
4	S-11	293.5	24.5	0.45	1.00	24.1	13.8	15.5

NOTE: Depths are measured from existing original ground level not from top of dyke.

ESTIMATED RANGE OF PRECONSOLIDATION LOAD  
 MOST PROBABLE PRECONSOLIDATION LOAD  
 (SCHMERTMAN 1953)

# SUMMARY of CONSOLIDATION TESTS and ESTIMATED PRECONSOLIDATION LOAD

W. P. 444-64 & 65  
JOB. 64-F-53

# WELLPOINT DEWATERING OF CANADA LIMITED

881 EAST 141st STREET • BRONX, N. Y. 10454

Airmail, First Class

May 18, 1966

Department of Highways  
Materials & Testing Division  
Highway 401 and Keele Street  
Downsview, Ontario, Canada

Attention: Mr. A. G. Stermac, Principal Foundation Engineer


Dear Mr. Stermac:

Please find enclosed two (2) FOUNDATION INVESTIGATION REPORTS for Carlton St. Tunnel under the Welland Canal at St. Catharines, Ontario. District 4 (Hamilton), which we are returning to you.

Thank you very much.

Very truly yours,

WELLPOINT DEWATERING OF CANADA, LTD.

  
Richard W. Loughney  
Vice President - Chief Engineer

RWL:jb  
encls. (2)



Department of Highways Ontario

Copy for the information of  
Mr. A. Stermac, Principal Foundation Engineer,  
Room 107, Lab. Building

*File*  
*62-53*  
*File*

Bridge Division,  
Downsview, Ontario,  
June 9, 1966

St. Lawrence Seaway Authority,  
P.O. Box 200,  
St. Laurent,  
Montreal 9, Quebec

Attention: Mr. T.G. Tustin, P. Eng.

RE: Wellpoint Installation

Dear Sir:

Herewith are two copies of the report on last winter's pumping test at the Carlton Tunnel site. Although this installation is probably redundant, these records may be of use for other sites.

The costs were as follows:

Installation	\$65,500 plus \$700 per eductor well
	\$ 340 per day while pumping
	\$ 140 per day when supervising but not pumping
	\$ 78 per day - no supervision
	\$ 39 per day when pumps are removed
Remove and Reset Pumps	\$1,074
Remove System	\$7,000

Yours truly,

*F.I. Hewson*  
F.I. Hewson, P. Eng.,  
Senior Bridge Liaison Engineer

FIH:rd  
Encls.

c.c. W.A. O'Neil (2 reports)  
A.G. Stermac

Hwy. 401 & Keele St.,  
Downsview, Ontario.

Materials and Testing Division

February 16, 1965

Mr. F. Sutcliffe,  
General Engineering Co. Ltd.,  
100 Adelaide Street West,  
Toronto, Ontario.

Proposed Draw-down Condition  
Carlton Street Tunnel  
W.P. 444-64 -- District 4

Dear Sir:

This is to confirm the telephone discussion of this morning concerning the planned filling and emptying of the tunnel excavation.

In view of the flooding that occurred and the subsequent pumping of the water, we feel that this incident could be considered as the planned draw-down condition. At the time of the flooding, about 80% of the material was already excavated. The steepest portion of the excavation slope (1:3 slope) was completed and part of it failed even before flooding. However, the remainder of the steep slope was not affected by the draw-down.

Another draw-down condition of the same duration as the one just being completed would not, in our opinion, provide any additional information that could be usefully applied.

It appears that the flooding and subsequent pumping had practically no effect on the piezometer readings on the North side of the excavation. It remains to be seen whether a prolonged submergence and subsequent pumping will be reflected on these piezometers.

Yours very truly,

*A. G. Stermac*  
A. G. Stermac,

Principal Foundation Engineer

AGS/MdeF

cc: Foundations Office  
Gen. Files

L011 - CARLTON STREET TUNNEL

MINUTES OF MEETING

DATE : 9:00 a.m., Thursday, December 17, 1964

PLACE : Department of Highways Ontario

PRESENT : Department of Highways Ontario

A. Stermac  
K. Y. Lo  
K. Selby  
T. Chan

General Engineering Company Limited

F. H. Sutcliffe

This meeting was held to discuss the results of the slope stability analyses being carried out by the DHO.

CONTRACT I (DHO 64-351)

All slopes for this winter's excavation were checked using the total stress analyses for construction conditions and an effective stress analysis for drawdown conditions. It was agreed that all slopes would be stable during construction, however, the effective stress analysis showed that the 1 to 3 slope on the north side of the excavation, and the 1 to 1, 15' berm, 1 to 1, slope on the south side may be unstable during drawdown.

The following suggestions were made concerning this winter's work:

1. Additional Slope

A 100' length of the north side of the excavation should be cut back to an average slope of 2 to 1, to give one additional case for study.

2. Drawdown

If the construction schedule will permit, a test drawdown should be carried out before the final flooding of the trench

and the canal. This would permit an on-site evaluation of the effect of drawdown and would be useful for the final design of slopes to be excavated in winter 1965-66.

### 3. Instrumentation

As soon as the canal is drained (by approximately December 21), DHO will install the following instruments:

- (a) Piezometers - 3 piezometers on each side of the trench before excavation starts.  
                  - 2 piezometers on the berm and 2 piezometers in the bottom of the trench after the excavation is completed.
- (b) Slope indicators - one slope indicator on each side of the trench before excavation starts.  
                          - one slope indicator on the berm after the excavation is completed.
- (c) Augers - 3 augers will be drilled down to about 4 feet below final excavated grade before excavation starts. They will be located, and elevations taken. Following completion of the trench, they will be uncovered by hand excavation, and elevations checked.

### CONTRACT II

DHO reported that boreholes taken around the perimeter of the proposed back-up dikes and in the drydock area uncovered no unusual conditions.

Further work is being carried out on the stability of the slopes of the excavation through the east and west dikes, and long-term stability of the approach slopes.

Hwy. 401 & Keele St.,  
Downsview, Ontario.

Materials and Testing Division

January 20, 1965

General Engineering Co. Ltd.,  
100 Adelaide Street,  
Toronto, Ontario.

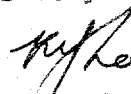
Attention: Mr. F. Sutcliffe

Dear Sir:

This is to confirm our telephone conversation regarding the abandonment of installation of vertical movement augers prior to the excavation at the bottom of the canal.

We feel that in order to obtain accurate levels of the auger, elaborate procedures may have to be used. Since time will not permit, we suggest that your idea of putting them in after the excavation is completed should be adopted.

Yours very truly,



K. Y. Lo,

SUPERVISING FOUNDATION ENGINEER

KYL/MdeF

R. M. P. HAMILTON B.Sc.  
PER. HALL M.Sc.  
NORMAN D. LEA S.M.  
P. D. P. HAMILTON B.Sc.  
E. H. McLEAN B.A. Sc.  
H. L. TAMPLIN P.Eng.  
JAMES BALL B.A. Sc.

## GENERAL ENGINEERING COMPANY LIMITED

100 ADELAIDE ST. W., TORONTO 1, CANADA. TELEPHONE 363-5681, Cable GECORING



October 7, 1964.

Your File W. P. 444-64  
Our File 4011

Mr. H. W. Adcock,  
Assistant Deputy Minister - Engineering,  
Department of Highways Ontario.

Dear Mr. Adcock:

### CARLTON STREET TUNNEL

As agreed during our meeting on September 18, 1964, we are enclosing eight (8) copies of our addendum to our report on the Carlton Street Tunnel.

Should you require any further details, please do not hesitate to contact us.

Yours very truly,

GENERAL ENGINEERING COMPANY LIMITED,

F. H. Sutcliffe, P. Eng.

R. M. P. HAMILTON B.Sc.  
PER HALL M.Sc.  
NORMAN D. LEA S.M.  
P. D. P. HAMILTON B.Sc.  
E. H. McLEAN B.A. Sc.  
H. L. TAMPLIN P.Eng.  
JAMES BALL B.A. Sc.

## GENERAL ENGINEERING COMPANY LIMITED

100 ADELAIDE ST. W., TORONTO 1, CANADA. TELEPHONE 363-5681, Cable GECORING



October 13, 1964.

Mr. A. Stermac,  
Principal Foundations Engineer,  
Department of Highways Ontario,  
Downsview, Ontario.

Dear Sir:

Re: Carlton Street Tunnel

Please find enclosed for your study and comments one copy of our specifications and the following drawings for the first contract of the above project:

General Plan

Details of Excavation and Gravel Road

Yours very truly,

GENERAL ENGINEERING COMPANY LIMITED,

*F. H. Sutcliffe*

F. H. Sutcliffe, P. Eng.

FHS/jf  
4011

SPECIAL PROVISIONS

FOR

CONTRACT NO.

GENERAL SPECIAL PROVISIONS:

The following DHO General Special Provisions shall form a part of this Contract:

Restoring Road Surfaces	(e)
Excess Loading of Motor Vehicles	(bb)
Other Contractors within or Adjacent to the Limits of Contract	(l)
Notice of Contractor-Subletting, Rental of Equipment, Purchase of Material	(n)
Surface Boulders	(p)
Weighing of Materials	(q)
Traffic Control, Flagging	(t)
Work in Open Trenches, Tunnels and Open Caissons	(u)
First Aid Equipment	(v)
Property Owner's Release of Pit Areas on Privately Owned Land Used by the Contractor	(w)

SCOPE OF WORK:

The work covered by this Contract consists generally of the following items:

- 1) Construction and maintenance of a gravel road from Carlton Street on the east side of the Welland Canal down to the bed of the canal.
- 2) The excavation of a trench in the bed of the Welland Canal, and the removal and storage outside the dykes of the excavated material.
- 3) The diversion of the existing drainage ditch in the bed of the canal around the trench excavation.



- 4) Contingent works as may be requested and directed by the Engineer.

LOCATION OF WORK:

The work is located in the bed of the Welland Canal, south of Lock No. 2 and Bridge No. 3 on St. Lawrence Seaway Authority property. The west side of the canal is in the city of St. Catharines, and the east side in the Township of Niagara, all in the County of Lincoln. The storage of excavated material will be on property owned or rented by either the St. Lawrence Seaway Authority, or the Department of Highways Ontario.

The longitudinal centre-line of the trench to be excavated in the canal bed is parallel to and approximately 200 feet south of Carlton Street on the east side of the canal. This dimension will be fixed accurately by the Engineer at the start of construction, but will not be more than 250 feet nor less than 150 feet.

PURPOSE OF WORK:

The work to be carried out is the first phase in the construction of a 4-lane vehicular tunnel under the Welland Canal. The purpose of this initial excavation is to permit the in-situ observation of the various soil strata encountered, and the performance of the excavation slopes. It is expected that the dimensions, grades and slopes including the widths of the berms, as shown on the drawings will be modified by the Engineer as a result of observations made while the work is being performed. The total quantity to be excavated, may therefore, be modified, upwards or downwards, at the discretion of the Engineer, subject to the Provisions of clause 103-1 "Estimated Quantities" of the Department of Highways Ontario "General Conditions of the Contract".

CO-OPERATION WITH OTHERS:

Due to the nature of the work, several types of testing device will be set up and operated by others at the site before, during and after the work.

The testing equipment will consist of piezometers, slope indicators, slope stakes, etc., and the Contractor shall ensure that adequate protection against damage to such installations is provided. In the case of their destruction or removal by him, his agents or employees, these installations shall be replaced by the Engineer at the Contractor's expense.

DRAWINGS:

The following drawings indicate the location, extent, and details of the work and form a part of this Contract:

<u>Drawing Number</u>	<u>Title</u>
	Soil Stratigraphy
	General Plan
	Details of Excavation & Gravel Road

CONSTRUCTION SCHEDULE:

The site of the trench excavation will be available to the Contractor, only during the winter non-navigation season 1964-65 while the canal is drained. The following table indicates the dates of drawdown and refilling of the reach of the canal above Lock No. 2 for the past four years. Drawdown and refilling each take about two days.

<u>Winter</u>	<u>Date Drawdown Completed</u>	<u>Date Refilling Completed</u>
1960-61	December 22	Uncertain, probably March 25
1961-62	December 19	March 25
1962-63	December 26	March 23
1963-64	December 24	March 27

The Contractor shall commence work on the gravel road immediately following drawdown in December 1964, and shall complete the entire work, clean-up, and remove all his equipment prior to the refilling of the canal. The Contractor will be advised of this date at least seven (7) days in advance by the St. Lawrence Seaway Authority. Under no circumstances whatsoever, will the refilling of the canal be delayed.

The Contractor shall prepare and submit with his tender a construction schedule indicating the programme he intends to follow in order to complete the work within the period described above. This schedule shall include a list of equipment he proposes to use for the work, and shall be based on a total elapsed time of 70 calendar days between the end of drawdown and the beginning of refilling.

The approval of this construction schedule by the Engineer shall not relieve the Contractor of any of his duties or responsibilities under the terms of the Contract.

DATUM PLANE:

The datum plane for elevations shown on the drawings or referred to in the specifications is that established by the St. Lawrence Seaway Authority.

SOIL CONDITIONS:

A preliminary soil investigation programme has revealed the following soil stratigraphy below the canal bed. Complete results of the soil investigation including laboratory test results on strength properties are available for inspection at the Department of Highways Ontario, Bridge Office or Foundation Office, Downsview, Ontario, and at the Hamilton District Office. Additional soils information obtained in the same general area is available from the St. Lawrence Seaway Authority.

(1) Hard Clay Till

Starting at the canal bed and extending down to an elevation of about 285, there is a stratum of hard clay till containing all grain sizes from clay to gravel. The till grades from hard at the surface to firm near the bottom, and the colour gradually changes from brown at the top to grey near the base.

(2) Firm clay

The firm clay zone generally extends from about elevation 285 to about elevation 265. The upper part of this stratum is designated as firm to stiff brown clay and the lower part is firm grey clay. These two materials are similar in character and show a well-developed horizontal structure with laminations of clay of varying plasticity. There is less evidence of silt and sand in the upper brown clay. The firm grey clay constitutes a zone of low shear strength.

(3) Dense Stratified Granular Material.

Below the stratum of firm clay there is a layer of

stratified granular material varying in thickness from about 6 feet on the east side of the canal to a maximum of about 20 feet on the west side. This material contains layers of silt, sand and gravel, generally water sorted and quite irregular. The stratum contains water and gas, which may be under pressure in certain locations.

(4) Hard Clayey Silt

The dense stratified granular material is underlain by hard clayey silt with N values of over 100.

(5) Very Dense Red Clay Till

A deposit of very dense red clay till containing considerable quantities of fragmented shaly material, overlies the Queenston shale bedrock. The elevation of bedrock is about 220.

The above description of the soil stratigraphy is based on the results of preliminary soil borings, and is representative of the stratigraphy at the locations of the boreholes only. Between the boreholes, the extent, nature and elevations of the layers of soil can be only interpolated and are therefore subject to considerable variation.

CLEAN-UP:

The Contractor shall complete the clean-up of the site prior to the date set by the St. Lawrence Seaway Authority for refilling the canal. He shall remove all equipment, debris, loose objects which may float, and shall ensure that no piles of earth or any object or material whatever protrudes above the original level of the canal bed. The only exceptions shall be the gravel road from

the top of the east dyke to the canal bed, which shall be left in place and any other material left in place with the approval of the Engineer. The Contractor shall be responsible for any damage or delay to navigation which may result from his failure to do so.

CLEARING - ACCESS ROADS Items # 1 and # 2.

The Contractor shall be responsible for the construction and maintenance throughout the duration of the Contract, of all access roads to the areas to be cleared and grubbed, these being the storage areas. The proposed location and alignment of these access roads shall be submitted by the Contractor to the Engineer for approval before the start of construction.

No separate payment will be made for the cost of access road right-of-ways, nor for the construction and maintenance of access roads. All such costs for the access roads to the east side storage area shall be included in the price bid for cleaning this area. All such costs for the access roads to the west side storage area shall be included in the price bid for cleaning this area.

GRUBBING - REMOVAL OF TOPSOIL Items # 3, # 4, # 5, and # 6.

Grubbing shall include the removal and disposal of all topsoil to a depth as required by the Engineer over the complete east and west side storage areas.

Payment will be made for the removal of topsoil at the Contract unit price for "Earth Excavation" items # 5 and # 6.

CONSTRUCTION & MAINTENANCE OF GRAVEL ROADS.

The Contractor shall construct the Gravel Road as shown on the drawings at the unit prices as bid in the tender. The cost of maintenance as necessary will be borne by the Contractor.

STREAM DIVERSION Item # 7

The work under this item is the diversion of a ditch flowing during the non-navigation season, located in the bed of the canal. This water course must be diverted as shown on the drawings before excavation of the trench is commenced.

Every effort must be made to ensure that no water will find its way into the open trench during and after excavation. The diversion dykes may be left in place at the end of the work, providing they do not extend higher than elevation 308.0. The cost of providing and maintaining any access roads from the Gravel road to the site of the Stream Diversion shall be borne by the Contractor and included in the unit price tendered for Stream Diversion.

EARTH EXCAVATION OF TRENCH Items # 8, 9, and 10.

The work under item # 8, 9 & 10 shall include all the earth excavation of the trench in the bed of the canal, as carried out in two stages, Stage I and Stage II, as shown on the drawings and shall include hauling, handling and incorporation of the excavated materials into stockpiles in the Storage areas, as shown on the drawings, all as directed by the Engineer. No excavated material shall be disposed of on the canal bed.

The Contractor shall not excavate outside of the slopes or below the established grade, unless directed by the Engineer.

No backfilling will be allowed to obtain required slopes excepting that when boulders are encountered in cut slopes they may be removed on instructions of the Engineer, and any resulting cavities shall be filled and thoroughly compacted.

Should the Contractor, unless ordered by the Engineer, excavate below grade, he may be required to backfill such excavations with suitable materials and compact it in 6" layers or less, for which no payment will be made for the obtaining, hauling, placing or compaction of such backfill material.

All side slopes, berms and bottom of excavation shall be left in a neat and workmanlike condition, true to the lines and grades shown on the drawings, or as directed by the Engineer. The finished lines and grades shall not deviate, on completion, more than one (1) foot from the cross section shown on the drawings, or as directed by the Engineer, except that the bottom of the excavation shall not deviate from the final grade by more than six (6) inches.

All the excavation of Stage I shall be carried out before the excavation of Stage II is commenced.

Special care shall be exercised and twice daily inspections shall be carried out by the Contractor during excavation of the trench in order to detect any of the following:

- (1) Surface slides or any movement whatsoever of the excavated slopes.
- (2) Tension cracks in the canal bed or main dykes in the vicinity of the excavation.
- (3) Heaving or a quicking condition in the bottom of the excavation.
- (4) Evidence of gas or water entering the excavation under pressure.



The occurrence of the above, or any other unusual or unexpected conditions, shall be reported to the Engineer without delay.

The material to be stockpiled shall be free of ice and snow and shall be spread uniformly and in an orderly manner and rough graded with adequate provision for drainage. The Engineer may direct that certain materials be separated from other materials and stockpiled in designated areas. All snow shall be removed from the stockpile surface before any material to be stockpiled is added.

The stockpile shall be compacted by ordinary travel of vehicles adding additional material, except that the final surface shall be compacted by a Sheepsfoot Roller, as directed by the Engineer.

The Engineer may direct that certain materials be classified as "Waste". The Contractor shall remove such material from the site of the works and dispose of same to his own satisfaction.

Boulders and rock fragments 27 cu. ft. and greater in volume excavated from within the trench will be measured and paid for as provided in D.H.O. Form # 200, Section 211 "Rock Excavation". 'Measurement for Payment' and 'Basis of Payment' shall be in accordance with D.H.O. Form # 200, Section 210 "Earth Excavation", except for Basis of Payment for "Earth Excavation - Waste" and Maintaining." Payment for "Earth Excavation - Waste" shall be at the unit price per cu. yard as bid in the tender and shall be full compensation for all costs required to dispose of the material away from the site of the works.

The cost of providing any access roads from the Gravel Road to the site of the Trench shall be borne by the Contractor and included in the unit price tendered for "Earth Excavation, Stage I and Earth Excavation Stage II".

The cost of compaction shall be included in the unit price tendered for "Earth Excavation, Stage I and Earth Excavation, Stage II", except that the compaction of the final surface shall be paid for in accordance with D.H.O. Form, Section 214, "Earth Embankment Construction".

UNWATERING OF TRENCH Item # 17.

The trench must be kept dry during and after excavation for the period of the contract. The payment of unwatering the trench shall be at a negotiated unit price per hour for approved equipment and such payment shall be full compensation for furnishing motive power for the pumps satisfactory to the Engineer, all labour, repairs, and all work subsidiary and incidental thereto for which separate payment is not elsewhere provided.

FLOODING OF TRENCH, Item No. 18.

Following the completion of excavation to the lines and grades shown on the drawings or as directed by the Engineer, and prior to the filling of the canal with water for navigation, the Contractor shall fill the trench with water under controlled conditions at a time approved by the Engineer. This time may be just one day prior to the filling of the canal by the St. Lawrence Seaway Authority. It shall be assumed that water for the purpose will be available from water flowing in the ditch in the bed of the canal.

The flooding shall be accomplished and in a manner which will prevent scour or erosion occurring to the slopes or the bottom of the excavation. The method proposed by the Contractor for flooding the trench shall be subject to approval by the Engineer.

Payment for "Flooding of Trench" will be made at the Contract lump sum price, and such payment will be compensation in full for all the work required to flood the trench and to remove, on completion, all necessary equipment from the site.

DEPARTMENT OF HIGHWAYS ONTARIO

## MEMORANDUM

23-64-351

to: Mr. M. Toye,  
Bridge Engineer,  
Bridge Division.

FROM: Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. F. I. Hewson,  
Consultant Liaison  
Engr.

DATE: June 21, 1965

OUR FILE REF.

IN REPLY TO

## SUBJECT:

## FOUNDATION INVESTIGATION REPORT

For

Carlton St. Tunnel under the Welland  
Canal at St. Catharines, Ontario.  
District 4 (Hamilton)

W.J. 64-F-53-2

--

W.P. 444-64

Enclosed, please find our supplemental report on subsoil conditions existing at the above-mentioned site. The report should be read in conjunction with our original Report #64-F-53-1.

It should be noted that we have revised the stratigraphical boundaries slightly and that the revised and new stratigraphical profiles and sections are shown on Drawings #64-F-53B & C.

We believe that the factual information and recommendations contained both in this and the original report will be sufficient for your design purposes.

Should additional information be required, or should any points in the report require further clarification, please feel free to call our Office.

KYL/MdeF

Attach.

cc: Messrs. A. M. Toye (2)  
E. A. Tregaskes  
D. W. Farrer  
G. K. Hunter (2)  
H. Greenland  
T. J. Kovlich

*KYL*  
K. Y. Lo,  
SUPERVISING FOUNDATION ENGINEER

W. Melnyshyn  
General Engineering Co. (GECO) (2)  
Foundations Office  
Gen. Files ✓

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-

# FOUNDATION INVESTIGATION REPORT

For

Carlton St. Tunnel under the Welland  
Canal at St. Catharines, Ontario.

District 4 (Hamilton)

W.J. 64-F-53-2      --      W.P. 444-64

## 1. INTRODUCTION:

Since the original Foundation Report #64-F-53-1 was issued on October 9, 1964, additional exploratory work and some construction in the field has been carried out. The additional exploratory work consisted of drilling a number of boreholes at the locations of the proposed temporary dykes on each side of the Canal and at locations within the Canal bed during the winter months when the water was drawn down and the area became accessible. The construction work in the field was carried out during the draw-down period, January - March 1965, and consisted of the excavation of a 100' wide trench some 410' long across the bed of the Canal. Various test slopes were constructed so that observations on their performance could be made both during and after construction, and in conjunction with this, a number of piezometers and slope indicators were installed and read at frequent intervals by the Foundation Section.

Certain facets of the original investigation such as effective shear strength of the cohesive layers and ground water conditions were not fully reported in Report #64-F-53-1, as the studies were not complete at the time the report was prepared.

cont'd. /2 ...

1. INTRODUCTION: (cont'd.) ...

In addition to this, a number of queries concerning the original investigation have been revised by the Consultant, General Engineering Co. Ltd. (GECO), which could only be answered by the work subsequently carried out.

The present report contains the results of the additional exploratory work, together with the results of our studies relating to the effective shear strength of the cohesive layers and to the ground water conditions prevailing at the site. Comments on the various queries raised by the Consultant are also included.

The results of our instrumentation of the test slopes do not form part of this present report, and will be fully reported at a later date.

This report should be read in conjunction with our original Report #64-F-53-1.

2. FIELD WORK:

A total of twenty-six additional boreholes was carried out in the second phase of the foundation investigation. These boreholes are numbered #14 - #39, inclusive. Drilling and sampling techniques were as described in Report #64-F-53-1. In B.H.'s #30 - #36, inclusive, observation wells for the purpose of measuring earth movements were installed after sampling operations were completed. Geonor type piezometers were also installed at B.H.'s #35, #36 & #39 after the completion of sampling operations.

At the location of B.H. #1, a number of vane tests were carried out in the brown and grey clay layers using a Sprague and

2. FIELD WORK: (cont'd.) ...

Henwood Vane Apparatus so that a comparison with the D.H.O. Vane results might be made.

The locations and elevations of all boreholes are shown on the attached Drawings #64-F-53B & C.

3. SUBSOIL CONDITIONS:

3.1) General:

As a result of the additional borings (#14 - #39), a more accurate estimate of the overall stratigraphical boundaries has been made than that shown on Drawing #64-F-53A which was based on B.H.'s #1 - #13. The most important difference occurs near the west side of the Canal bed where the lower boundary of the grey clay layer was found to extend some 12 feet lower than was previously estimated. Drawing #64-F-53B & C show the revised and new stratigraphical profiles and cross sections and are based on the results of the complete investigation to date. The only other major deviation from our original assessment of subsoil conditions, is the consistency of the upper glacial till deposit in the west half of the Canal bed where the strength was found to be markedly lower than at other locations.

Apart from the points noted above, the information from the additional borings, in general, confirmed our original assessment of subsoil conditions and no further description of individual soil types is considered necessary. Plots of Plasticity Index versus Liquid Limit for the different strata are shown on Figs. 1A & 1B, and grain size distribution curves are shown in Figs. 2 - 4 inclusive.

cont'd. /4 ...



3. SUBSOIL CONDITIONS: (cont'd.) ...

3.1) General: (cont'd.) ...

Various aspects of the subsoil conditions which require further comments and clarification, are discussed below:

3.2) Upper Glacial Till:

As a result of the additional borings and from a visual inspection of the Canal bed after drawdown, the following observations regarding the upper till stratum have been made:

(1) The Canal bed in the vicinity of the future tunnel is covered with up to about 8 feet of material having a soft to firm consistency.

(2) In B.H.'s #35 - #39, which were located in the west half of the excavation across the Canal bed, the remaining portion of the till layer has a much lower undrained strength than that of the same deposit elsewhere at corresponding elevation. This strength is estimated to be in the order of 1,500 p.s.f. - 2,000 p.s.f., and varies randomly with depth.

(3) In all other borings the till deposit was observed to be, in general, as described in our original Report #64-F-53-1. No slickensides or evidence of fissuring was observed in any samples recovered from B.H.'s #1 - #39, inclusive.

3.3) Undrained Shear Strength of the Cohesive Layers:

Till Material -

Reviewing the results of the complete field and laboratory testing programmes covering B.H.'s #1 - #39, it is estimated that the average undrained shear strength of this deposit lies in the

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.3) Undrained Shear Strength of the Cohesive Layers: (cont'd.) ...

Fill Material - (cont'd.) ...

range 2,000 p.s.f. to 2,500 p.s.f. However, for slope design purposes, a somewhat lower value of 1,500 p.s.f. is recommended when the stress/strain characteristics of the soil are taken into account and compared with those of the underlying silty clay deposits.

Upper Glacial Till -

For design purposes, an undrained shear strength of 1,500 p.s.f. is recommended. This value is based on a review of all field and laboratory strength tests carried out in the entire investigation and our estimate of the probable stress/strain behaviour of the hard upper portion of the deposit.

Silty Clay Layers -

Table 1A shows the average undrained shear strengths for the brown clay and grey clay layers computed numerically, for B.H.'s #13 - #39, for both field vane and unconfined compression tests. The grand averages for these boreholes are also given. This Table shows favourable agreement with Table I which gives the same information for B.H.'s #1 - #12, and in consequence, no change in our original recommendations concerning the undrained shear strength of these layers, for slope design purposes, is contemplated. The recommended values are 900 p.s.f. and 700 p.s.f. for the brown clay and grey clay, respectively.

A number of vane tests was carried out in holes drilled adjacent to B.H. #1 using a Sprague and Henwood Vane Apparatus.

cont'd. /6 ...

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.3) Undrained Shear Strength of the Cohesive Layers: (cont'd.) ...

Silty Clay Layers - (cont'd.) ...

This vane apparatus is fitted with a device which enables the torque to be applied at a constant rate of strain and which also permits measurement of the angular displacement of the vane at failure of the soil. The results of these tests are plotted on Fig. 16(a), together with the results of vane tests taken at corresponding elevations in an adjacent hole using a D.H.O. vane. A plot of some typical stress/strain curves obtained from the Sprague and Henwood Vane tests is shown on Fig. 16(b). In general, the D.H.O. vane gave values about 10% higher than the Sprague and Henwood vane, but it is believed that the latter is somewhat unreliable because of the sensitivity of the proving ring and dial gauge which, in this particular case, did not perform well under field conditions. It is also believed that the stress/strain curves plotted on Fig. 16(b) are unreliable from a quantitative point of view and should only be used for comparison purposes.

W.J. 64-F-53

	B R O W N   C L A Y					G R E Y   C L A Y				
	FIELD VANE lbs./sq.ft. AVERAGE MINIMUM		UNCONFINED lbs./sq.ft. AVERAGE MINIMUM		TRIAXIAL lbs./sq.ft. AVERAGE MINIMUM	FIELD VANE lbs./sq.ft. AVERAGE MINIMUM		UNCONFINED lbs./sq.ft. AVERAGE MINIMUM		TRIAXIAL lbs./sq.ft. AVERAGE MINIMUM
1	1390	1280	939	628		1787	1360	736	65	
2	1260	1080	1030	1030		1220	1160	718	679	645   645
15	1360	1360								
16						800	800			
17	740	740								
18	1060	1040	720	580		1333	1200	689	668	
19	1140	600	802	724						
20	1760	1260	1108	774		1093	960	506	506	
21	1027	640	716	699				714	714	
22	1440	1440				1300	1080			
23	1320	1320								
24	1200	1120	907	884		1440	1440	805	805	
25	1280	1280				1040	960	645	645	
26										
27										
28										
29										
35	992	800				995	640			
36	1028	720				856	600			
37	1186	960				1585	1200			
38	1185	1040				1712	1280			
39	1573	1440				1813	1680			
3-39	1232/48 tests		889/24 tests			1305/45 tests		668/11 tests		645/1 test

TABLE I-A

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.4) Effective Shear Strength of the Cohesive Layers:

A number of triaxial compression tests was carried out to determine the effective shear strength parameters,  $C'$  and  $\phi'$ , of the fill material, upper till, brown clay and grey clay layers. The results of these tests are summarized in Table II and are shown in greater detail on Figs. 12 - 15, inclusive.

As a result of these tests, the following soil parameters are recommended for slope design purposes:

<u>Soil Type</u>		
Fill Material	120	23°
Upper Glacial Till	100	27°
Brown Clay	200	20°
Grey Clay	200	20°

cont'd. /9 ...

W.J. 64-P-53

TYPE OF TEST	SOIL TYPE	B.H. No.	SAMPLE No.	Elevation (feet)	C' (lbs./sq.ft.)	$\phi'$ (Degrees)
Consolidated Drained Triaxial Compression Stress Tests With Pore Pressure Measurements.	Fill	12	1D	331.2	134	23
		26	6	321.4	259	20.5
		27	2	333.0	52	28
		28	6	319.8	121	24.5
	Upper Till	7	6	289.8	110	25
		7	7	287.8	0	31
		21	4A	298.9	288	29
	Brown Clay	1	15D	284.2	390	19
	Grey Clay	9P	11	266.5	520	19.5
		9P	11	265.9	230	19
Consolidated Drained Triaxial Compression Test.	Brown Clay	1	6C	286.1	300	18

TABLE II

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.5) Compressibility of the Cohesive Layers:

The results of 10 consolidation tests performed on samples from the upper till, brown clay and grey clay layers are summarized on Fig. 11 (Rev.) which is included in the Appendix of this report. The method and purpose of these tests is described in some detail in Report #64-F-53-1. The individual test results are plotted on Figs. 11(a) - 11(b) and include computed values of the Coefficient of Compressibility ( $M_v$ ) and the Permeability (K) for various consolidation pressures.

For computations of settlement, rebound and recompression due to the consolidation and swelling of the various strata, the recommendations given in the previous report should be followed.

For computations of elastic deflections and rebounds, the following values of soil constants may be used:

<u>Soil Type</u>	<u>Modulus of Elasticity (E)</u>	<u>Poisson's Ratio (<math>\mu</math>)</u>
Upper Till	50 t.s.f.	0.5
Brown Clay	50 t.s.f.	0.5
Grey Clay	50 t.s.f.	0.5

4. GROUND WATER CONDITIONS:

A number of piezometers was installed at this site during the original investigation which commenced in July 1964. The method of installation has been fully described in Report #64-F-53-1 and the locations and elevations of the piezometers are shown on Drawing #64-F-53A. The purpose of the installation was to determine the

cont'd. /11 ...

4. GROUND WATER CONDITIONS: (cont'd.) ...

piezometric head prevailing at different elevations within the various strata between the fill material and the permeable stratified deposits underlying the grey clay. Readings were taken at regular intervals after installation up to April 1965.

Fig. 17(a) shows a plot of Piezometric Head versus Piezometer Tip Elevation for all piezometers installed prior to the Canal drawdown (December 24, 1965), and Fig. 17(b) shows the same plot for the same piezometers some two months after drawdown. An examination of Figs. 17(a) and 17(b) clearly shows that the piezometric head decreases with depth fairly uniformly, and that the effect of the Canal drawdown is mainly a reduction in scatter of the various piezometer readings about the same average line. It is believed that the above phenomenon can be explained by the fact that the permeable stratified deposits occurring below el. 265.0 at the tunnel site occur also in the bed of the Canal in the adjacent lock some 500 feet north. The Canal bed elevation in the downstream lock is 259.0, and the mean water level during the shipping season is about 289.0. In view of these facts, it is believed that the piezometric head in the soil at the tunnel site would tend towards a minimum value of 269.0 if the permeable stratum mentioned above, communicates between the downstream and upstream locks. This, in fact, does appear to be the case.

cont'd. /12 ...



5. MISCELLANEOUS:

The field work for this project was carried out under the supervision of Mr. Paul Payer, Project Foundation Engineer, during the period July 6, 1964 to April 6, 1965. Equipment used was owned and operated by Dominion Soil Investigation, Ltd. This report was prepared by Mr. K. G. Selby, Senior Foundation Engineer.

June 1965

APPENDIX 1.

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - 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>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.		SAMPLE ADVANCED HYDRAULICALLY
	P.M.		SAMPLE ADVANCED MANUALLY

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS 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
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

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 64-F-53-2

LOCATION Sta. 39+09 (363' Lt.)

ORIGINATED BY K.S.

W.P. 444-64 & 5

BORING DATE Nov. 6, 1964

COMPILED BY H.S.

DATUM G.S.C.

BOREHOLE TYPE Penndrill

CHECKED BY P.Mc

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — w			BULK DENSITY P G C F	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			WP	w	WL		
318.5	Groundlevel										
0.0											
			1	SS	42						
					310						
	Clayey silt with traces of sand and occasional gravel. Hard to stiff. (Glacial Till) Brown coloured.		2	SS	41						
			3	SS	31						
					300						
			4	SS	25						
			5	SS	21						
					290						
			6	SS	18						
285.5											
33.0											
	Silty-clay Firm to stiff Brown coloured.		7	TW	PM	+2.3					
					280						
			8	TW	PM	+2.6					118.0
273.2											
46.3			9	TW	PM	+3.2					124.5
	Silty-clay with layers of clayey silt and silt. Firm Grey coloured				270						
			10	TW	PM	+2.0					117.0 114.5
264.1											
54.4	Silt V. dense		11	SS	71						
260.5	Grey coloured.		12	SS	55						
58.0	End of borehole				260						

DEPARTMENT OF HIGHWAYS - ONTARIO

# RECORD OF BOREHOLE NO. 15

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 40+86 (165' Lt.)

ORIGINATED BY K.S.

W.P. 444-64 & 5

BORING DATE Nov. 3, 1964.

COMPILED BY H.S.

DATUM G.S.C.

BOREHOLE TYPE Pennndrill

CHECKED BY P. Mc

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT <u>W<sub>L</sub></u> PLASTIC LIMIT <u>W<sub>P</sub></u> WATER CONTENT <u>W</u>			BULK DENSITY pcf	REMARKS
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT		500	1000	1500	2000	2500	20	40	60		
317.3	Groundlevel														
0.0															
		1	SS	64	310										
	Clayey-silt with sand and occasional gravel.														
	V. stiff to hard.	2	SS	65											
	(Glacial Till)	3	SS	31	300										
		4	SS	20											
		5	SS	19	290										
	Brown coloured														
		6	SS	50											
284.7															
32.6															
	Silty-clay	7	SS	9	280										
	Stiff														
	Brown coloured														
279.8															
37.5	End of borehole														

W.L. @  
Elev 285.8  
31.5

DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 16

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 64-F-53 LOCATION Sta. 41/42 (E) ORIGINATED BY K.S.  
U.S.

W.P. 444-64 & 5 BORING DATE Nov. 3, 1964. COMPILED BY H.S.

DATUM G.S.C. BOREHOLE TYPE Penndrill CHECKED BY P.Mc

SOIL PROFILE		SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ———— WL PLASTIC LIMIT ———— WP WATER CONTENT ———— W wp ———— w ———— L	BULK DENSITY PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE BLOWS / FOOT					
316.7	Groundlevel							
0.0								
	Clayey-silt with sand and occasional gravel.		1 SS 56	310				
			2 SS 62					
			3 SS 25	300				
			4 SS 28					
	Hard to very stiff.  (Glacial Till)		5 TW PH	290				
			6 SS 30					
	Brown		7 TW PH					
282.2			8 SS 62	280				
34.5	Silt with traces of sand and gravel V. dense Brown to grey.		9 SS 71					
			10 SS 52					
276.0								
40.7	Silty-clay stiff Brown							
273.0			11 SS 18					
43.7	Silty clay Firm to stiff							
270.7	Grey							
46.0	End of borehole							

WL @

elev 293.7

23.0

+2.0

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 38+13 (31)

BOREHOLE NO. 17

FOUNDATION SECTION

W. P. 444-64 & 5

BORING DATE Nov. 4, 1904.

ORIGINATED BY K.S.

DATUM G.S.C.

BOREHOLE TYPE Penndrill.

COMPILED BY H.S.

DATUM G.S.C.

BOREHOLE TYPE Pennsylv.

CHECKED BY P. Mc

SOIL PROFILE		SAMPLES		ELEV SCALE	RESISTANCE	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE ROWS / FOOT				
317.4	Groundlevel				500 1000 1500 2000 2500	WATER CONTENT % 20 40 60	
0.0	Clayey silt with sand and occasional gravel.  Hard to very stiff (Glacial Till) Brown coloured)		1 SS 58	310		○ —	
			2 SS 26				
			3 SS 22	300		○ —	
			4 SS 16			○ —	
			5 SS 11			○ —	
289.4	Silty clay. Firm to stiff Brown coloured		6 TW PM	290			
284.4							
33.0	End of borehole			280			

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT





OVER

## FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

LOCATION Sta. 39+83 (683' E.)

ORIGINATED BY K.S.

BORING DATE Nov. 5, 1964.

COMPILED BY H.S.

BOREHOLE TYPE Penndrill

CHECKED BY           P. Mc          

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		BULK DENSITY P T	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. ○ Unconf. Shear Strength + Field Vane Test	WATER CONTENT % 20 40 60		
319.3 0.0	Groundlevel								
	Clayey-silt with sand and traces of gravel.  V. stiff to hard.  (Glacial Till)  Brown coloured.		1 SS 47		310				
			2 SS 34						
			3 SS 15						
			4 TW PH		300				
			5 TW PH						
			6 TW PM		290				134
290.4 28.9	Silty-clay  Firm to stiff.  Brown coloured.		7 TW PH						121.0
			8 TW PM		280				118 117
279.3 40.0		End of borehole							

ORIGINALS OF BORE HOLE LOGS

FILED IN DRAFTING ROOM

UNDER (64-P-53 2H. 1-39)

FOUNDATION SECTION

ORIGINATED BY K.S.

COMPILED BY P. Mc

CHECKED BY P.Mc

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W W P ——— W ——— WL WATER CONTENT % 20 40 60	BULK DENSITY P C F	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE					
107.0 0.0	Groundlevel								
	Clayey-silt with sand and traces of gravel.		1	SS	38	300			
			2	SS	35				
			3	SS	27				
			4	SS	11				
285.5 21.5	Silt with some sand and gravel and traces of clay.  Compact to dense.  Brown to grey.		5	TW	PM	280			
			6	TW	PM and Driven				
			7	SS	27				
			8	TW	PM				
272.2 34.8	Silty-clay Firm to stiff Brown coloured		9	TW	PM	270			
268.0 39.0			10	TW	PM				
	Silty-clay with layers of silt and clayey silt. Some traces of gravel. Firm to stiff.  Grey coloured.		11	TW	PM	260			
258.5 48.5			12	TW	36				
	Silt and sand with traces of gravel.  Dense to very dense  Grey Coloured.		13	SS	73 for 6"	250			
249.5 57.5			14	SS	97 for 10"				

FOUNDATION SECTION

DATUM G.S.C. BOREHOLE TYPE Penndrill CHECKED BY P. Mc

SOIL PROFILE		STRAT PLOT	SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY  P C F	REMARKS
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. O Unconf. Shear Strength + Field Vane Test	WATER CONTENT % 20 40 60				
314.5 0.0	Groundlevel											
	Clayey silt with sand and traces of gravel.											
	V. stiff to hard.											
	(Glacial Till)											
	Brown coloured.											
			1A	TW	PH	310						
			2A	TW	PM							
			3A	TW	PH							
			4A	TW	PH							
			3	SS	25							
			4	TW	PM							
290.5 24.0						290						
	Silty-clay.		5	TW	PM							
	Firm to stiff.						+3.0					
	Brown coloured.		6	TW	PM							
							+2.0					
			7	TW	PM	280						
							+2.7					
276.3 38.2	Silty-clay Firm to stiff											
273.0 41.5	Grey coloured.		8	TW	PM							
	End of borehole					270						

FOUNDATION SECTION

BOREHOLE TYPE Penndrill

[illegible]

FOUNDATION SECTION

ORIGINATED BY K.S.

COMPILED BY: P. Mc

CHECKED BY P. Mc

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL	PLASTIC LIMIT ——— WP	WATER CONTENT — W	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.					
							Unconf. Shear Strength + Field Vane Test					
							500 1000 1500 2000 2500					
319.2 0.0	Groundlevel											
	Clayey silt with sand and occasional gravel.		1	SS	64							
	V. stiff to hard.					310						
	(Glacial Till)		2	SS	55							
	Brown coloured.											
			3	SS	19							
						300						
			4	TW	PH							
			5	TW	PH							
290.2						290						
29.0	Silty-clay V. stiff to stiff.		6	TW	PM							
	Brown coloured.											
282.7			7	TW	PM							
36.5	End of borehole.											
						280						





DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 25

FOUNDATION SECTION

JOB 64-F-53LOCATION Sta. 29+45 (405' Lt.)ORIGINATED BY K.S.W.P. 444-64 & 5BORING DATE Nov. 10, 1964COMPILED BY H.S.DATUM G.S.C.BOREHOLE TYPE Penn-drill & Diamond Drill-NK CasingCHECKED BY P.Me

SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. o Unconfined Lab. Test + Field Vane Test		WATER CONTENT %		
338.9 0.0	Groundlevel					500 1000 1500 2000 2500		20 40 60		
			1 TW PH							
	Silty clay with traces of sand and gravel.		2 TW PH							138 134
	Stiff to v. stiff		3 TW PH		330					134
	(Fill Material)		4 SS 25							
	Brown coloured.		5 TW PH							132
			6 TW PH							
			7 TW PH		320					123
			8 TW PH							
			9 TW PH							133 138
311.3 27.6			10 TW PH	6 <sup>th</sup> PH 12 <sup>th</sup> 26 blows	310					
			11 SS 54							
	Clayey-silt with traces of sand and gravel.		12 SS 45		300					
	Hard.									
			13 TW PH		290					140
287.8 51.1	Silty-clay Firm to stiff Brown coloured.		14 TW PH							140
			15 TW PH		280					
277.4 61.5	Silty-clay with layers of clayey silt and silt. Firm to stiff. Grey coloured.		16 TW PH							No Recovery
			17 TW PH		270					123
266.8 72.1	Silt V. dense Grey coloured.		18 TW PH							
260.9 78.0	End of borehole.		19 TW 20		260					
			20 SS 58							

DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 26

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 23+94 (347' Rt.)

ORIGINATED BY K.S.

W.P. 444-64 & 5

BORING DATE Nov. 10, 1964

COMPILED BY H.S.

DATUM G.S.C.

BOREHOLE TYPE **Pennndrill**

CHECKED BY P. MC

SOIL PROFILE		STRAT PLOT	SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	Liquid Limit — WL Plastic Limit — WP Water Content — W	BULK DENSITY POUND PER CUBIC FOOT	REMARKS
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. o Unconf. Shear Strength + Field Vane Test	WATER CONTENT % WP      W      WL 20    40    60		
340.5 0.0	Groundlevel					340				
	Silty clay with traces of sand and gravel.		1	TW	PH					
	Stiff to very stiff.		2	TW	PH					
	(Fill Material)		3	TW	PM	330				
	Brown coloured		4	TW	PM					
			5	TW	PM					
			6	TW	PM	320				
			7	TW	PM					
316.7 23.8	Clayey silt with traces of sand and gravel.		8	TW	PM & Driven					
	Hard		9	SS	50					
	(Glacial Till)		10	SS	51	310				
	Brown coloured.		11	SS	32					
302.0 38.5	End of borehole.					300				

DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 27

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 64-F-53 LOCATION Sta. 37+68 (395' Lt.) ORIGINATED BY K.S.

W.P. 444-64 & 5 BORING DATE Nov. 12, 1964. COMPILED BY H.S.

DATUM G.S.C. BOREHOLE TYPE Penndrill CHECKED BY P.Mc

[illegible]

FOUNDATION SECTION

CHECKED BY P. Mc

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DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 29

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 28485 (405' Lt.)

ORIGINATED BY K.S.

W P 444-64 & 5

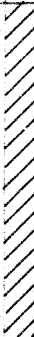

BORING DATE Nov. 19, 1964

COMPILED BY P. Mc

DATUM G.S.C.

BOREHOLE TYPE Washboring - NX Casing.

CHECKED BY P. Mc

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W	BULK DENSITY P O F	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.		
824.6	Groundlevel					500 1000 1500 2000 2500			
0.0	Silty clay with traces of sand and gravel. V. stiff (Fill Material) Brown coloured.		1	SS	16	320			
			2	SS	29				
811.1									
13.5	Clayey silt with traces of sand and gravel.  V. stiff to hard.  (Glacial Till)  Brown to grey coloured.		3	SS	75	310			
			4	SS	72				
			5	SS	34	300			
			6	SS	25				
293.1									
31.5	End of borehole.					290			

FOUNDATION SECTION

CHECKED BY \_\_\_\_\_ T.C.

SOIL PROFILE			SAMPLES	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LQUID LIMIT ——— W <sub>L</sub> PLASTIC LIMIT ——— W <sub>P</sub> WATER CONTENT ——— W	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE BLCHS / FOOT	+ Field Vane 500 1000 1500 2000 2500	W <sub>P</sub> W    W <sub>L</sub> WATER CONTENT % 20    40    60	P.C.F.	
308.0	Groundlevel						
0.0							
	Clayey silt with sand and gravel.		1 SS 4				
	Hard to stiff.						
	Brown		2 SS 24				
	(Glacial Till)						
			3 SS 16				
			4 SS 13				
			5 SS 12				
280.5							
27.5			6 SS 4				
	Silty clay.						
	Firm to stiff.						
	Brown.						
			7 SS 5				
			8 SS 13				
268.5							
39.5			9 SS				
	Silty clay with layers of clayey silt.						
	Firm to very stiff.		10 SS 7				
	Grey.						
			11 SS 5				
			12 SS 20				
251.0							
57.0							
	Sandy silt to silty sand. Very dense						
246.5	Grey		13 SS 35 2"				
61.5	End of borehole						

DEPARTMENT OF HIGHWAYS - ONTARIO		<b>RECORD OF BOREHOLE NO. 36</b>		FOUNDATION SECTION	
MATERIALS & TESTING DIVISION		LOCATION <u>Sta. 33+25 (15' Rt.)</u>		ORIGINATED BY <u>P.P.</u>	
JOB <u>64-F-53</u>		BORING DATE <u>Jan. 8-16, 1965.</u>		COMPILED BY <u>A.B.</u>	
W.P. <u>444-64 &amp; 5</u>		BOREHOLE TYPE <u>Washboring HK Casing.</u>		CHECKED BY <u>T.C.</u>	
DATUM <u>G.S.C.</u>					

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F. + Field Vane					WATER CONTENT %				
							500	1000	1500	2000	2500	WP	W			WL
308.0	Groundlevel															
0.0																
	Clayey silt with sand and gravel. Hard to stiff. Grey-brown (Glacial Till)		1	SS	2			+								
						300				+						
			2	SS	18					+	→					
										+	→					
			3	SS	10					+	→					
					290					+	→					
			4	SS	9				+							
								+								
			5	SS	10				+							
			6	SS	10											
280.0					280											
28.0	Silty clay.		7	SS	8			+								
	Firm to stiff.								+							
	Brown		8	SS	7				+							
			9	SS	7											
			10	SS	6			+								
								+								
270.0					270											
38.0			11	SS	5			+								
	Silty clay.								+							
	Firm to stiff.								+							
	Grey.		12	SS	4											
									+							
					260											
260.0																
48.0			12A	SS	42											
	Clayey silt, silt and sand.		13	SS	46											
	- Layered -															
	Dense to very dense		14	SS	109											
	Grey															
					250											
			15	SS												
246.5																
61.5	End of borehole.															

W.L.  
302.0

Piez. 'D'  
Tip Elev.  
285.0

Piez. 'C'  
Tip Elev.  
275.0

Piez. 'B'  
Tip Elev.  
263.0

DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 37

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 32400 (52' Lt.)

ORIGINATED BY P.P.

W P 444-64 & 5

BORING DATE Jan. 13-15, 1965.

COMPILED BY A.B.

DATUM G.S.C.

BOREHOLE TYPE Washboring, BX & NX Casing.

CHECKED BY T.C.

[illegible]



DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 38

FOUNDATION SECTION

## MATERIALS &amp; TESTING DIVISION

JOB 64-F-53

LOCATION Sta. 31/27 (50' Lt.)

ORIGINATED BY P.P.

W P 444-64 & 5

BORING DATE Jan. 19-20, 1965

COMPILED BY           A.B.          

DATUM G.S.C.

BOREHOLE TYPE Washboring BX & NX Casing.

CHECKED BY T.C.

[illegible]

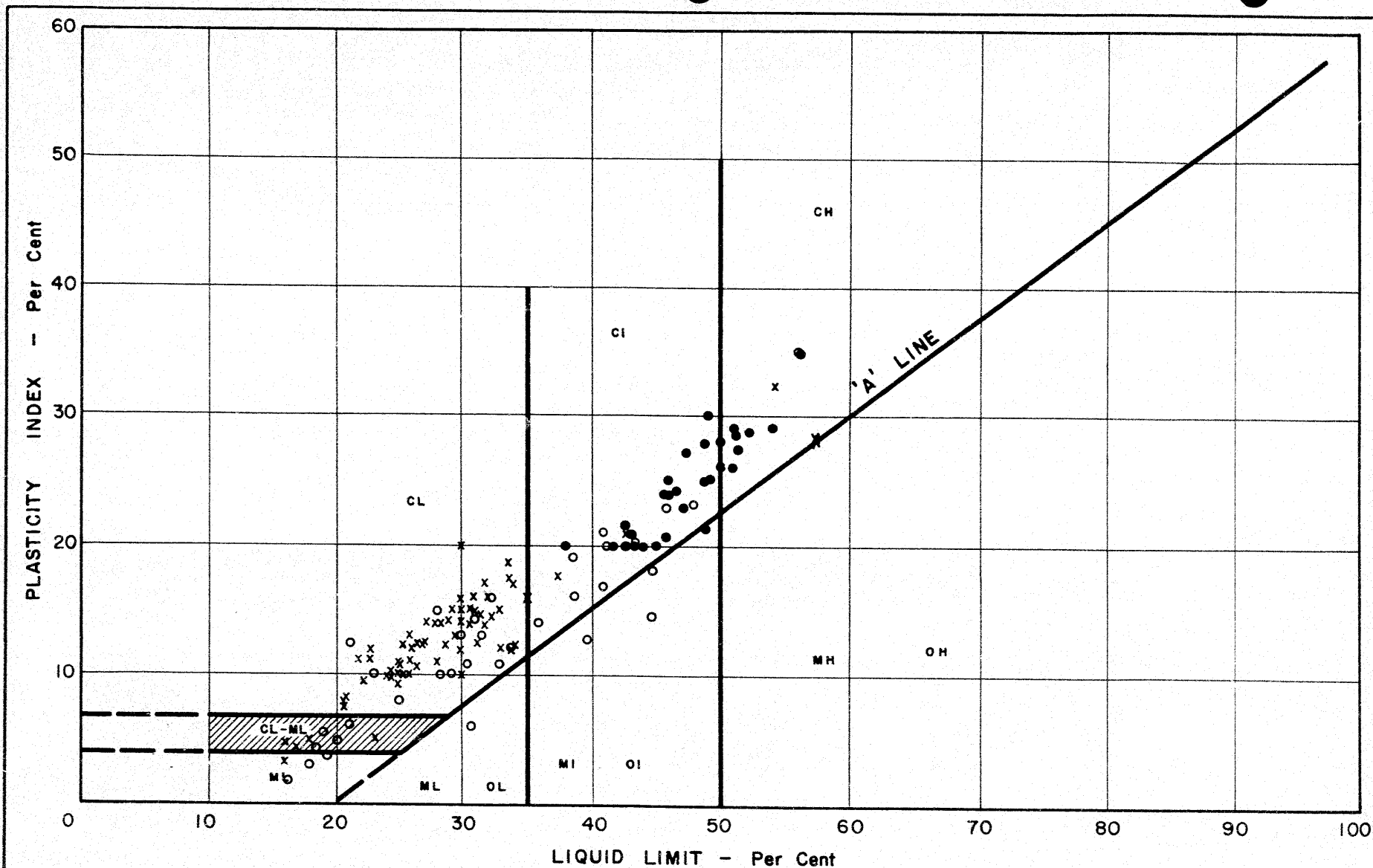
DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 39

FOUNDATION SECTION

JOB 64-P-53 LOCATION STA. 32 + 41 (55' .) ORIGINATED BY P.P.  
W P 44-64 & 65 BORING DATE Jan. 22-25, 1965 COMPILED BY K.B.  
DATUM U.S.C. BOREHOLE TYPE Washboring, NX Casing CHECKED BY \_\_\_\_\_

SOIL PROFILE		SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— wp WATER CONTENT ——— w		BULK DENSITY P.C.F.	REMARKS
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER		TYPE	BLOWS / FOOT	SHEAR STRENGTH P.S.F. + Field Vane	WATER CONTENT % wp      w      wl		
306.2	Ground Level									
0.0.										
	Clayey Silt with Sand & Gravel Soft to Very Stiff (Glacial Till)		1	SS	10	300				
			2	SS	17					
			3	SS	27	290				
			4	SS	44					
			5	SS	28	280				
			6	SS	21					
272.2										
274.0	Silty Clay Stiff Brown		7	SS	11	270				
266.2										
264.0	Silty Clay to Clayey Silt - Layered - Stiff to Very Stiff Grey		8	SS	8					
259.7										
246.5'	End of Borehole		9	SS	18	260				



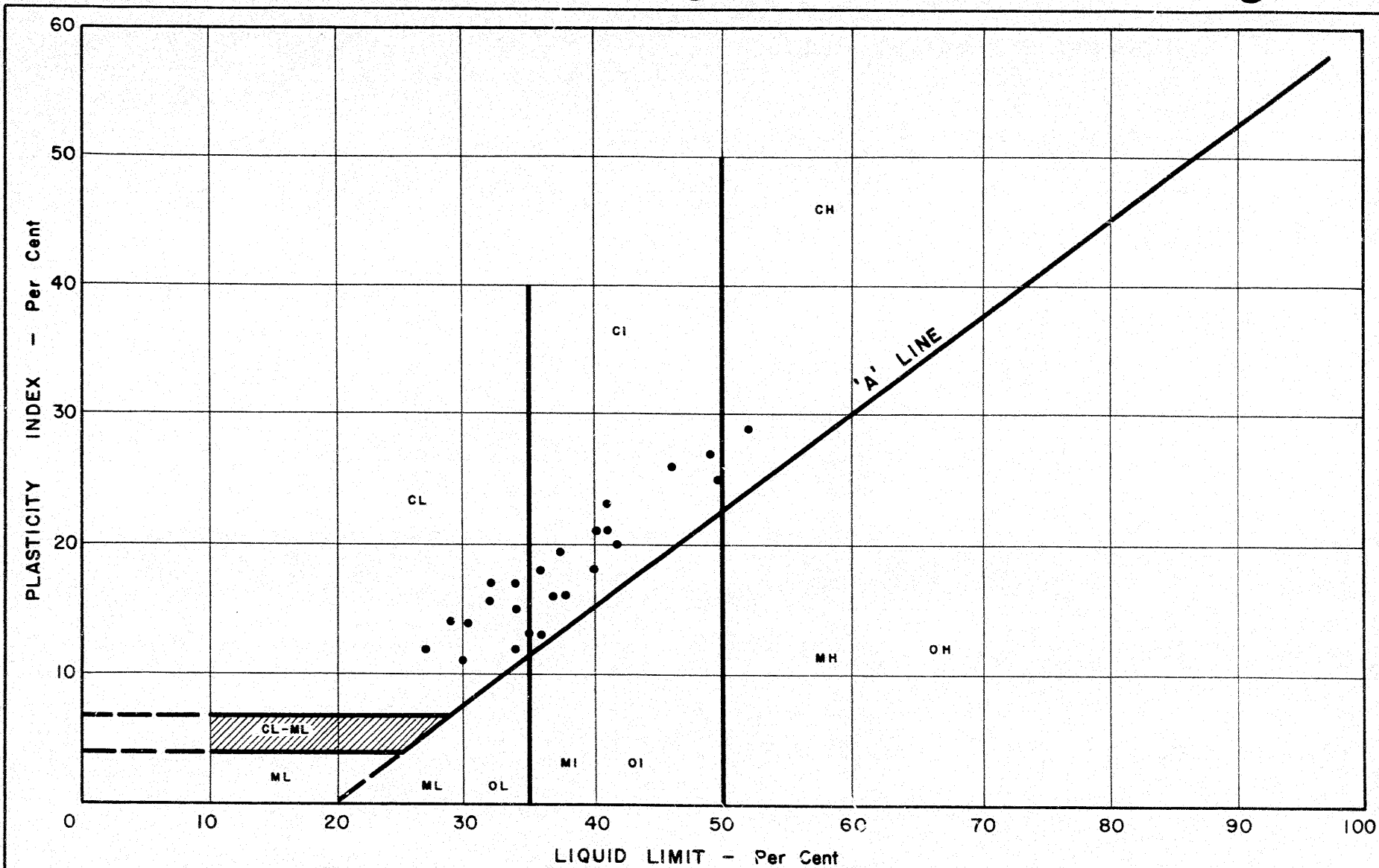
NOTES

- x GLACIAL TILL
- FIRM BROWN CLAY
- FIRM GREY CLAY
- (BORE HOLES 12 TO 39)

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION  
**PLASTICITY CHART**

Job No. 64 - F - 53 W.P. No. 444 - 64  
Location CARLTON ST. - ST. CATHARINES

FIG. 1A



NOTES • FILL MATERIAL - BORE HOLES 25 TO 29

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MATERIALS & RESEARCH DIVISION  
PLASTICITY CHART

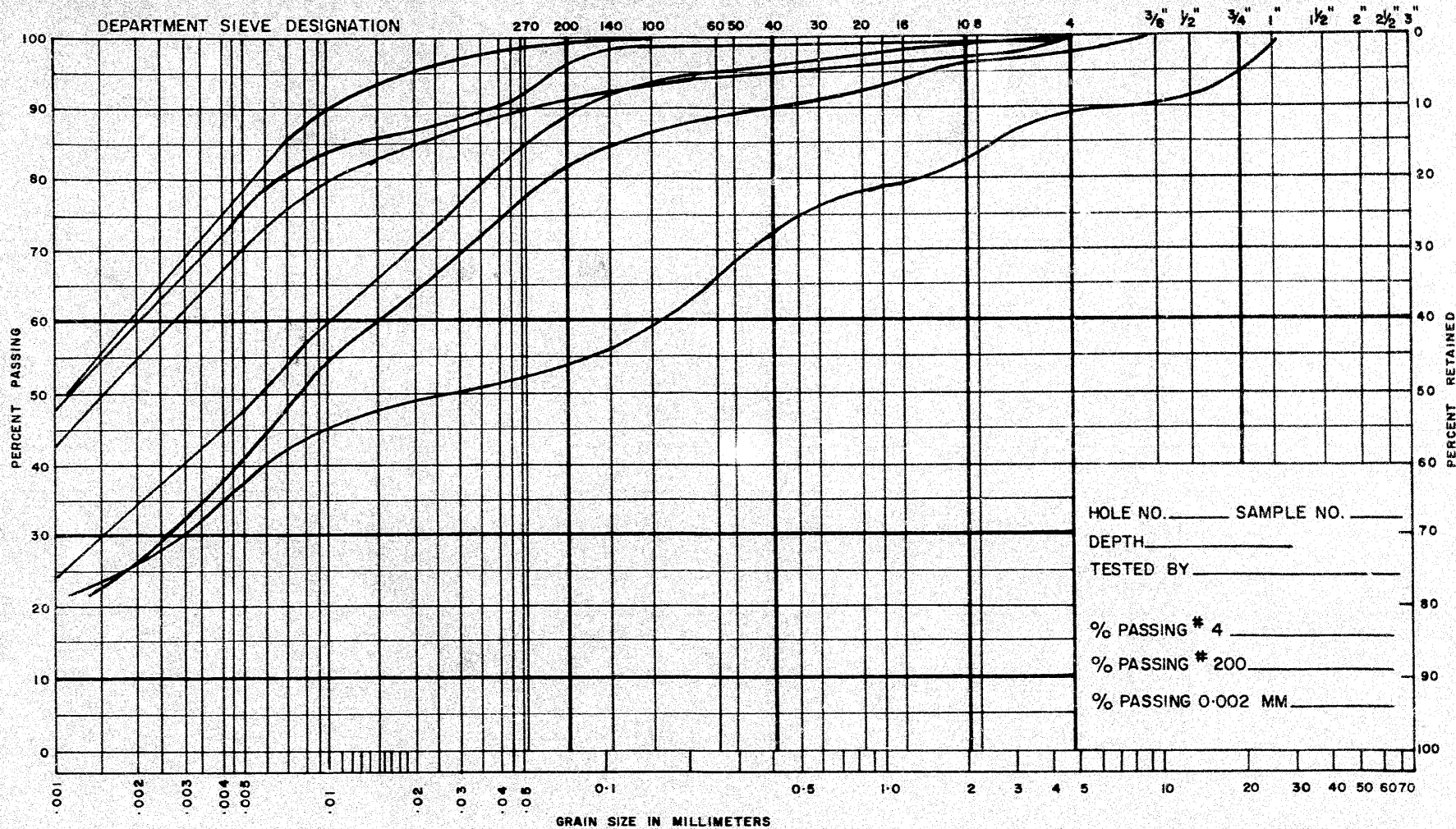
Job No. 64 - F - 53 W.P. No. 444 - 64

Location CARLTON ST. - ST. CATHARINES

FIG. 1 B

# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



NOTES TYPICAL CURVES  
FILL MAT'L. FROM BORE HOLES NO. 1, 8 P & 12 P & 25

DEPARTMENT OF HIGHWAYS — ONTARIO  
 MATERIALS & TESTING DIVISION  
**GRAIN SIZE DISTRIBUTION**  
 JOB NO. 64-F-53 W.P. NO. 444-64  
 LOCATION CARLTON STREET ST. CATHARINES

FIG. 2 (Rev.)



# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt

Sand

Gravel

Fine

Medium

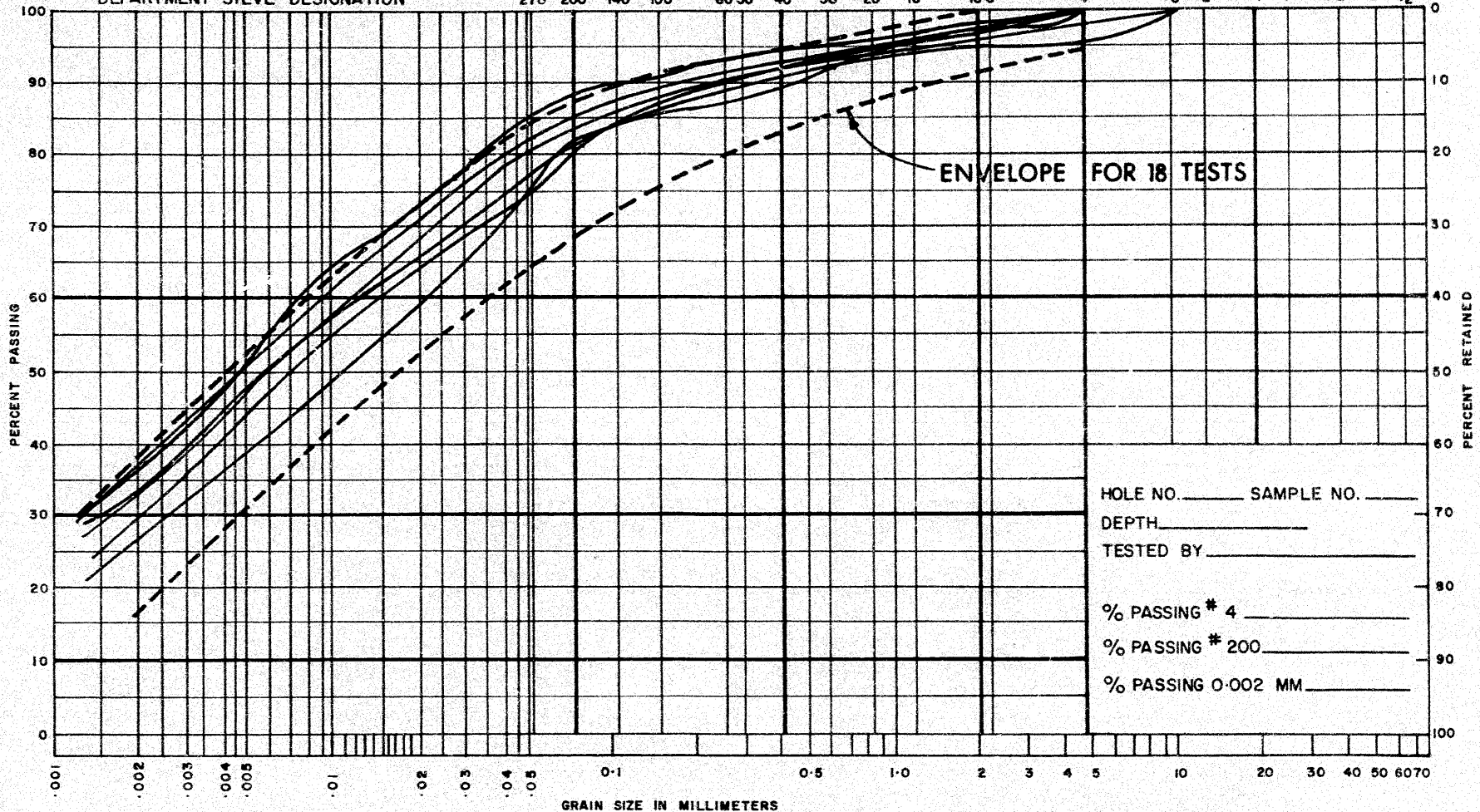
Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270 200 140 100 60 50 40 30 20 16 10 8 4 3/8" 1/2" 3/4" 1" 1 1/2" 2" 2 1/2" 3"



NOTES TYPICAL CURVES

TILL FROM BORE HOLES NO. 1, 2, 3, 4 & 7 & 13

DEPARTMENT OF HIGHWAYS — ONTARIO  
MATERIALS & TESTING DIVISION

## GRAIN SIZE DISTRIBUTION

JOB NO. 64 - F - 53

W. P. NO. 444 - 64

LOCATION CARLTON STREET

ST. CATHARINES

# UNIFIED SOIL CLASSIFICATION SYSTEM

Clay & Silt

Sand

Gravel

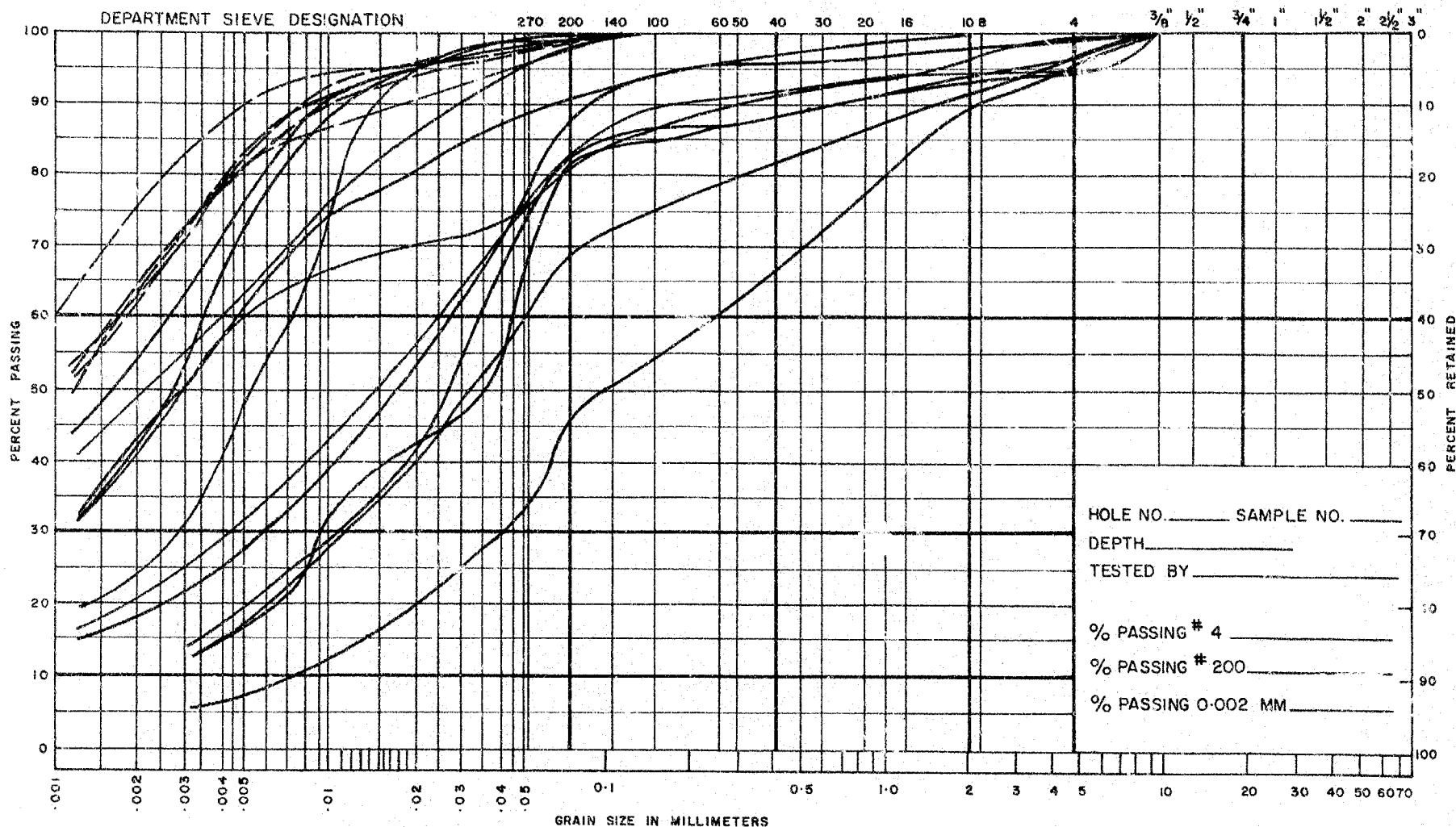
Fine

Medium

Coarse

Fine

Coarse



## NOTES TYPICAL CURVES

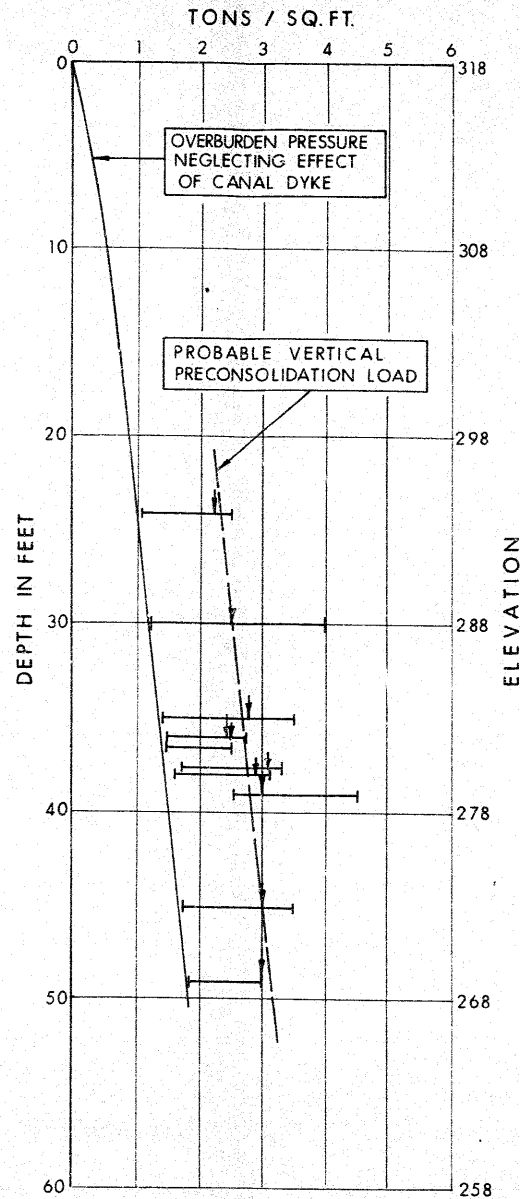
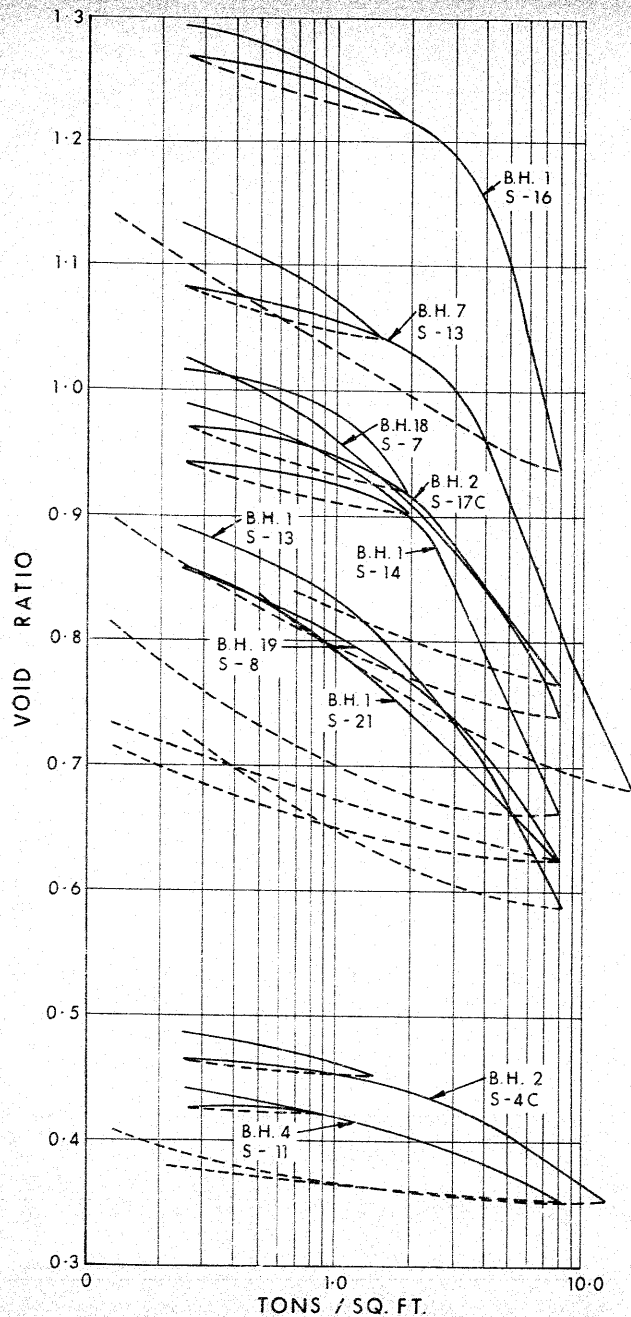
- GREY CLAY FROM BORE HOLES NO. 2C, 8P, 13, 21, 24 & 25
- BROWN CLAY FROM BORE HOLES NO. 1, 4, 18, 21 & 24

DEPARTMENT OF HIGHWAYS — ONTARIO  
MATERIALS & TESTING DIVISION

## GRAIN SIZE DISTRIBUTION

JOB NO. 64 - F - 53 W.P. NO. 444 - 64  
 LOCATION CARLTON STREET ST. CATHARINES

FIG. 4 (Rev.)



B.H. NO.	SAMPLE NO.	ELEVATION FEET	DEPTH FEET	$e_o$	$P_o$ T.S.F.	$W_L$ %	$W_P$ %	$W_i$ %
1	S-13	283.0	35.0	0.96	1.30	50.8	26.7	37.2
1	S-14	282.0	36.0	1.05	1.35	44.1	23.6	39.1
1	S-16	279.0	39.0	1.31	1.40	44.6	23.2	48.6
1	S-21	273.0	45.0	0.91	1.60	30.5	24.2	34.1
2	S-4C	288.0	30.0	0.49	1.15	25.3	15.7	18.1
2	S-17C	269.0	49.0	1.10	1.71	39.9	27.5	40.5
4	S-11	293.5	24.5	0.45	1.00	24.1	13.8	15.5
7	S-13	269.5	38.0	1.15	1.13	36.8	16.4	39.6
18	S-7	281.2	36.0	1.04	1.23	52.1	22.7	39.4
19	S-8	280.8	38.0	0.89	1.29	47.5	21.6	33.9

NOTE: Depths are measured from existing original ground level not from top of dyke

ESTIMATED RANGE OF PRECONSOLIDATION LOAD  
 MOST PROBABLE PRECONSOLIDATION LOAD  
 (SCHMERTMAN 1953)

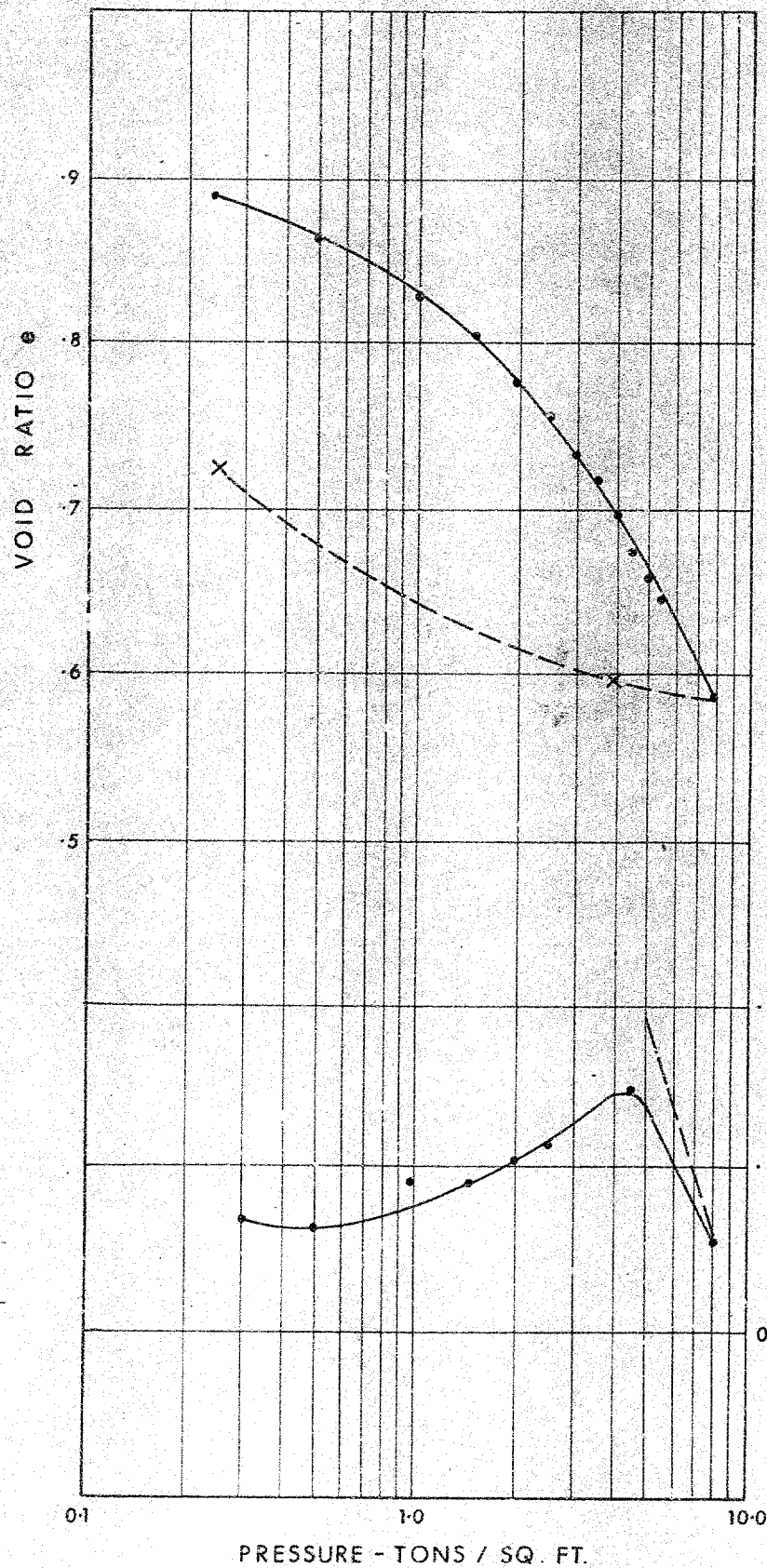
## SUMMARY of CONSOLIDATION TESTS and ESTIMATED PRECONSOLIDATION LOAD

W. P. 444 - 64 & 65  
 JOB. 64 - F - 53

FIG. 11 (REV.)



# BROWN CLAY



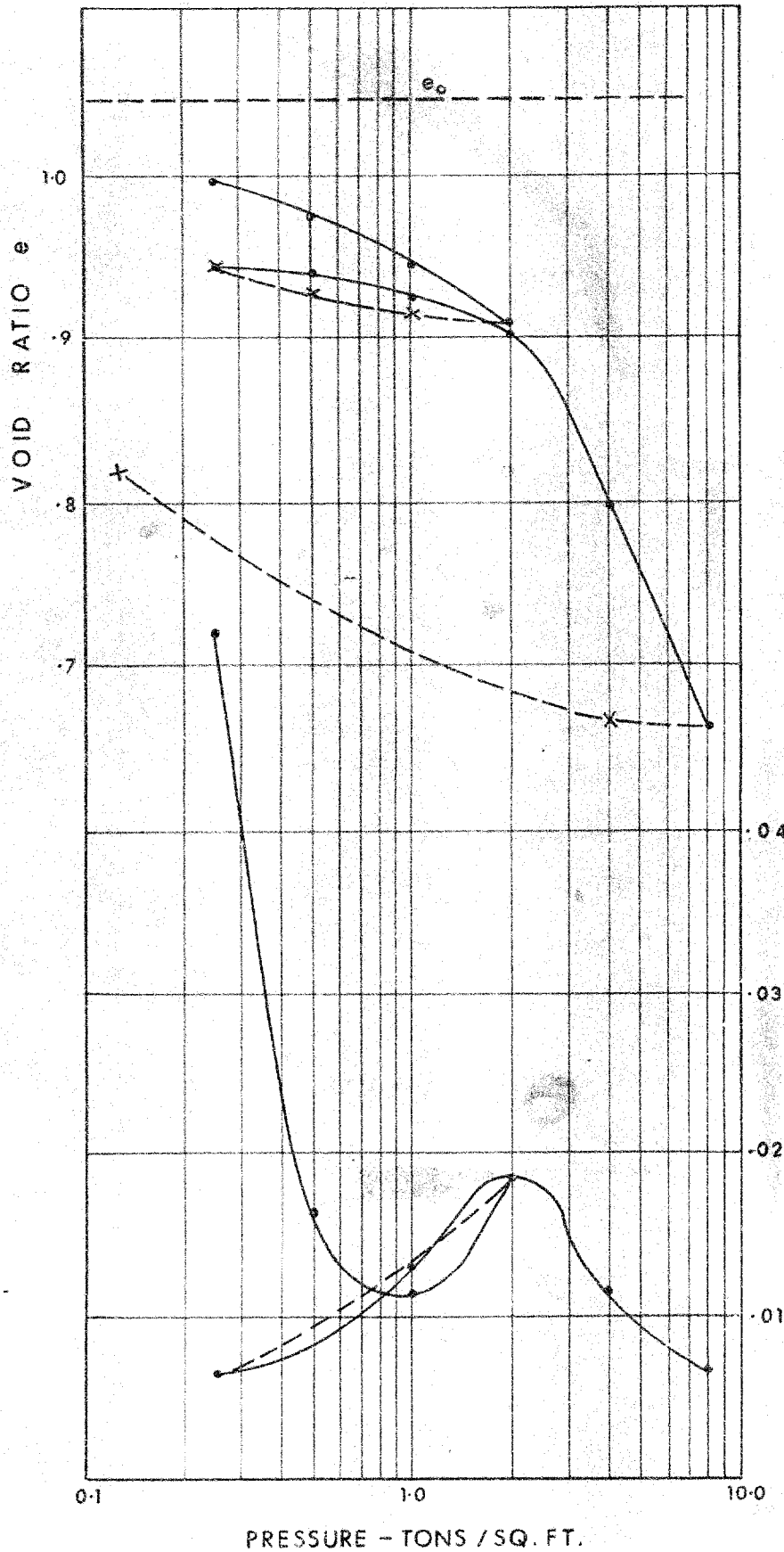
B.H. 1 SAMPLE 13

DEPTH	ELEVATION	$w_L$	$w_P$	$w_I$	$P_o$	$e_o$
35.0'	283.0	50.8%	26.7%	37.2%	1.3 T.S.F.	0.96

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY $M_v$ -FT. <sup>2</sup> / TON	PERMEABILITY $K$ -INS. / MIN.
0.25	$10^{-2} \times 6.67$	$1.19 \times 10^{-6}$
0.50	$10^{-2} \times 4.67$	$0.78 \times 10^{-6}$
1.00	$10^{-2} \times 3.01$	$0.71 \times 10^{-6}$
1.50	$10^{-2} \times 2.66$	$0.62 \times 10^{-6}$

FIG. 11(a)

# BROWN CLAY



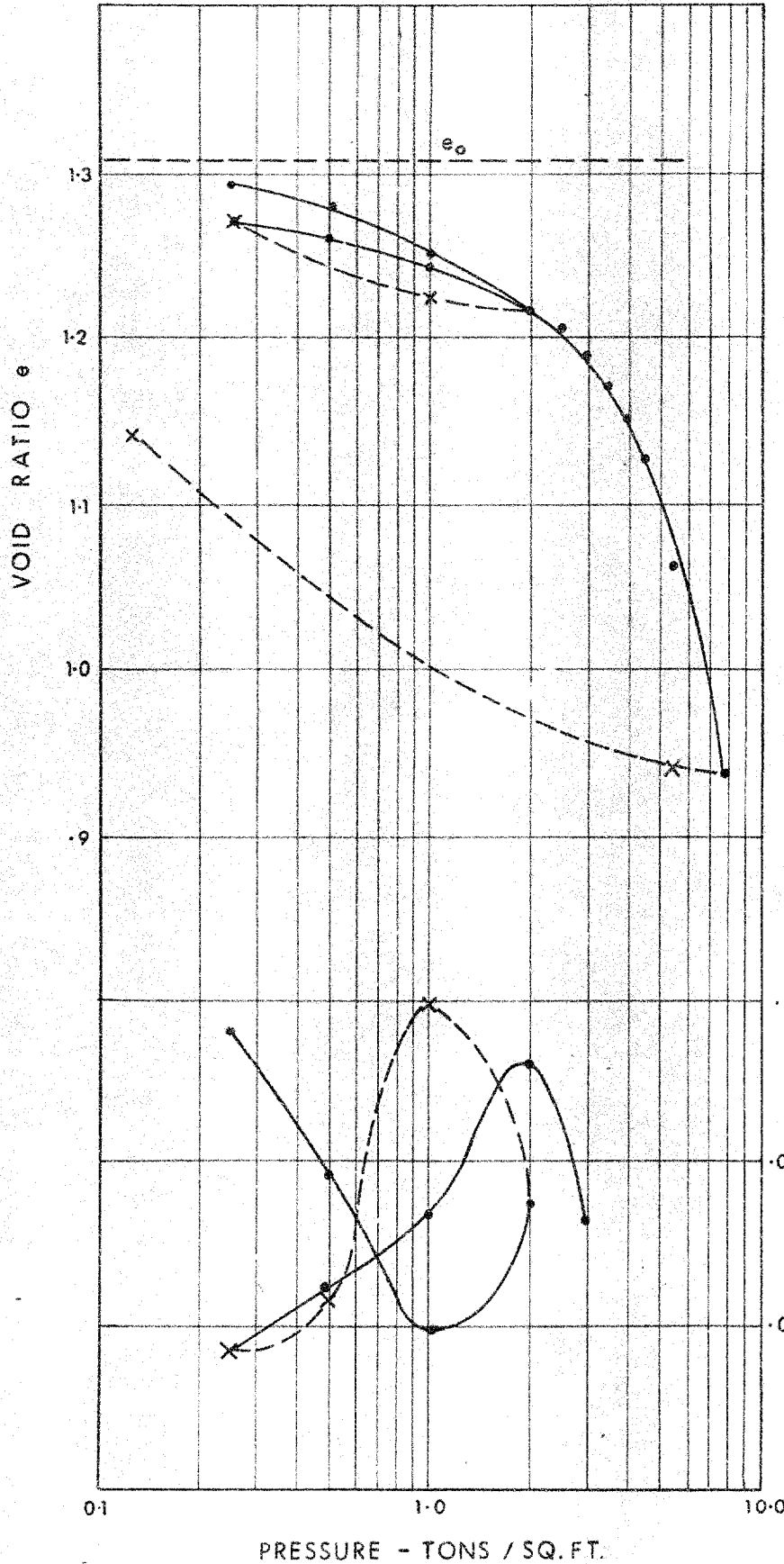
B.H.1 SAMPLE 14

DEPTH	36.0'
ELEVATION	282.0
$w_L$	44.1%
$w_P$	23.6%
$w_I$	39.1%
$P_0$	1.35 T.S.F.
$e_0$	1.05

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY $M_v$ - FT. <sup>2</sup> /TON	PERMEABILITY $K$ - INS. / MIN.
0.25	$10^{-2} \times 5.75$	$7.80 \times 10^{-6}$
0.50	$10^{-2} \times 3.91$	$1.67 \times 10^{-6}$
1.00	$10^{-2} \times 2.54$	$0.72 \times 10^{-6}$
2.00	$10^{-2} \times 1.46$	$0.67 \times 10^{-6}$

FIG. 11(b)

BROWN CLAY

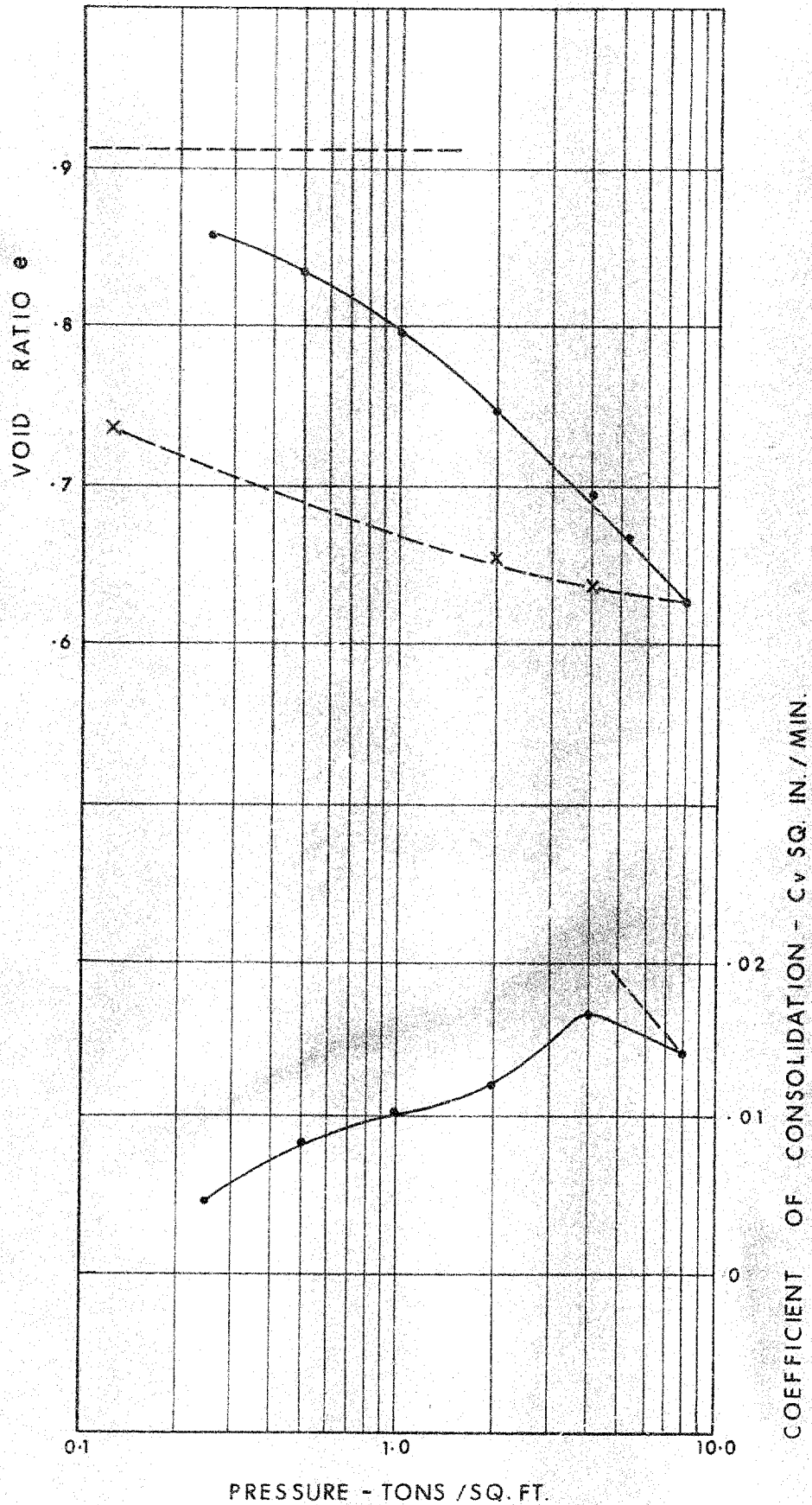


B.H. 1 SAMPLE 16	
DEPTH	39.0'
ELEVATION	279.0
WL	44.6%
WP	23.2%
WI	48.6%
PO	1.40 T.S.F.
eo	1.31

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY M <sub>v</sub> - FT. <sup>2</sup> /TON	PERMEABILITY K - INS. / MIN.
0.25	10 <sup>-2</sup> X 2.66	1.94 X 10 <sup>-6</sup>
0.50	10 <sup>-2</sup> X 2.46	1.28 X 10 <sup>-6</sup>
1.00	10 <sup>-2</sup> X 2.08	0.53 X 10 <sup>-6</sup>
2.00	10 <sup>-2</sup> X 1.35	0.61 X 10 <sup>-6</sup>

FIG. 11(c)

# GREY CLAY



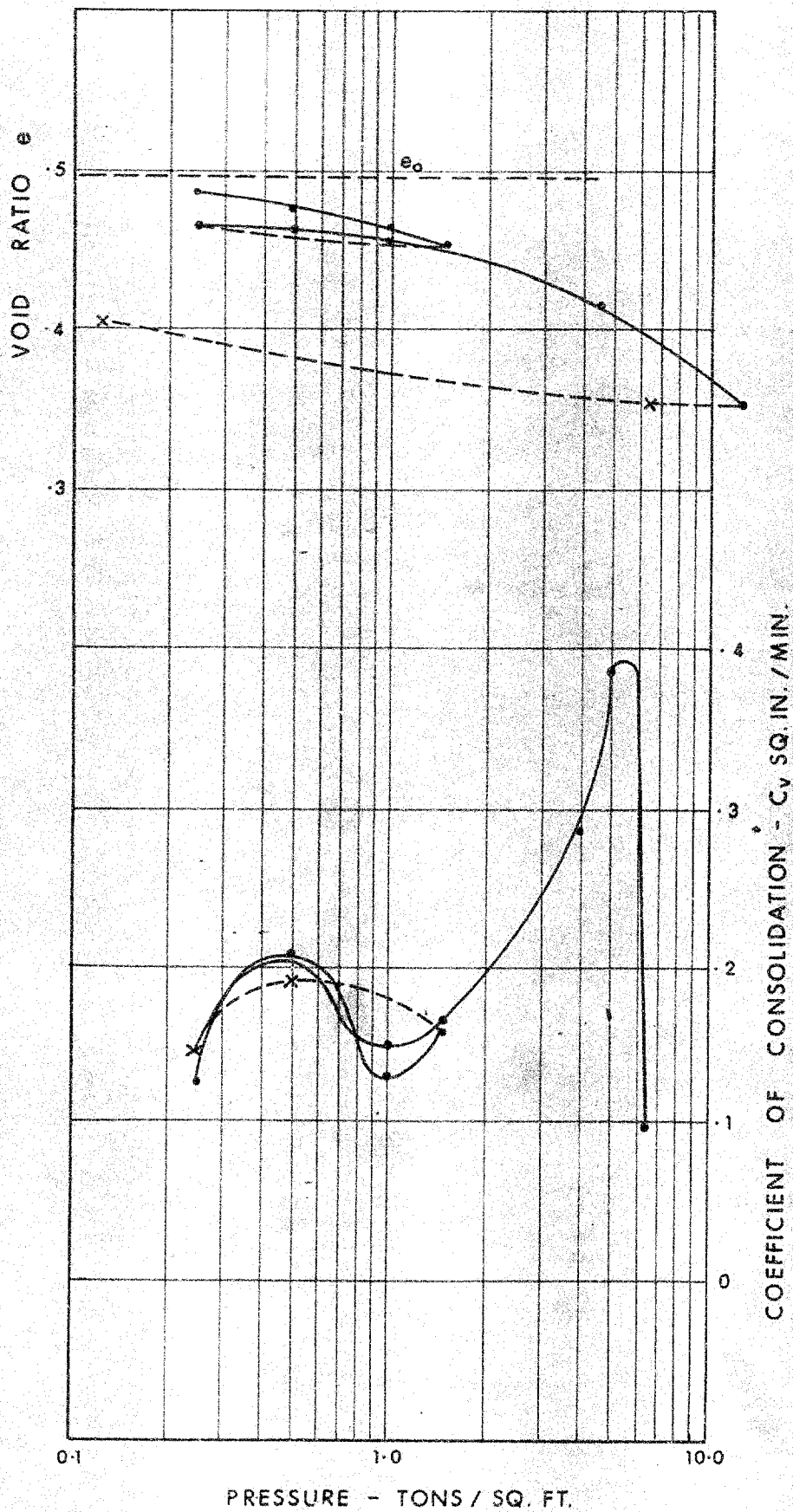
B.H. 1 SAMPLE 21

DEPTH	45.0'
ELEVATION	272.5
$W_L$	30.5 %
$W_P$	24.2 %
$W_I$	34.1 %
$P_o$	1.60 T.S.F.
$e_o$	0.91

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY $M_v$ - FT. <sup>2</sup> / TON	PERMEABILITY $K$ - INS. / MIN.
0.25	$10^{-2} \times 6.94$	$0.70 \times 10^{-6}$
0.50	$10^{-2} \times 5.12$	$1.15 \times 10^{-6}$
1.00	$10^{-2} \times 3.56$	$1.01 \times 10^{-6}$
2.00	$10^{-2} \times 2.06$	$0.63 \times 10^{-6}$

FIG. 11(d)

# UPPER TILL



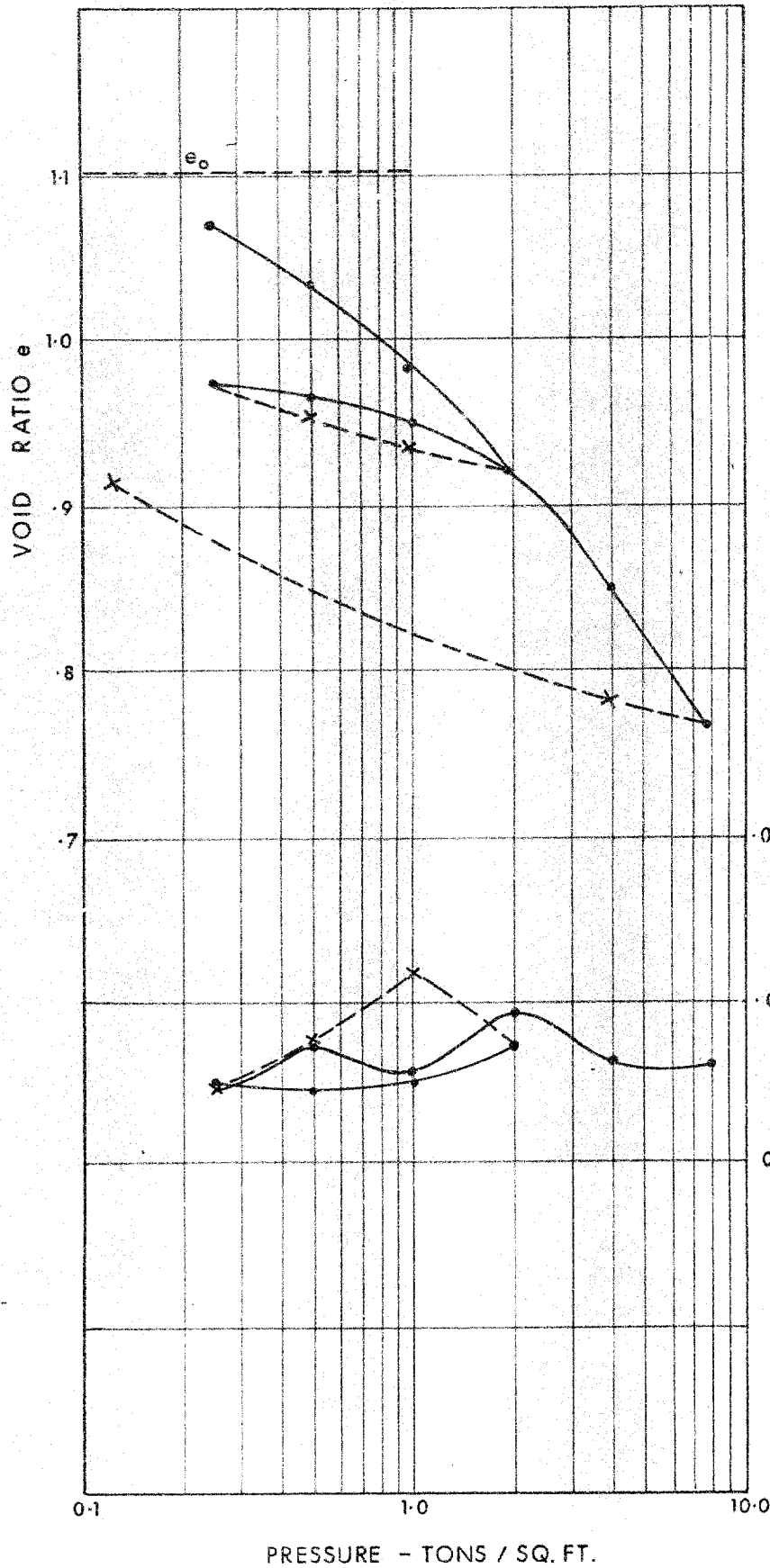
## B.H.2 SAMPLE 4C

DEPTH	30.0'
ELEVATION	288.0
WL	25.3%
WP	15.7%
WI	18.1%
Po	1.15 T.S.F.
eo	0.49

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY MV-FT <sup>2</sup> / TON	PERMEABILITY K - INS. / MIN.
0.25	10 <sup>-2</sup> X 2.63	0.84 X 10 <sup>-6</sup>
0.50	10 <sup>-2</sup> X 2.16	1.17 X 10 <sup>-6</sup>
1.00	10 <sup>-2</sup> X 1.57	0.51 X 10 <sup>-6</sup>
2.00	10 <sup>-2</sup> X 0.97	0.41 X 10 <sup>-6</sup>

FIG. 11(e)

# GREY CLAY



## B.H. 2 SAMPLE 17C

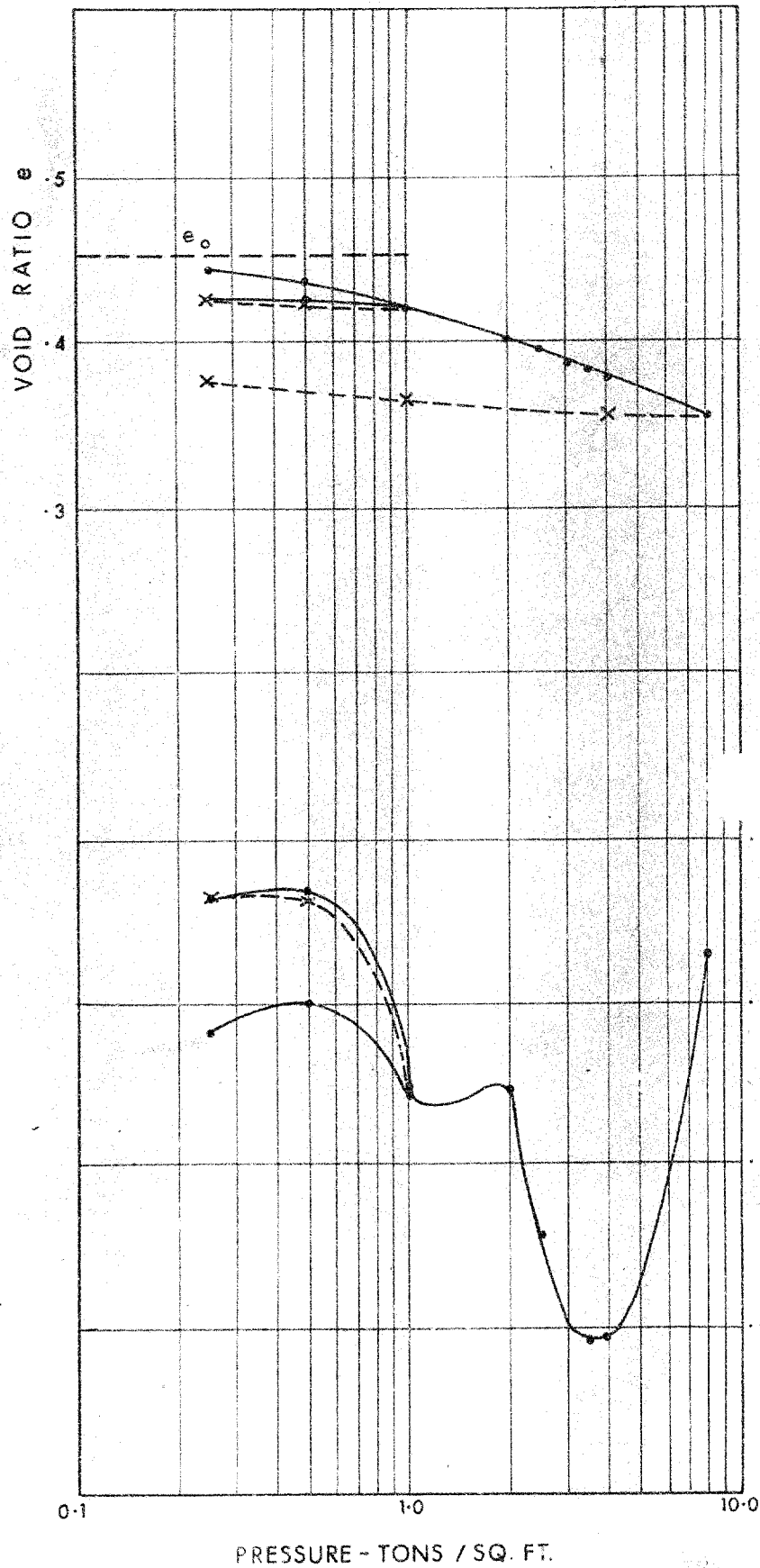
DEPTH	49.0'
ELEVATION	269.0
WL	39.9%
WP	27.5%
WI	40.5%
Po	1.71 T.S.F.
e <sub>o</sub>	1.10

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY M <sub>v</sub> - FT. <sup>2</sup> / TON	PERMEABILITY K - INS. / MIN.
0.246	10 <sup>-2</sup> X 7.25	1.03 X 10 <sup>-6</sup>
0.493	10 <sup>-2</sup> X 5.47	0.65 X 10 <sup>-6</sup>
0.986	10 <sup>-2</sup> X 3.94	0.50 X 10 <sup>-6</sup>
1.972	10 <sup>-2</sup> X 2.66	0.49 X 10 <sup>-6</sup>

FIG. 11(F)



# UPPER TILL



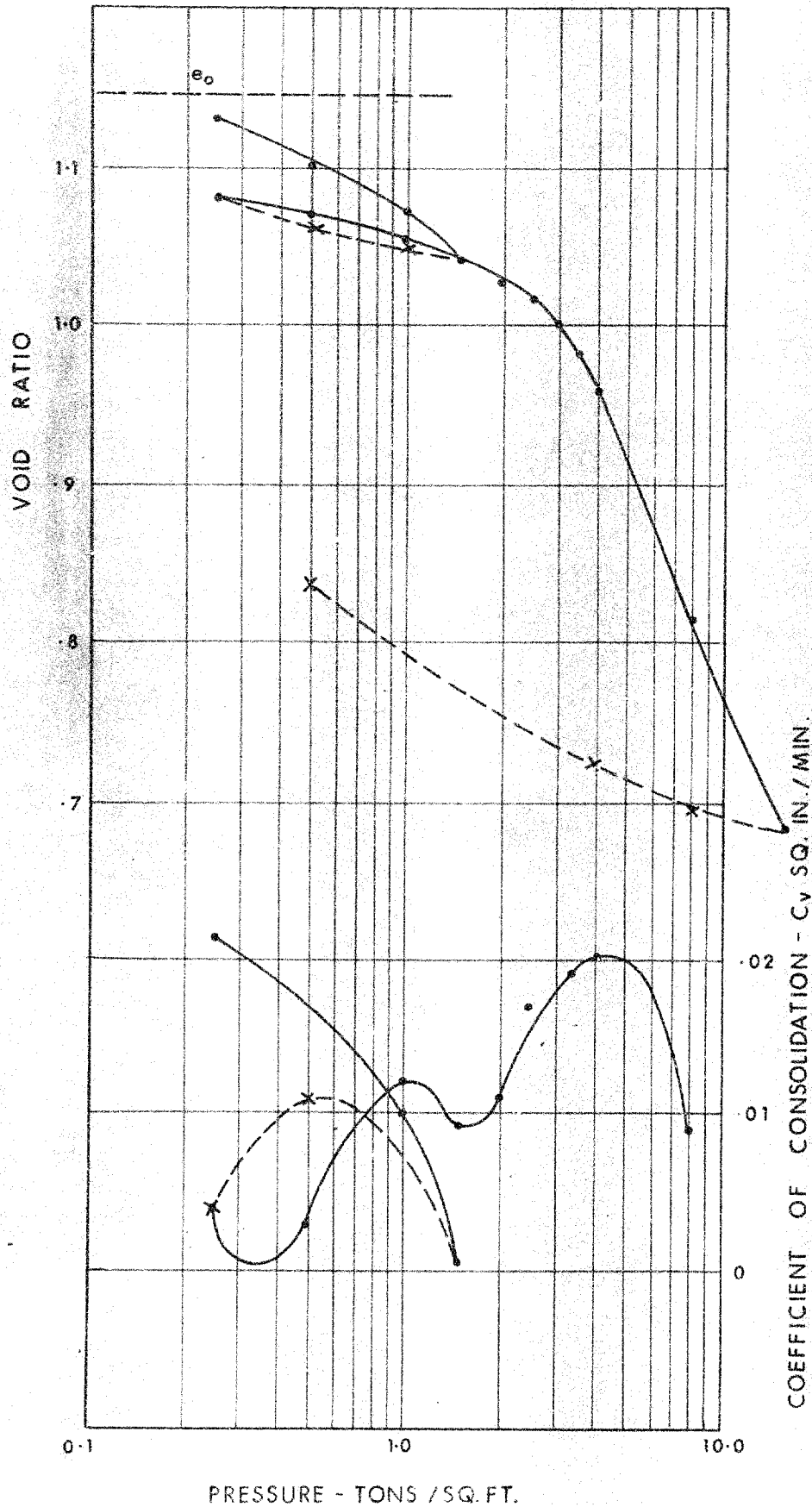
## B.H. 4 SAMPLE 11

DEPTH	24.5'
ELEVATION	293.5
WL	24.1%
WP	13.8%
WI	15.5%
P <sub>0</sub>	1.00 T.S.F.
e <sub>0</sub>	0.45

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY M <sub>v</sub> - FT. <sup>2</sup> / TON	PERMEABILITY K - INS. / MIN.
0.25	10 <sup>-2</sup> X 2.49	1.83 X 10 <sup>-6</sup>
0.50	10 <sup>-2</sup> X 2.16	1.68 X 10 <sup>-6</sup>
1.00	10 <sup>-2</sup> X 1.69	1.06 X 10 <sup>-6</sup>
2.00	10 <sup>-2</sup> X 1.07	0.69 X 10 <sup>-6</sup>

FIG. 11(g)

# GREY CLAY



B.H. 7 SAMPLE 13

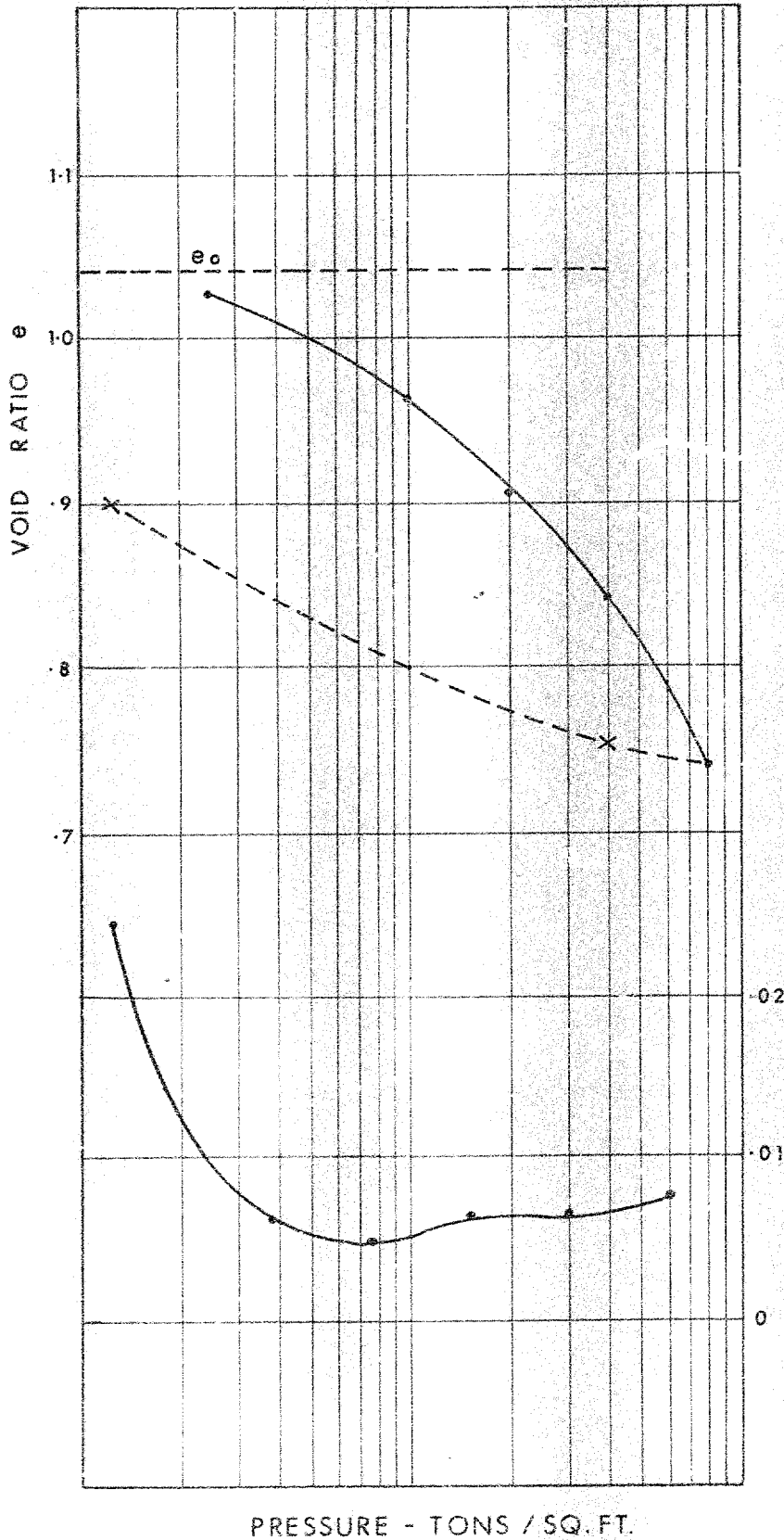
DEPTH	38.0'
ELEVATION	269.5
$w_L$	36.8%
$w_P$	16.4%
$w_I$	39.6%
$P_0$	1.13 T.S.F.
$e_0$	1.15

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIONITY $M_v$ -FT. <sup>2</sup> /TON	PERMEABILITY K - INS. / MIN.
0.247	$10^{-2} \times 7.25$	$1.03 \times 10^{-6}$
0.986	$10^{-2} \times 2.70$	$0.70 \times 10^{-6}$
1.479	$10^{-2} \times 2.16$	$0.65 \times 10^{-6}$
1.972	$10^{-2} \times 1.82$	$0.54 \times 10^{-6}$

FIG. 11(h)



# BROWN CLAY



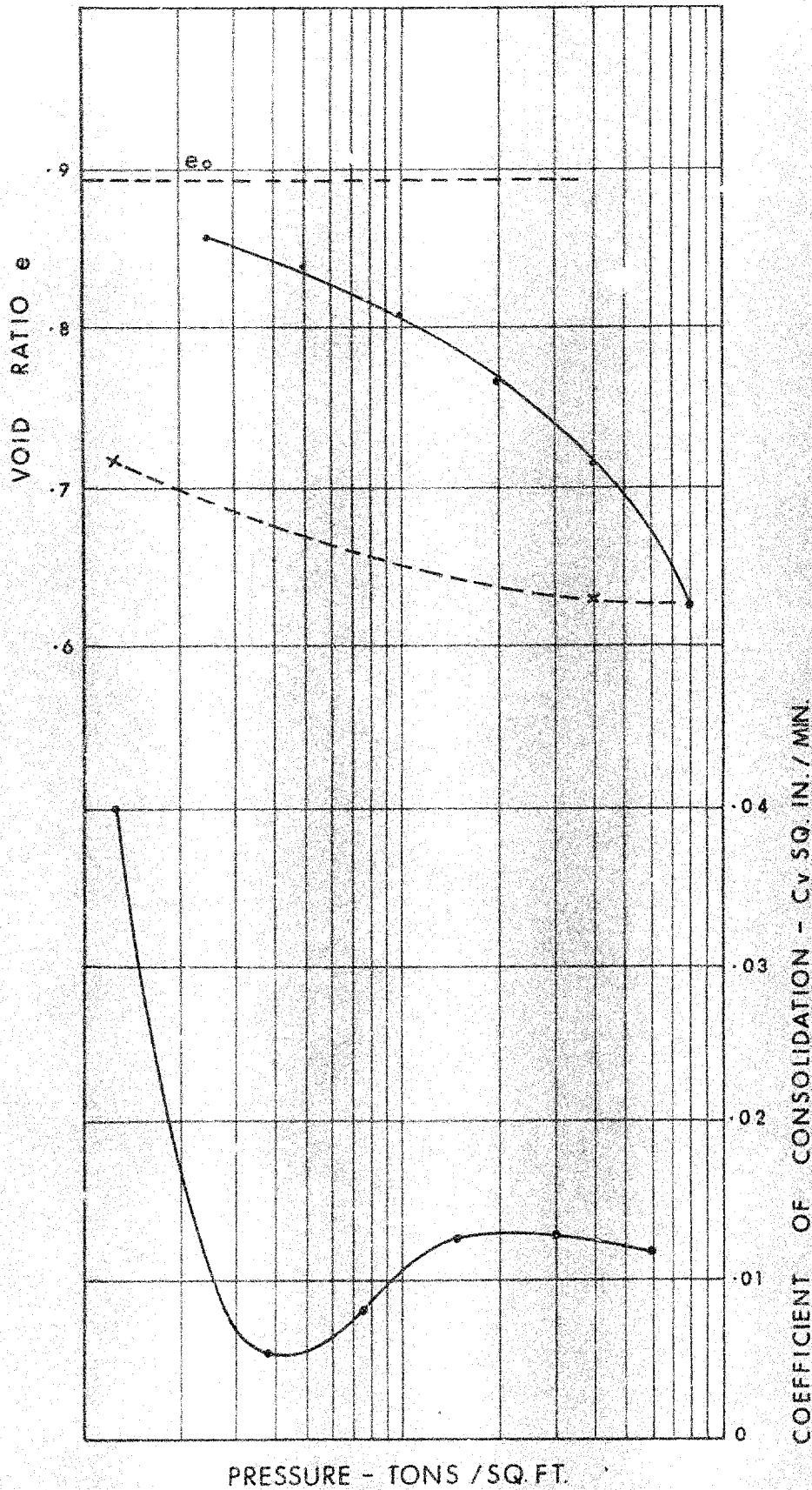
B.H.18 SAMPLE 7

DEPTH	36.0'
ELEVATION	281.2
W.L.	52.1%
W <sub>L</sub>	22.7%
V <sub>L</sub>	39.4%
P <sub>0</sub>	1.23 T.S.F.
e <sub>0</sub>	1.042

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY M <sub>V</sub> - FT. <sup>2</sup> / TON	PERMEABILITY K - INS / MIN.
0.25	10 <sup>-2</sup> X 4.44	2.84 X 10 <sup>-6</sup>
0.50	10 <sup>-2</sup> X 4.00	0.65 X 10 <sup>-6</sup>
1.00	10 <sup>-2</sup> X 3.06	0.41 X 10 <sup>-6</sup>
2.00	10 <sup>-2</sup> X 2.36	0.42 X 10 <sup>-6</sup>

FIG. 11(i)

# BROWN CLAY



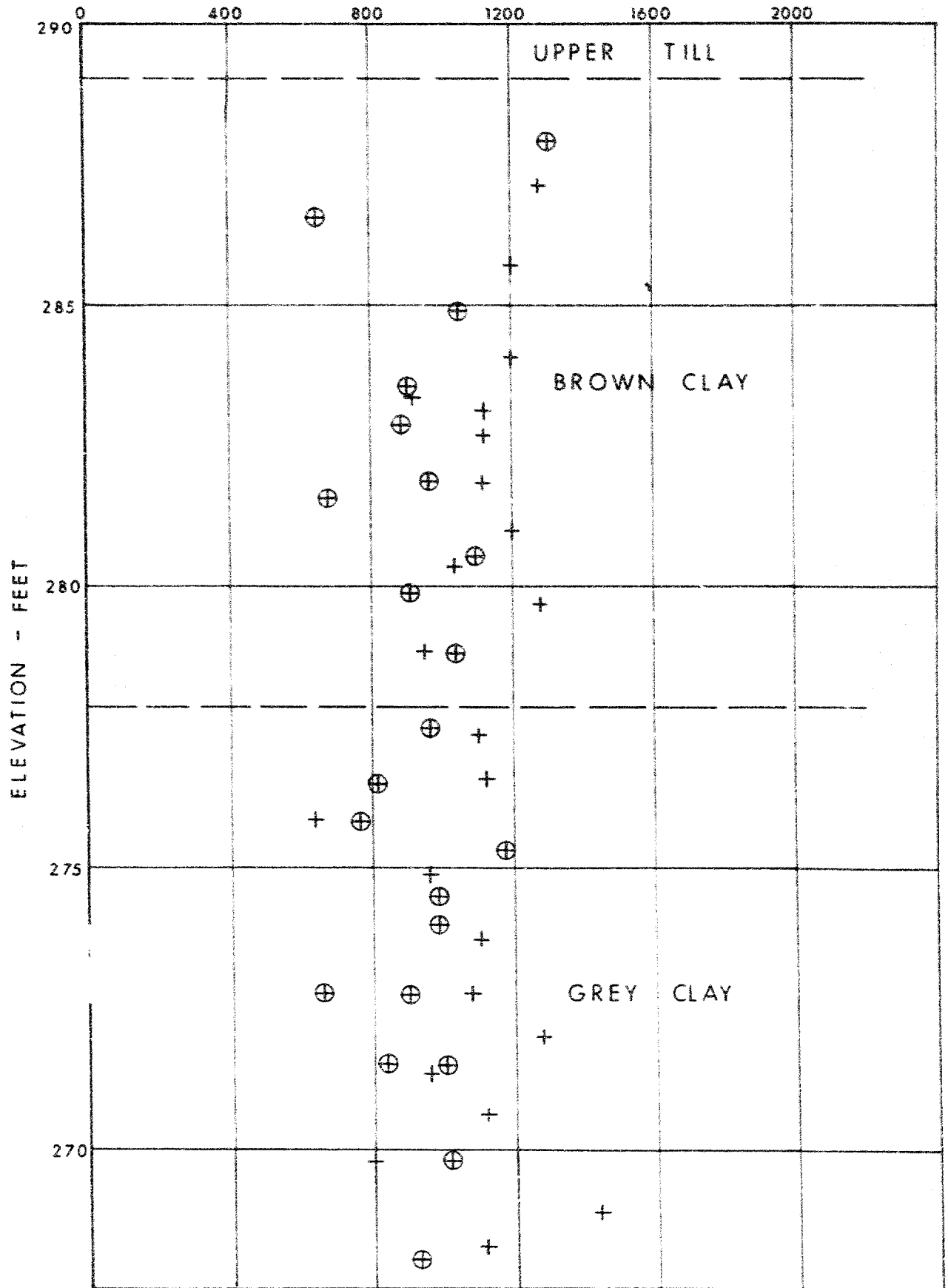
B.H.19 SAMPLE 8	
DEPTH	38.0'
ELEVATION	280.8
W <sub>L</sub>	47.5 %
W <sub>P</sub>	21.6 %
W <sub>I</sub>	33.9 %
P <sub>o</sub>	1.29 T.S.F.
e <sub>o</sub>	0.892

PRESSURE TONS/FT. <sup>2</sup>	COEFFICIENT OF COMPRESSIBILITY M <sub>v</sub> - FT. <sup>2</sup> /TON	PERMEABILITY K - INS./MIN.
0.25	10 <sup>-2</sup> X 5.66	5.89 X 10 <sup>-6</sup>
0.50	10 <sup>-2</sup> X 4.35	0.62 X 10 <sup>-6</sup>
1.00	10 <sup>-2</sup> X 2.76	0.58 X 10 <sup>-6</sup>
2.00	10 <sup>-2</sup> X 1.70	0.57 X 10 <sup>-6</sup>

FIG. 11(j)

# FIELD VANE TESTS - BOREHOLE NO. 1

SHEAR STRENGTH - P.S.F.

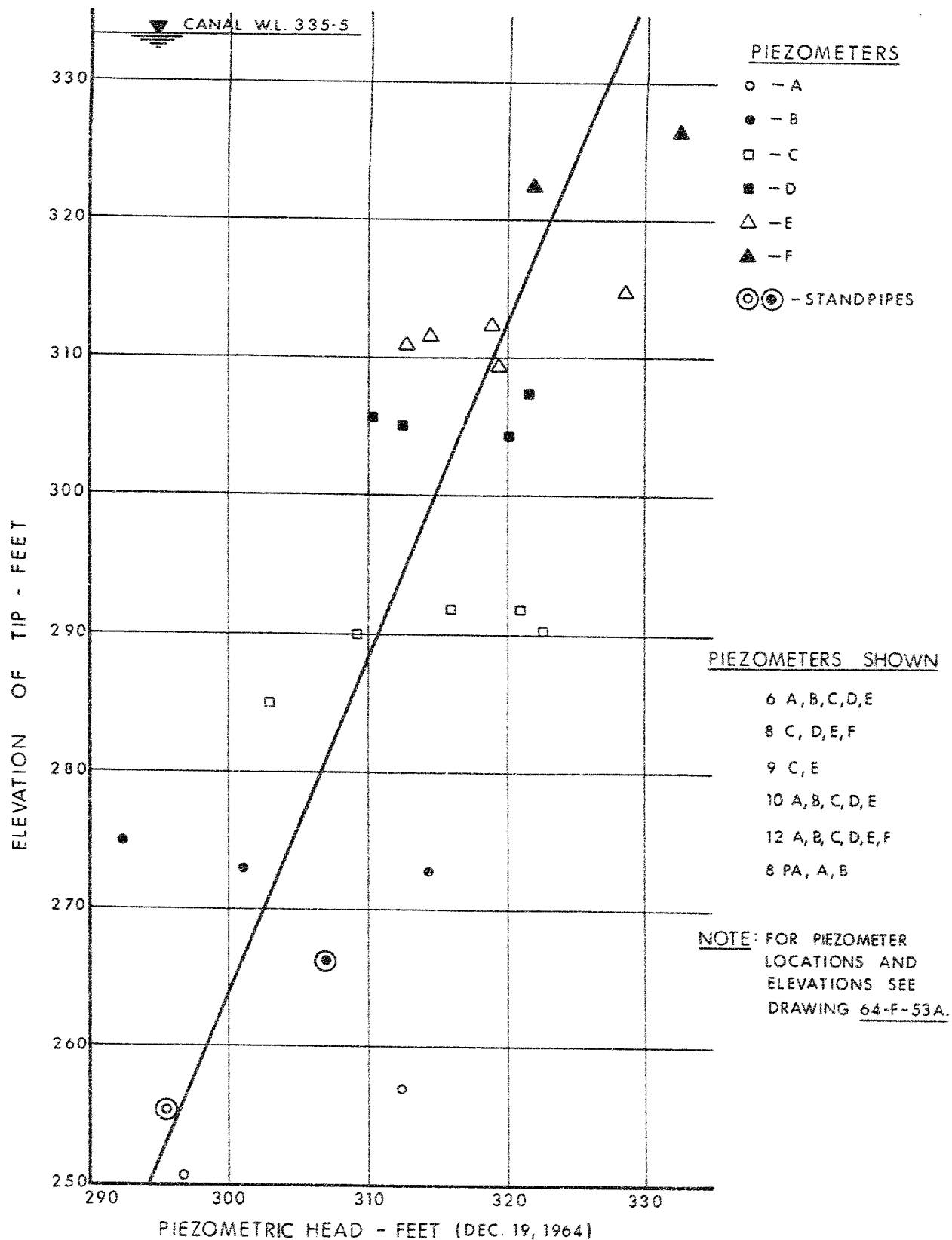


⊕ - SPRAGUE & HENWOOD VANE

+ - D.H.O. VANE

FIG. 16 (a)

JOB NO. 64-F-53



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

ONTARIO

# PIEZOMETRIC HEAD VS. TIP ELEVATION PRIOR TO CANAL DRAWDOWN

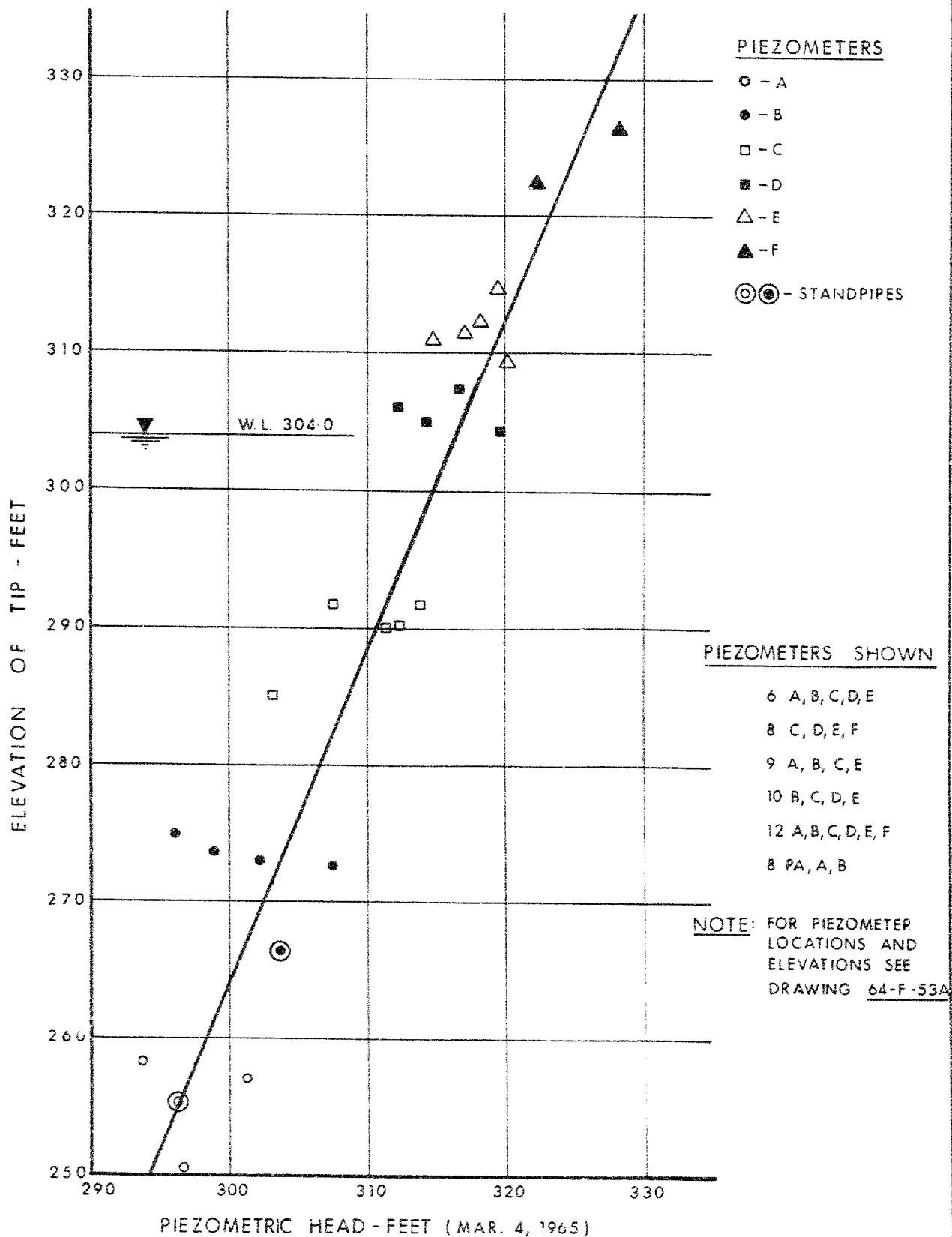
W.P. 444-64 (CARLTON ST. TUNNEL)

JOB. 64-F-53

DATE JUNE 16, 1965

APPROVED *W. C. Smith*

DRAWING NO. FIG. 17(a)



ONTARIO

DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

# PIEZOMETRIC HEAD VS. TIP ELEVATION

AFTER CANAL DRAWDOWN

W.P. 444-64 (CARLTON ST. TUNNEL)

JOB 64-F-53

DATE JUNE 16, 1965

APPROVED

*[Signature]*

DRAWING NO. FIG. 17 (b)

CC: FOUNDATIONS  
K.M. 111

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. M. Toye  
Bridge Engineer,  
Bridge Division.

FROM: Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. F. I. Hewson  
Consultant Liaison Engineer.

DATE: January 22, 1965

OUR FILE REF.

IN REPLY TO

SUBJECT:

SLOPE STABILITY  
INTERIM DESIGN REPORT  
EXCAVATIONS IN CANAL BED

Prop. Carlton St. Tunnel under the  
Welland Canal at St. Catharines, Ont.  
District 4.

W.J. 64-F-53-3 -- W.P. 444-64

Attached, please find the Interim Design Report dealing with the stability of slopes of the excavation in the Canal bed.

In the chapter "Discussion and Conclusions", the results of the various types of analyses are critically examined and commented upon. Due to reasons mentioned, different analyses produce some quite different results and therefore, an elaborate instrumentation of the present excavation operation has been initiated. It is believed that the results of this instrumentation and the observation of the performance of the various slopes will provide the necessary data for final design.

Should there be any queries with respect to this report, please do not hesitate to contact our Office.

AGS/MdeF  
Attach.

*A. G. Stermac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
H. D. McMillan  
G. K. Hunter (2)  
H. Greenland  
T. J. Kovlich  
W. Melinyshyn  
General Engineering Co. Ltd. (2)

Foundations Office ✓  
Gen. Files (1)

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  2. DESIGN CRITERIA
  3. SLOPES FOR EXCAVATION IN THE BOTTOM  
OF THE CANAL (CONTRACT I).
  4. DISCUSSION AND CONCLUSIONS.
  5. MISCELLANEOUS.
-

SLOPE STABILITY  
INTERIM DESIGN REPORT  
EXCAVATIONS IN CANAL BED

Prop. Carlton St. Tunnel under the  
Welland Canal at St. Catharines, Ont.  
District 4

W.J. 64-F-53-3    --    W.P. 444-64

1. INTRODUCTION:

The site of the proposed Carlton St. Tunnel under the Welland Canal is situated at the eastern outskirts of the City of St. Catharines, about  $1\frac{1}{2}$  miles north of the Garden City Skyway. A foundation investigation on the subsoil conditions and properties has been carried out and a report submitted on October 9, 1964. The present report deals with the design of slopes for the excavations in the canal bed.

Tentative slopes for the excavation in the channel bottom during the winter of 1964 (Contract 1) were proposed by General Engineering Co. Ltd., Consultant for the project, as shown in Fig. 1. These slopes and alternative cross-sections were analyzed, and the results of analyses are contained in this interim report. Design for the piercing of the east and west dykes, cross-sections for the existing tie-walls and approach slopes, will be presented in another report.

During the course of the design work, additional field work was carried out. The results of the field and laboratory investigation modified the soil properties only very slightly. These revised values are used in the stability analyses. However, borings put down around the west end of the excavation after the



1. INTRODUCTION: (cont'd.) ...

Canal was drawn down, revealed appreciable changes of soil boundaries, the most significant being that the lower limit of the grey clay was found to be at El. 250 rather than El. 260 as anticipated from the soil report.\* Analyses were therefore re-run for the revised soil horizons.

2. DESIGN CRITERIA:

(a) Methods of Analysis:

Excavation at the bottom of the Canal will be completed in March 1965. These slopes will be submerged during the navigation season and subjected to complete drawdown in the winter of 1965-66. The earth cofferdam has been scheduled to be constructed during the summer of 1965 and the existing dykes will be pierced in the following winter. However, drawdown will occur during construction of the tunnel units. For these slopes, therefore, both total and effective stress analyses are used. For total stress analyses, the circular arc method is employed, but sliding block analyses are also used to check the stability. For effective stress analysis, use is made of Bishop's method. As drawdown is complete in the course of a few days and the material is quite impervious, instantaneous drawdown is assumed for the purpose of analysis.

\*

It was impossible to put down borings in this area during the navigation season when the Canal was filled.

cont'd. /3 ...

2. DESIGN CRITERIA: (cont'd.) ...

(b) Soil Properties:

The soil parameters used in the analyses are based on test results contained in the Soils Report (W.J. 64-F-53) prepared by this Section, supplemented by the addendum of the Report - (W.J. 64-F-53-2).

(i) Total Stress Analysis:

<u>Soil Type</u>	<u>Undrained Shear Strength</u> (p.s.f.)	<u>Bulk Density</u> (p.c.f.)	<u>Submerged Density</u> (p.c.f.)
Fill	1500	128	66
Till	1500	135	73
Brown Clay	900	115	53
Grey Clay	700	115	53

(ii) Effective Stress Analysis:

<u>Soil Type</u>	<u>c'</u>	<u><math>\phi'</math></u>
Fill	120	23°
Till	100	27°
Brown Clay	200	20°
Grey Clay	200	20°

In all cases,  $\bar{B}$  is assumed to be unity.

cont'd. /4 ...

3. SLOPES FOR EXCAVATION IN THE BOTTOM OF THE CANAL -  
(CONTRACT I):

The original proposed slopes for excavation in the bottom of the Canal are shown in Fig. 1. The 1/3:1 slope on the south side is intended to be a test section and will be cut back to the same section as the 1:1, 15-ft. berm, 1:1 slope on the north side.

(a) South Slope:

The result of stability analysis for the south slope is shown in Fig. 2. The factor of safety from total stress analysis with the preliminary soil boundaries is 1.21. With the revised boundaries, the factor of safety is reduced to 1.00, Fig. 3.

From results of analyses on other slopes, it is obvious that the factor of safety in terms of effective stresses, is far below unity. No effective stress analysis is therefore carried out for this section.

(b) North Slope:

Total Stress Analysis:

The results of total stress analysis using cylindrical slip surfaces are shown in Fig. 4, the factor of safety given by this method being 1.50. Sliding block analysis using active and passive earth pressures, was also carried out and the factor of safety is 1.02. It may be noted that the bottom of the slope is assumed to be El. 280 according to the contract drawing. The deepest point of the cut is, however, at El. 275. With revised soil boundaries and bottom elevation of slope at El. 275, the factors of safety are 1.15 and 0.68, respectively, for circular and block analysis, Fig. 5.

cont'd. /5 ...

3. SLOPES FOR EXCAVATION IN THE BOTTOM OF THE CANAL -  
(CONTRACT I):

(b) North Slope: (cont'd.) ...

Effective Stress Analysis:

Fig. 6 shows the results of effective stress analysis. The factor of safety of the entire slope is 0.80 with the preliminary soil boundaries and of the partial slopes 0.56 and 0.80 for the upper and lower portion, respectively. It may be noted, however, that the critical circles of the partial slopes are very shallow and these factors of safety are therefore, on the conservative side. With the revised soil boundaries, the factors of safety for the entire slope are 0.67 (Fig. 7) and 0.64, respectively, for circular and block analysis.

(c) Slope Through West Dyke and Excavation:

With the preliminary soil boundaries, the factors of safety in terms of total and effective stress analysis, are 1.29 and 1.33, respectively (Fig. 8 and 9). The effect of the revised soil boundaries is to reduce these values to 1.05 and 1.27, (Fig. 10 and 11) while block analysis yields factors of safety of 0.66 (Fig. 18) and 1.24 (Fig. 19), respectively, for short and long-term conditions. Analyses carried out on other sections show that the partial slopes are also stable.

cont'd. /6 ...

3. SLOPES FOR EXCAVATION IN THE BOTTOM OF THE CANAL -  
(CONTRACT I): (cont'd.) ...

(d) Slope Through East Dyke and Excavation:

The factors of safety from circular method are 1.16 and 1.26, respectively, in terms of total and effective stress analysis, Fig. 12 and 13. Block analysis yields values of 0.88 and 1.28, Fig. 18 and 19.

(e) Proposed North-South Slope:

In view of the low factors of safety obtained for the north and south slopes, an alternative design for the north-south slope is proposed. This consists of a 2:1 slope, 30-ft. berm and another 2:1 slope, as shown in Fig's. 14 - 17. The factors of safety are summarized in Table 1. It may be noted that the factors of safety are adequate, both for the entire and partial slopes in terms of total and effective stresses. Results for block analysis are shown also in Fig. 18 and 19.

4. DISCUSSION AND CONCLUSIONS:

The results of stability analysis using cylindrical slip surfaces for the various sections are shown in Fig. 2 to 17, while those from block analysis are shown in Fig. 18 and 19, together with an illustration of the method used. All results are summarized in Table 1.

A comparison of the values of safety factors in Table 1 obtained by circular and earth pressure methods, indicates that the results agree well in the case of effective stress analysis. However, in total stress analysis, the factors of safety from block

cont'd. /7 ...

4. DISCUSSION AND CONCLUSIONS: (cont'd.) ...

analysis are much lower, in some cases by as much as 40%. While this method is applicable in some situations in which a definite thin layer of weak soil exists, it is considered the mechanism of failure assumed in the analysis is not strictly realistic in the present case. Therefore, emphasis should be placed on the factor of safety obtained by the circular method.

The factors of safety for the north and south slopes of the excavation are low, especially in terms of effective stress. However, because of the difficulty in assessing the effective cohesion  $C'$  applicable to the field condition, the degree of accuracy of the factor of safety obtained is not certain. A full-scale field test has been proposed in a meeting with the Consultant, dated December 17, 1964. From the performance of the slopes during and subsequent to the field experiment, the most economical design may be evolved. In case such a test cannot be carried out due to circumstances beyond control, the proposed section with 2:1 slope, 30-ft. berm and 2:1 slope should be adopted.

For the east and west end slopes, the factors of safety in terms of effective stresses, are satisfactory. The safety factors in terms of total stresses, however, are only slightly above unity. To increase the factor of safety, the lowest berm may be extended by an addition of 30 ft. Alternatively, it is proposed that the present design be accepted since instruments for construction control have been installed and these measures will reveal the performance of these slopes. In the event that significant movements do occur, they may be arrested by flooding the trench.

5. MISCELLANEOUS:

Stability analyses were performed both by an Electronic Computer (IBM 7040) operated by the Electric Computing Branch and by manual computations. The co-ordination with the Computer Branch was undertaken by Mr. H. T. Chan and computations performed by Mr. W. W. Kulmatickas, Mr. P. McGlone, Project Foundation Engineers, and other members of the Foundation Section. Mr. K. Sinclair of General Engineering Co. Ltd. also participated in the initial phase of the work. The work was carried out under the supervision of Mr. K. Y. Lo, Supervising Foundation Engineer, who also prepared this report.

January 1965

# SUMMARY OF STABILITY ANALYSIS OF SLOPES IN EXCAVATION

Slope	Soil Boundary	Circular Method		Earth Pressure Method		Remarks
		Total Stress	Effective Stress	Total Stress	Effective Stress	
South Slope (1/3:1)	Preliminary	1.21	-	-	-	Toe at El. 280.
	Revised	1.00	-	-	-	
North Slope	Preliminary	1.50	0.80	1.02	-	Toe at El. 280.
	Revised	1.15	0.67	0.68	0.64	
Proposed Slope	Preliminary	1.71	1.16	-	-	
	Revised	1.29	1.09	0.94	1.18	
West Slope	Preliminary	1.29	1.33	-	-	
	Revised	1.05	1.27	0.66	1.24	
East Slope		1.16	1.26	0.88	1.28	

Note: F.S. for partial slopes are shown in individual drawings.



APPENDIX I.

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - 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 COMESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS 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
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

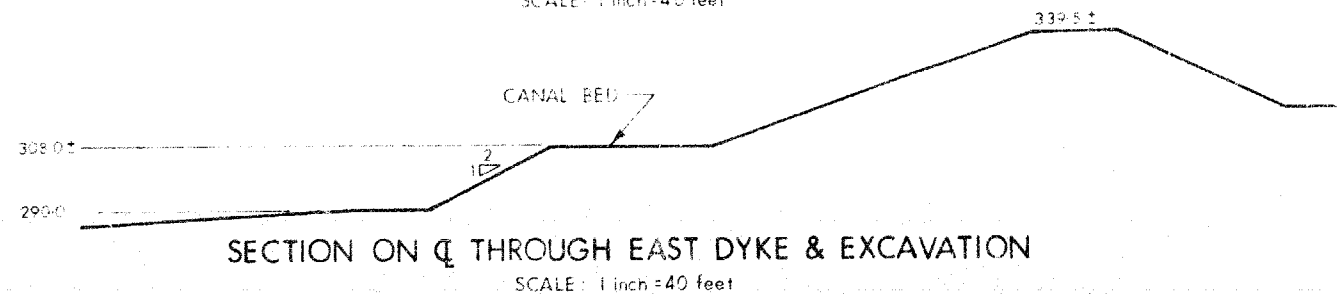
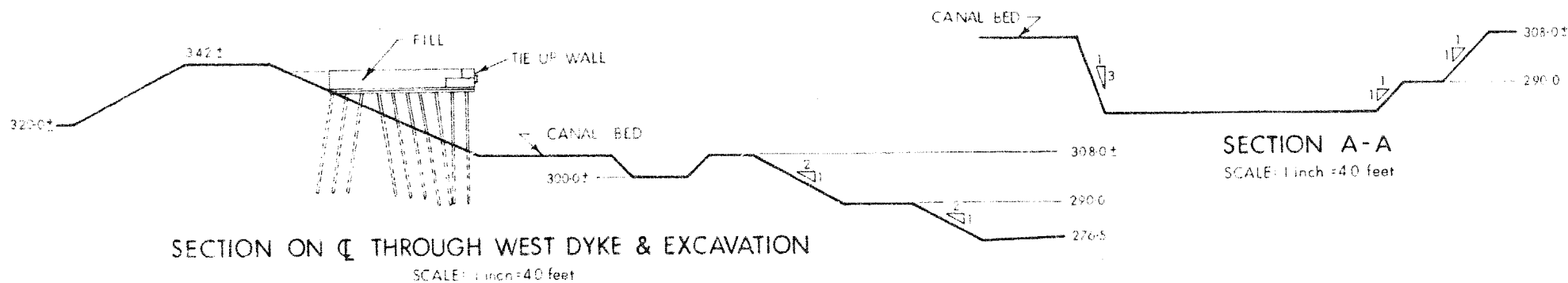
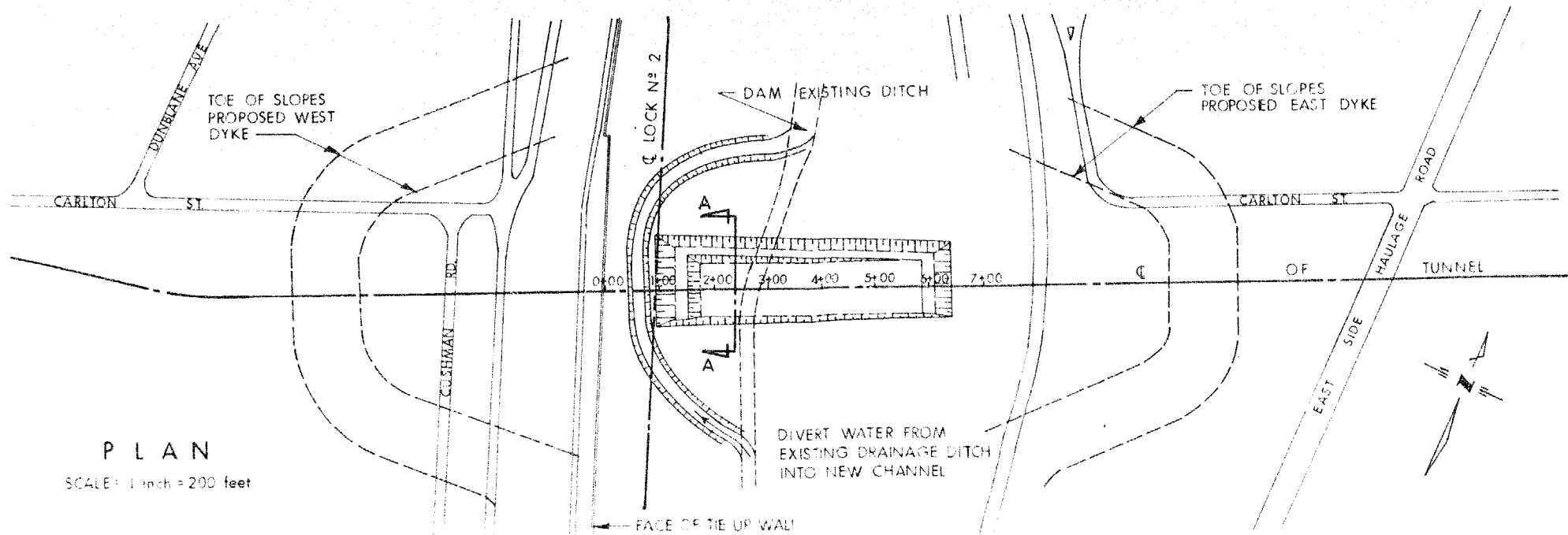
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### 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



# SOUTH SLOPE OF EXCAVATION

## TOTAL STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET

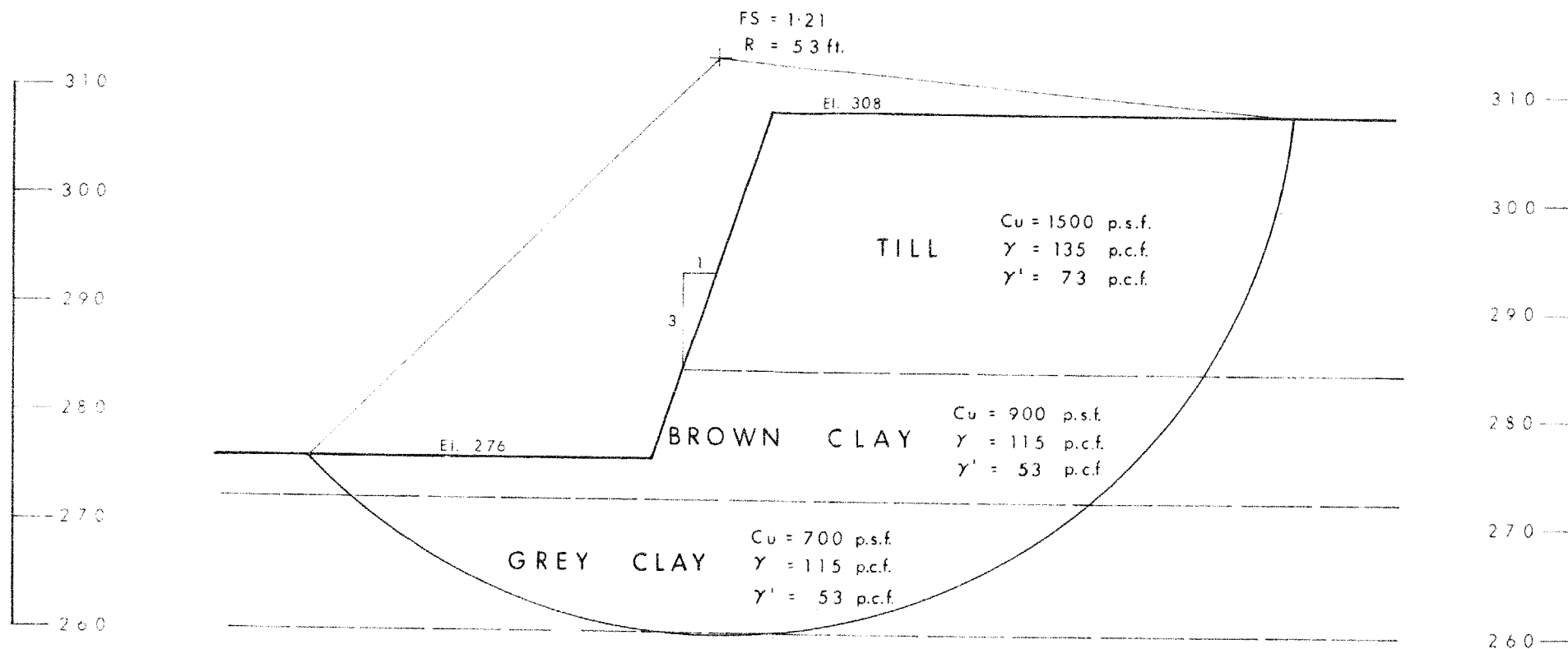
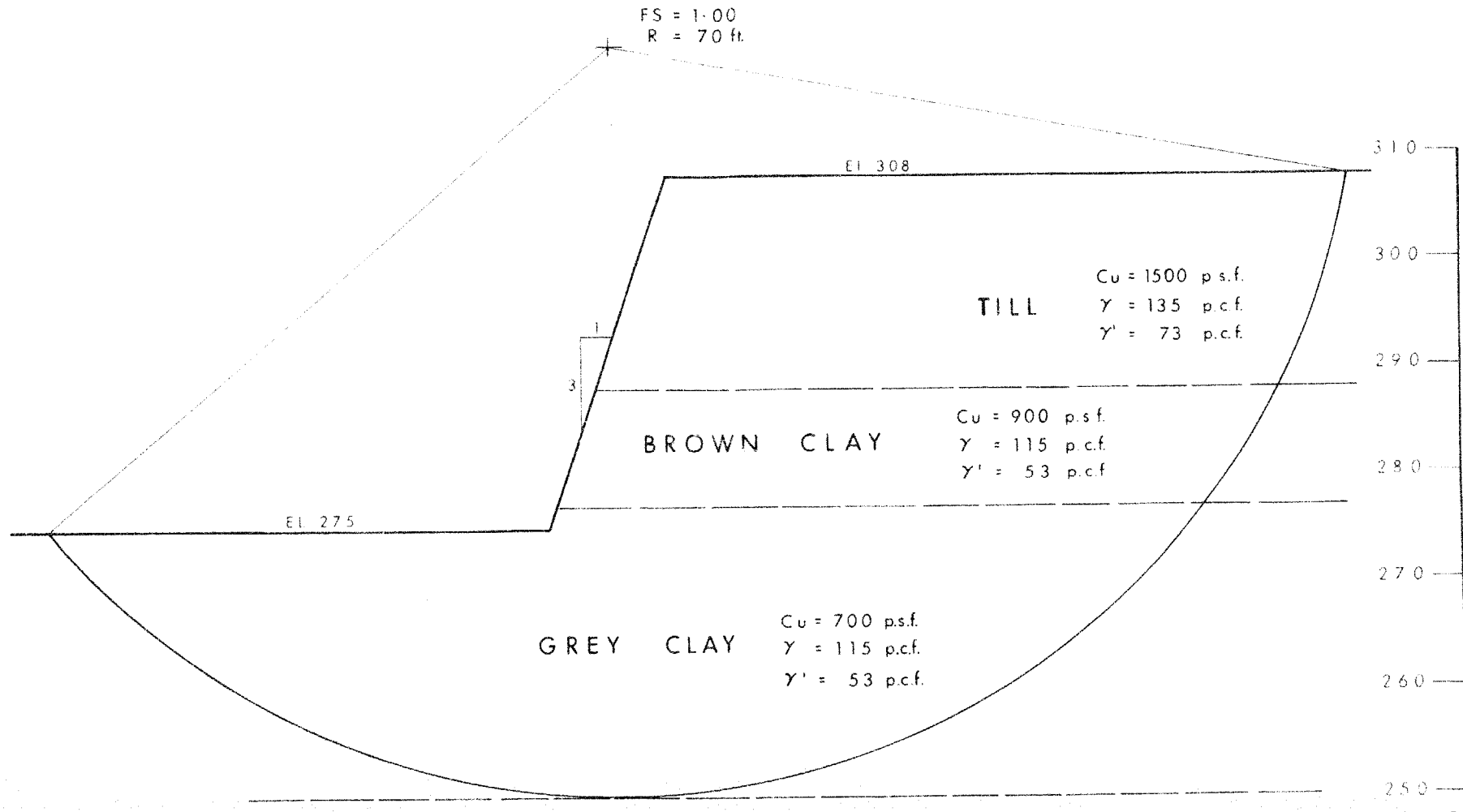


FIG. 2

# SOUTH SLOPE OF EXCAVATION

## TOTAL STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET



64-F-53-3

# NORTH SLOPE OF EXCAVATION

## TOTAL STRESS ANALYSIS

SCALE : 1 INCH = 10 FEET

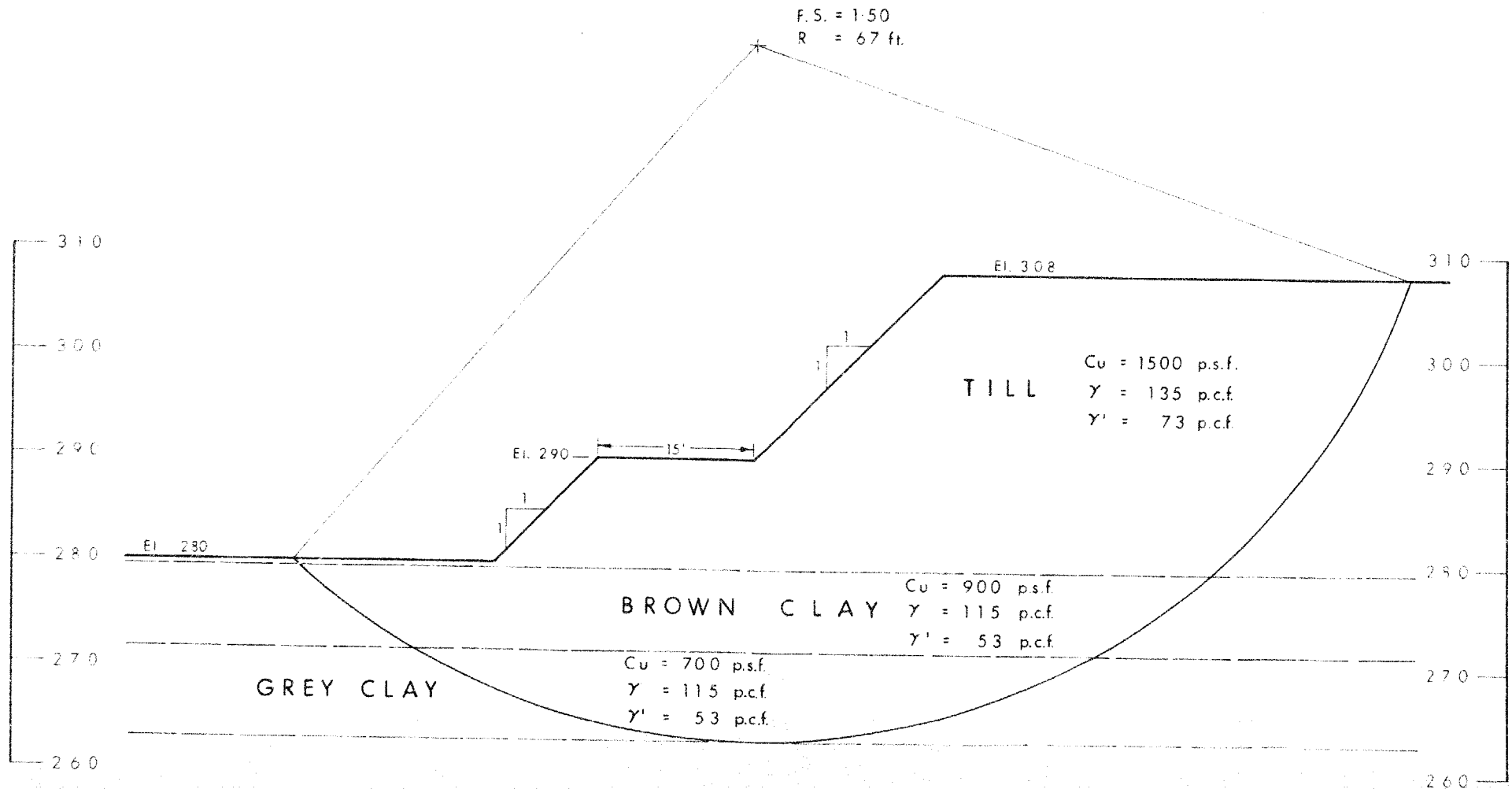


FIG. 4

# NORTH SLOPE OF EXCAVATION

## TOTAL STRESS ANALYSIS

SCALE : 1 INCH = 10 FEET

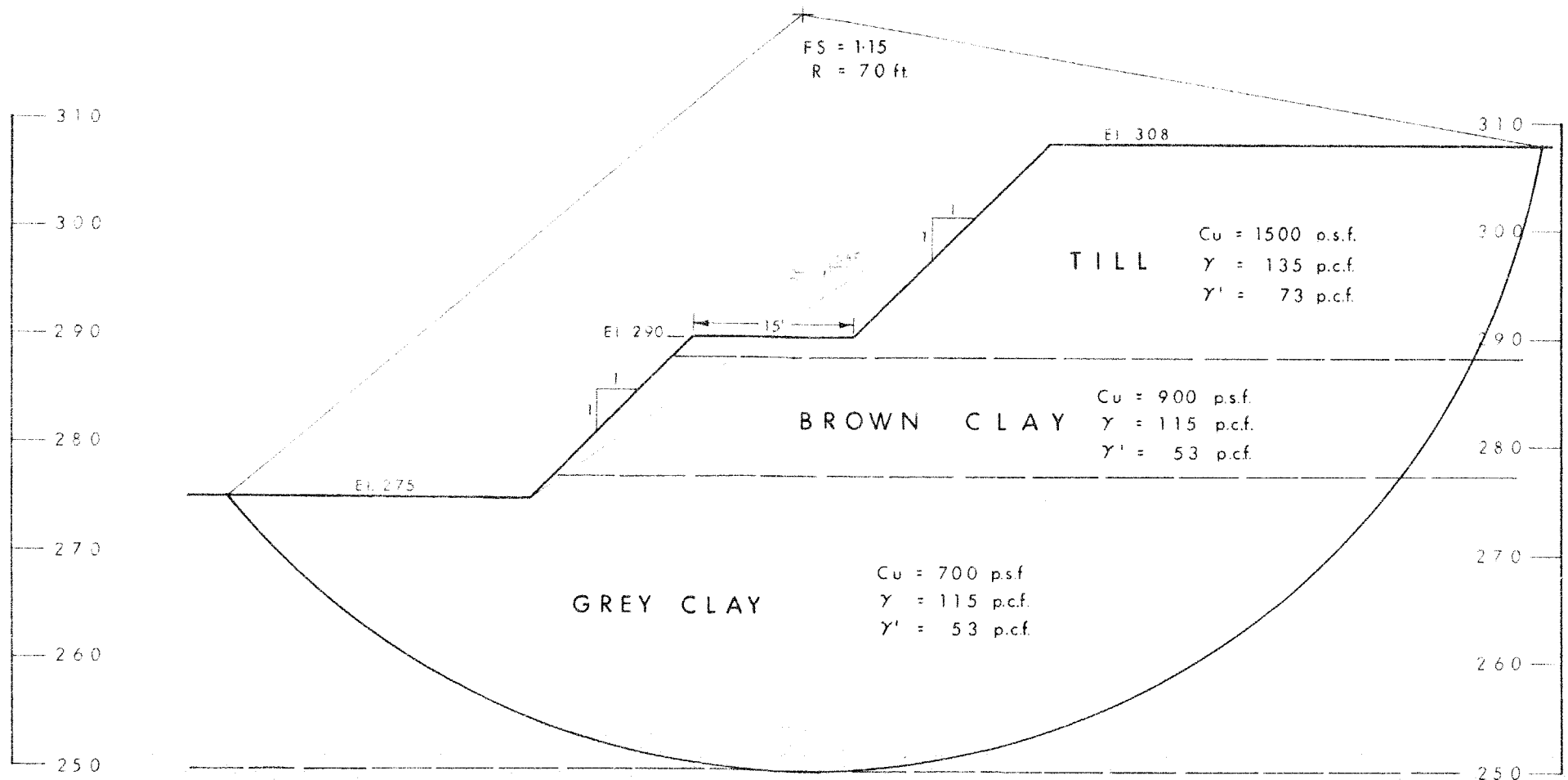


FIG. 5

64-F-53-3



# NORTH SLOPE OF EXCAVATION EFFECTIVE STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET

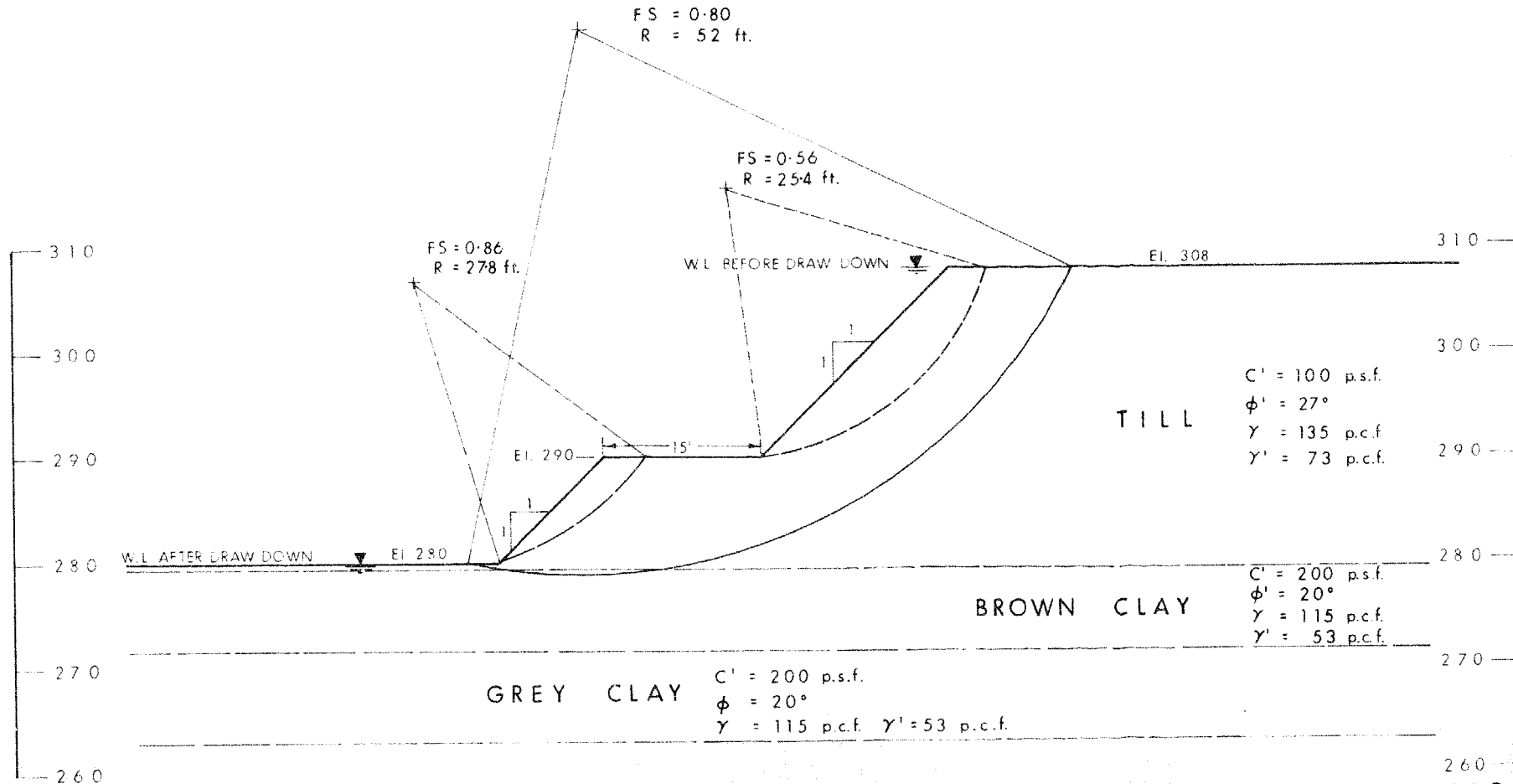


FIG. 6

# NORTH SLOPE OF EXCAVATION

## EFFECTIVE STRESS ANALYSIS

SCALE : 1 INCH = 10 FEET

FS. = 0.67  
R = 65 ft.

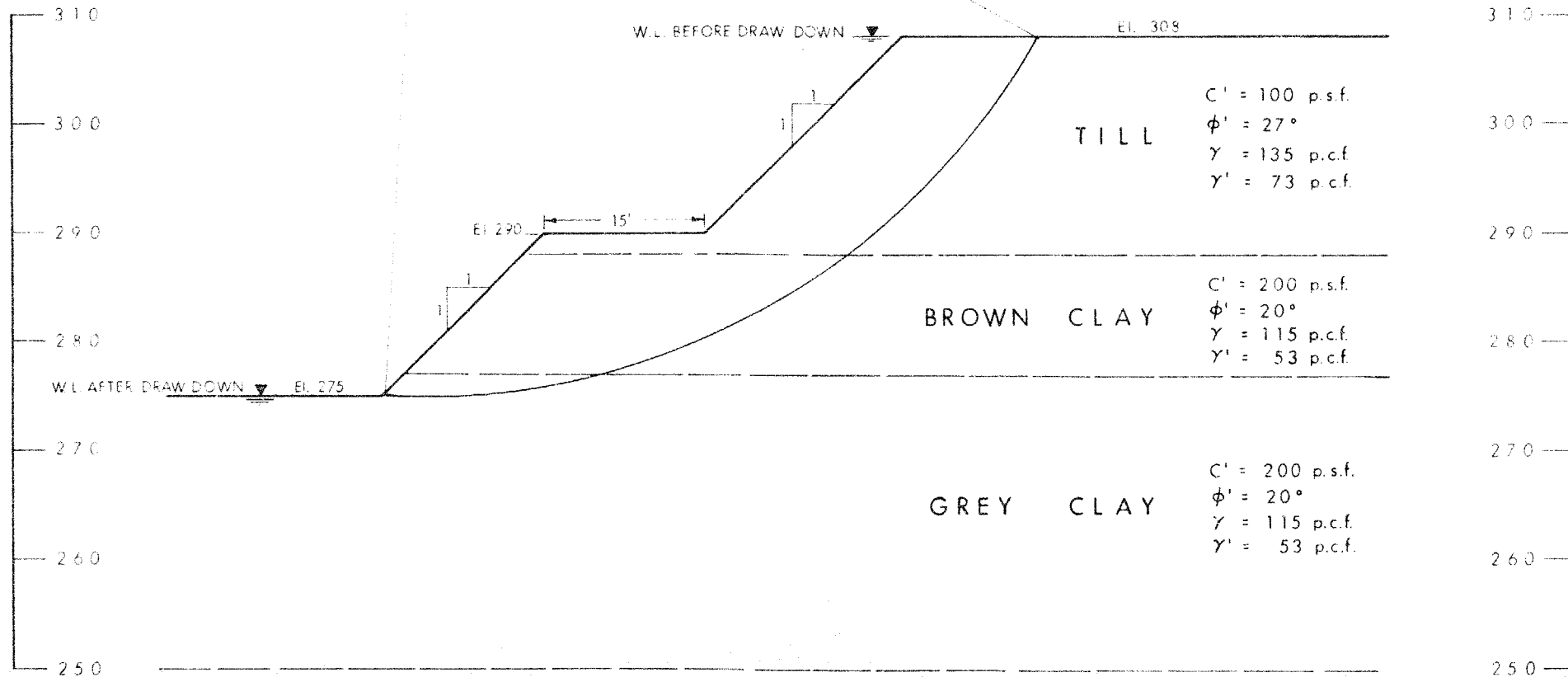


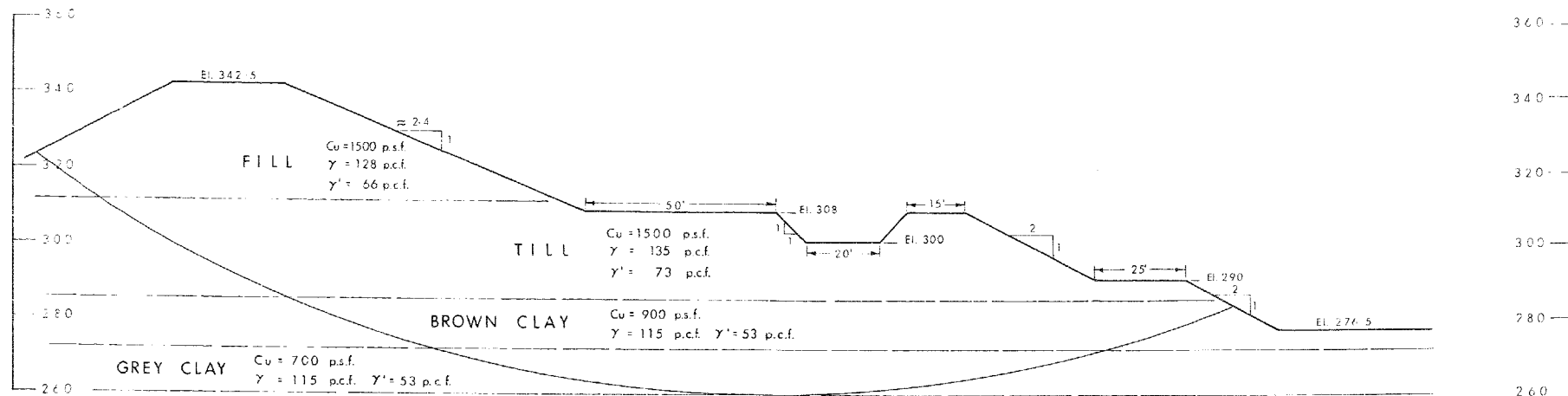
FIG. 7

# SECTION THROUGH WEST DYKE AND EXCAVATION

## TOTAL STRESS ANALYSIS

FS = 1.29  
R = 342 ft.

SCALE: 1 INCH = 20 FEET



# SECTION THROUGH WEST DYKE AND EXCAVATION

## EFFECTIVE STRESS ANALYSIS

FS = 1.33

R = 362 ft.

SCALE : 1 INCH = 20 FEET

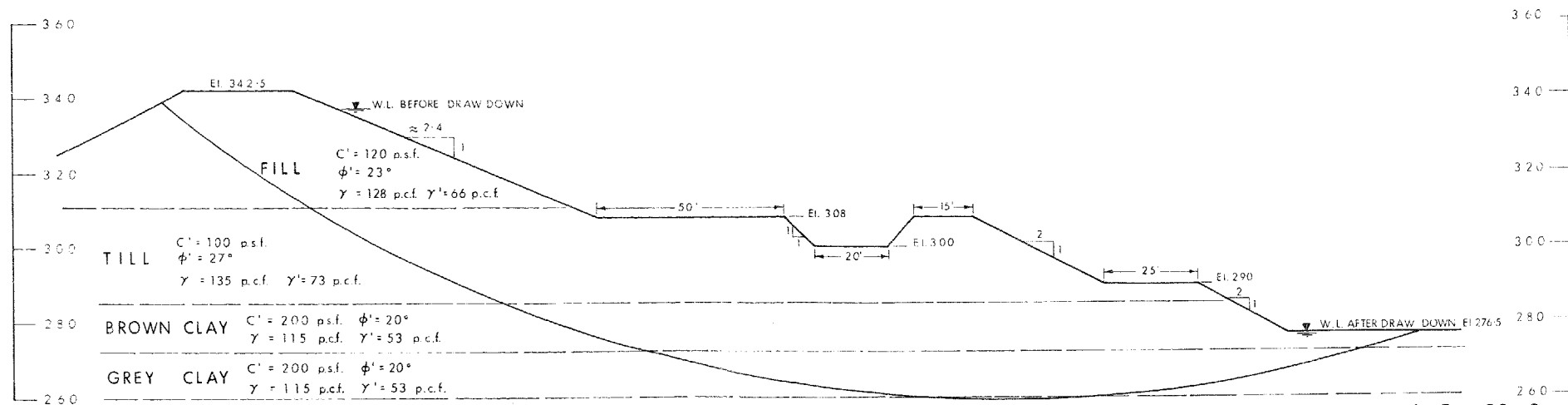


FIG. 9

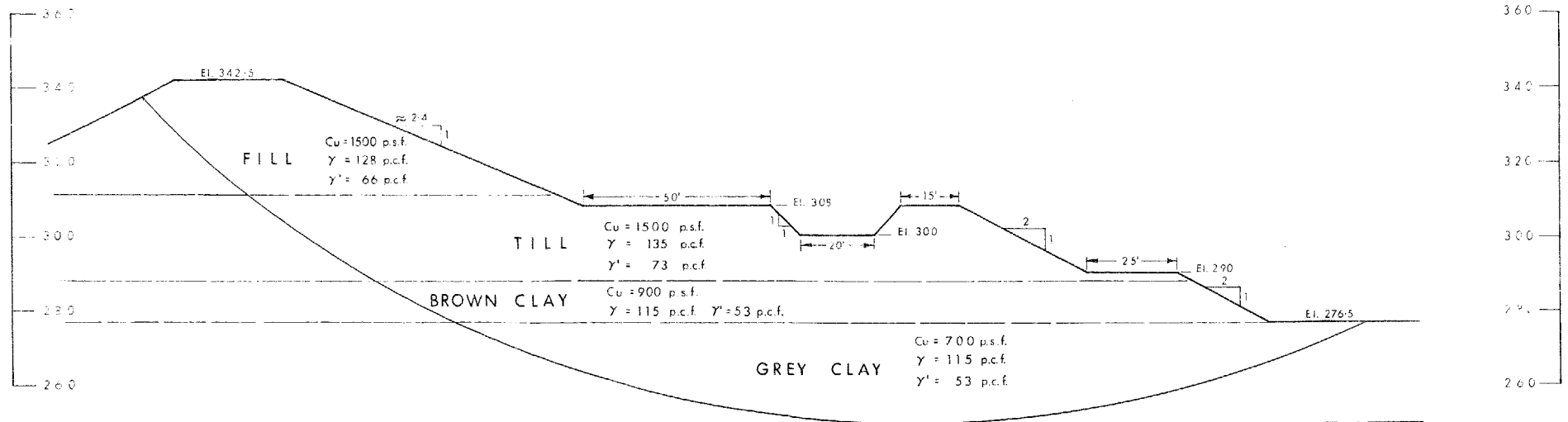
# SECTION THROUGH WEST DYKE AND EXCAVATION

## TOTAL STRESS ANALYSIS

FS = 1.05

R = 292 ft.

SCALE: 1 INCH = 20 FEET

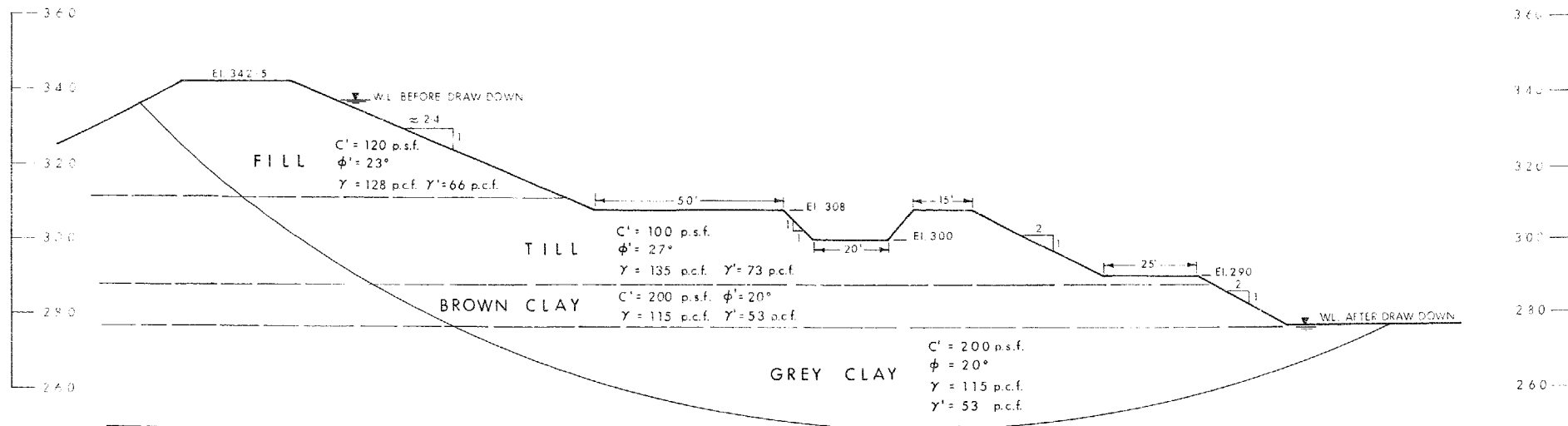


# SECTION THROUGH WEST DYKE AND EXCAVATION

## EFFECTIVE STRESS ANALYSIS

FS = 1.27  
R = 300 ft.

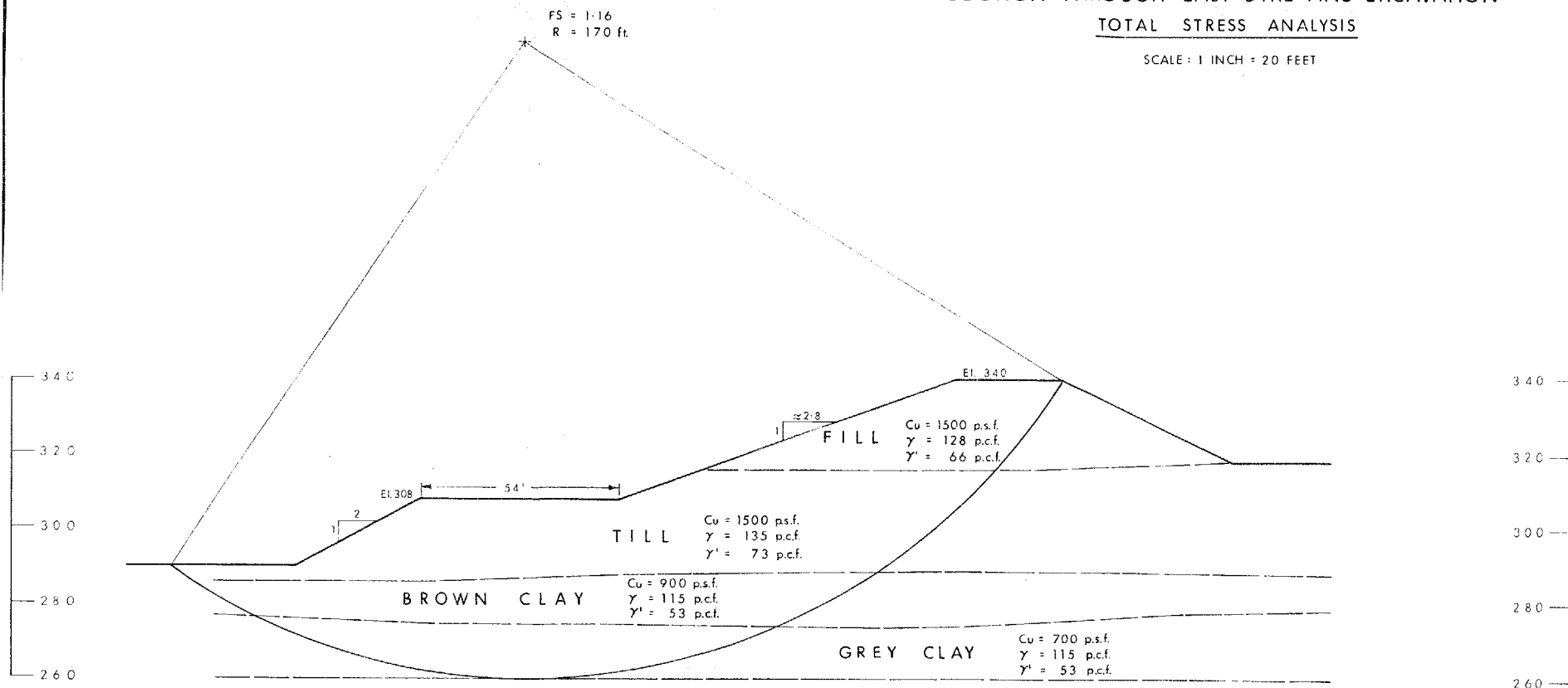
SCALE : 1 INCH = 20 FEET



# SECTION THROUGH EAST DYKE AND EXCAVATION

## TOTAL STRESS ANALYSIS

SCALE: 1 INCH = 20 FEET

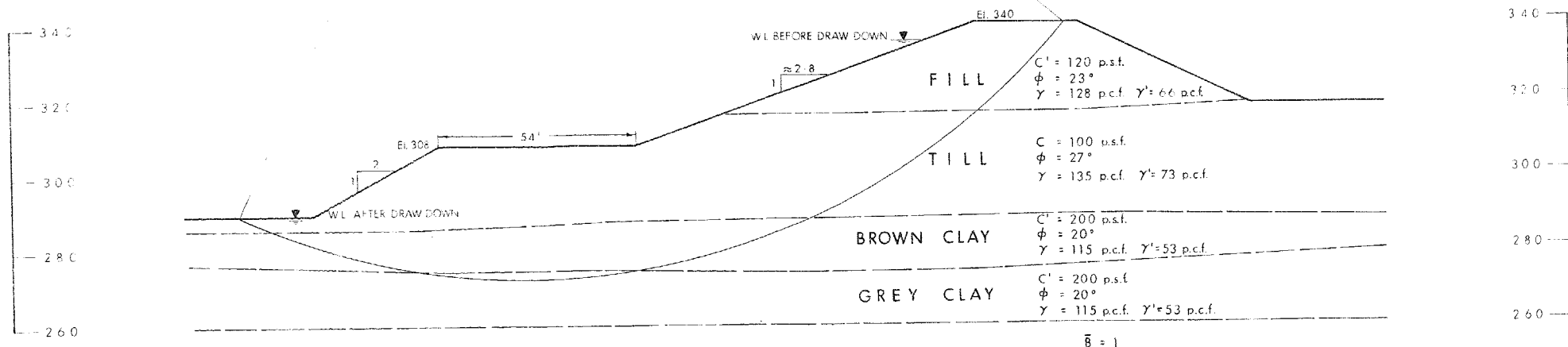


FS = 1.26  
R = 186 ft.

# SECTION THROUGH EAST DYKE AND EXCAVATION

## EFFECTIVE STRESS ANALYSIS

SCALE : 1 INCH = 20 FEET





# NORTH SLOPE OF EXCAVATION TOTAL STRESS ANALYSIS

SCALE : 1 INCH = 10 FEET

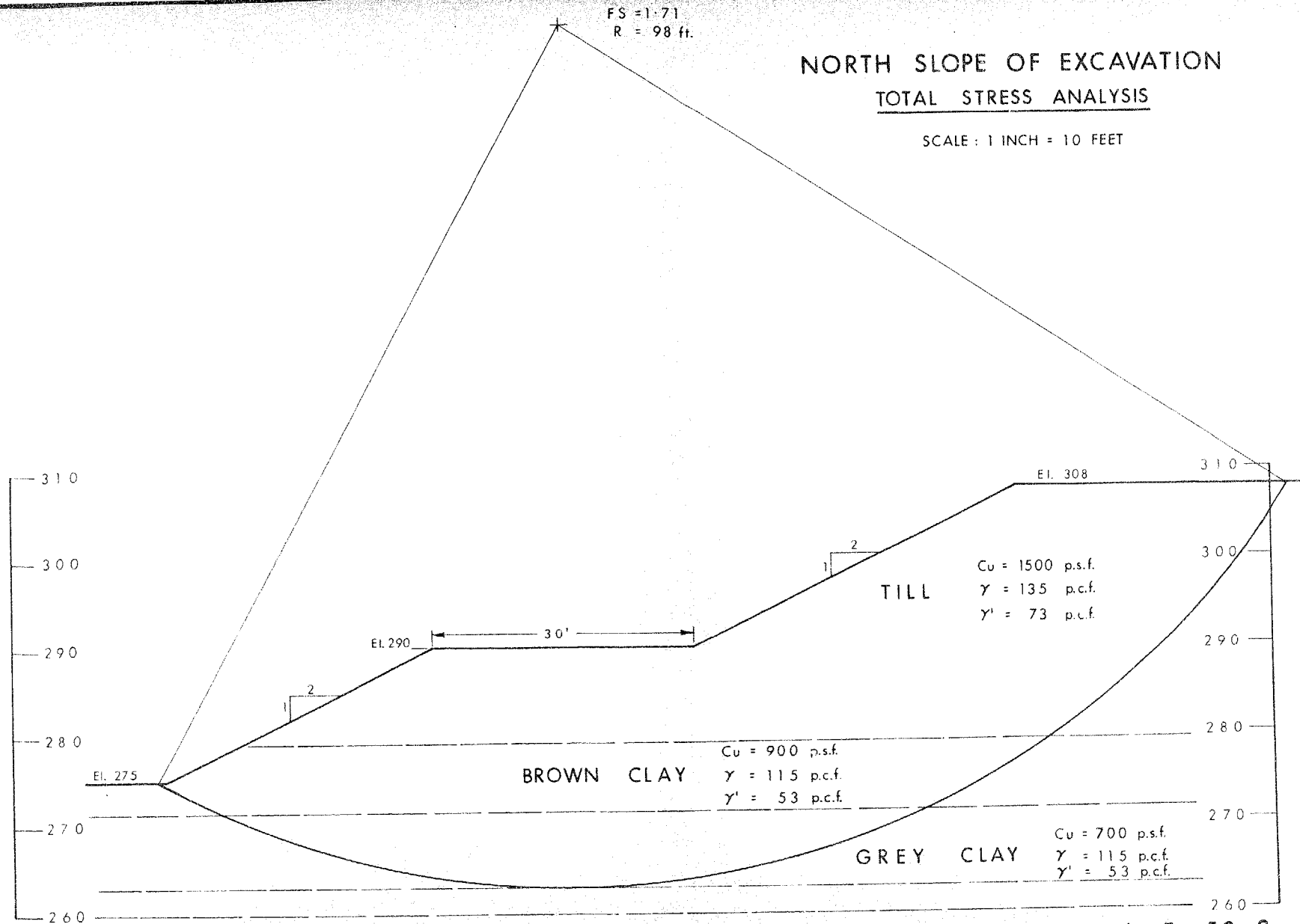


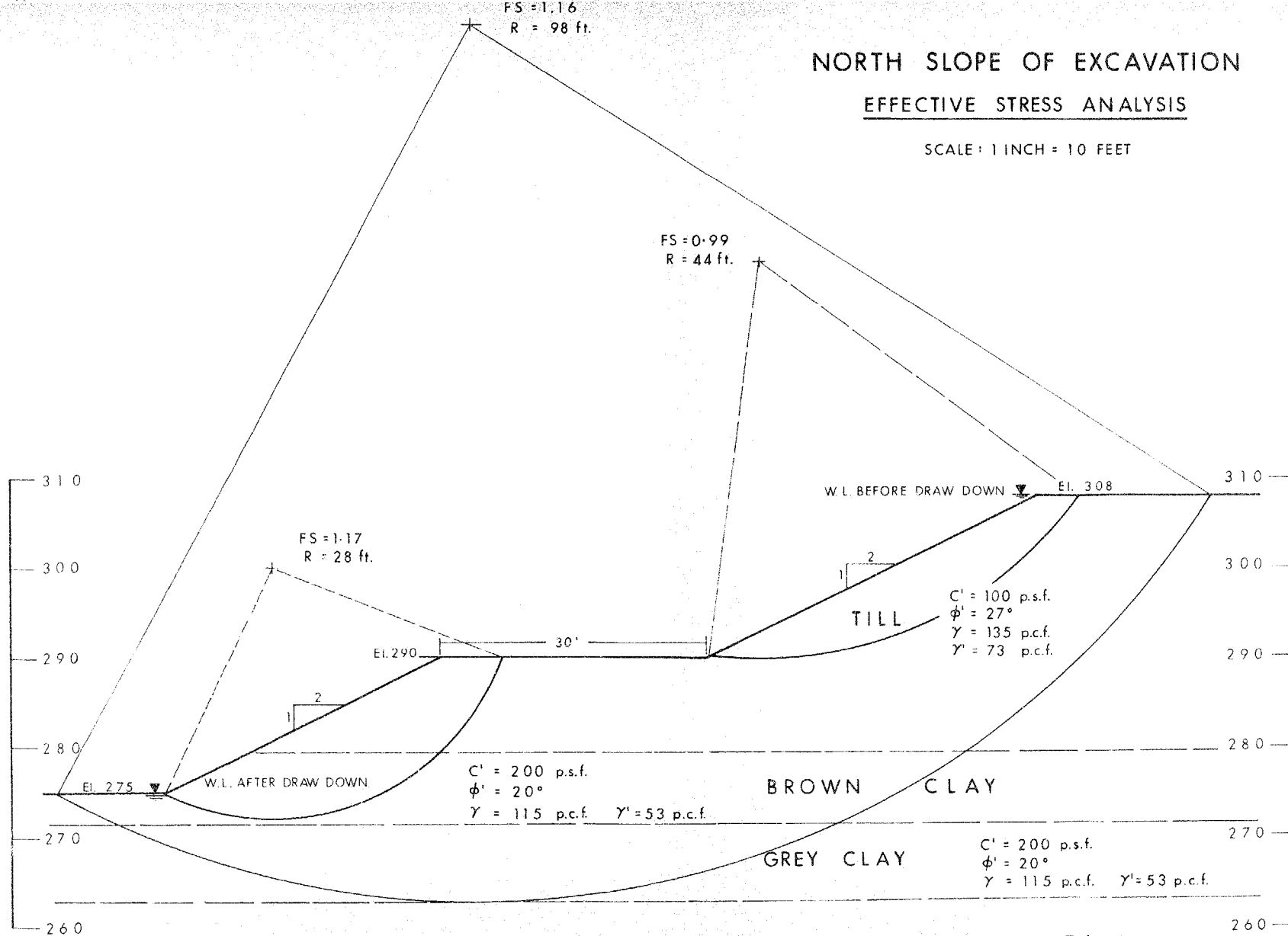
FIG. 14

64 - F - 53 - 3

# NORTH SLOPE OF EXCAVATION

## EFFECTIVE STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET



# NORTH SLOPE OF EXCAVATION

## TOTAL STRESS ANALYSIS

FS = 1.29

R = 111 ft.

SCALE: 1 INCH = 10 FEET

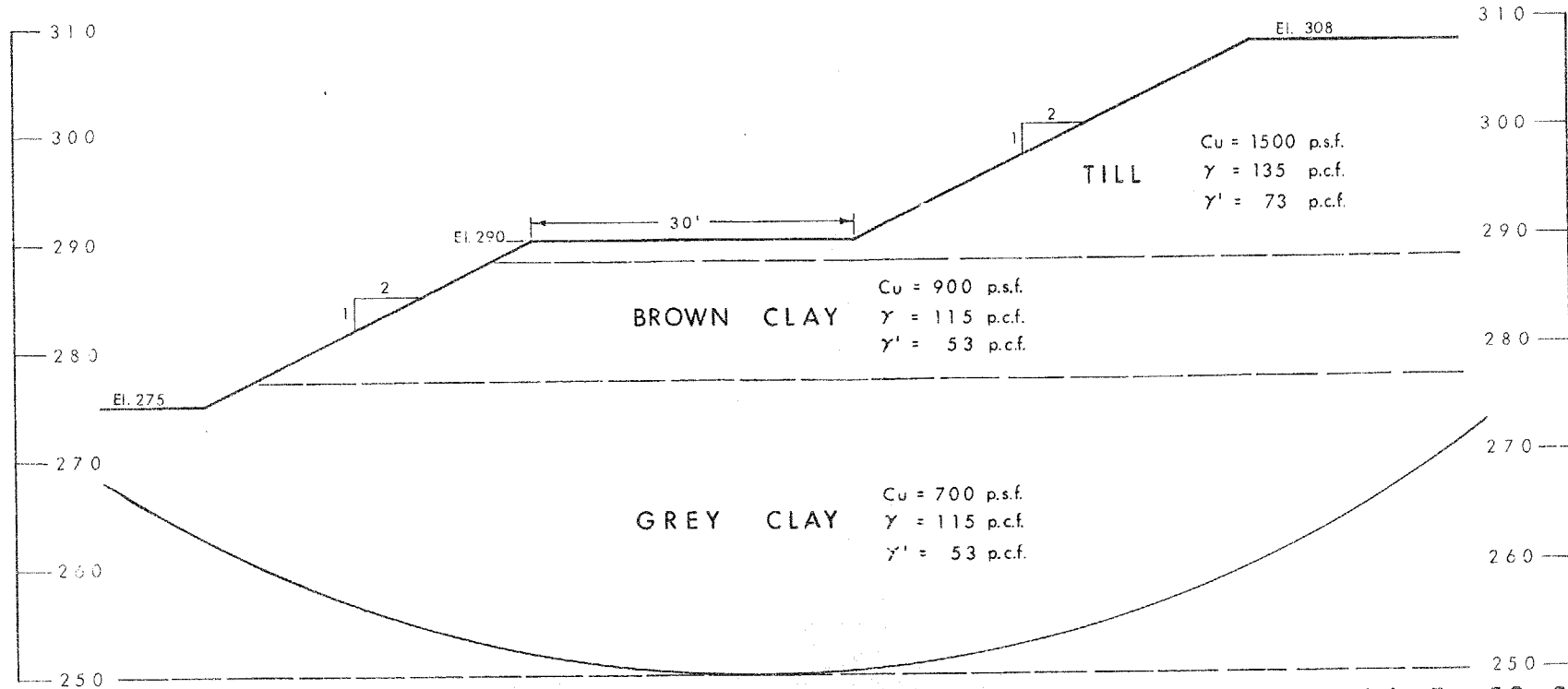


FIG. 16

64-F-53-3

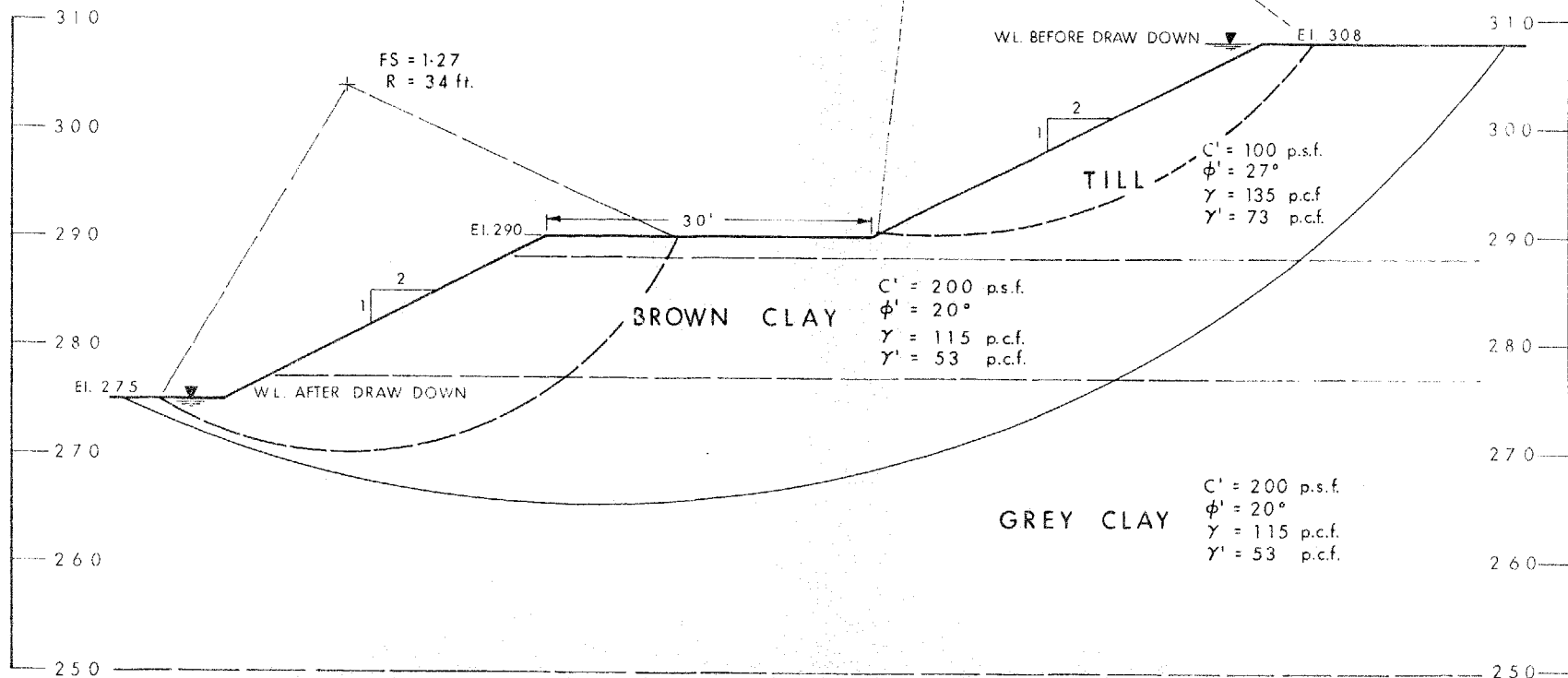
# NORTH SLOPE OF EXCAVATION

## EFFECTIVE STRESS ANALYSIS

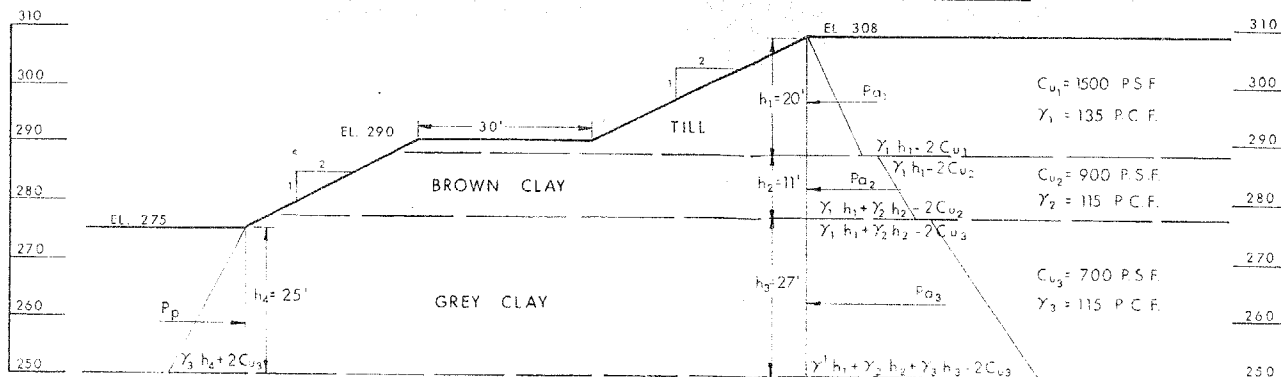
FS = 1.09  
R = 10.5

FS = 0.99  
R = 4.4 ft.

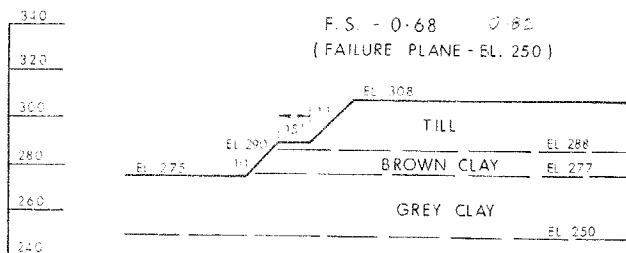
SCALE : 1 INCH = 10 FEET



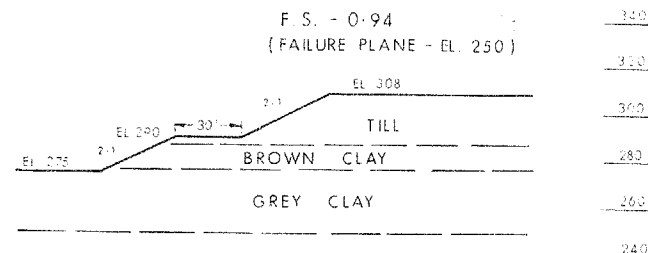
# TOTAL STRESS ANALYSIS - SLIDING BLOCK METHOD



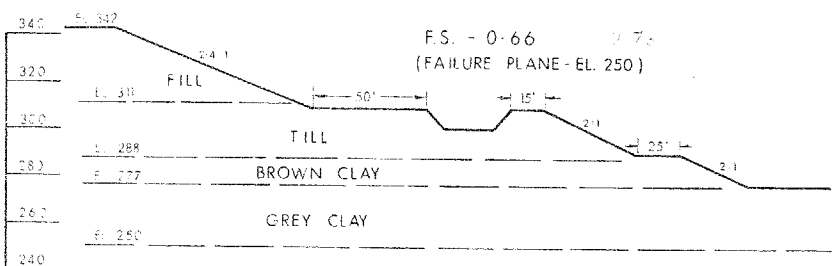
TYPICAL SECTION



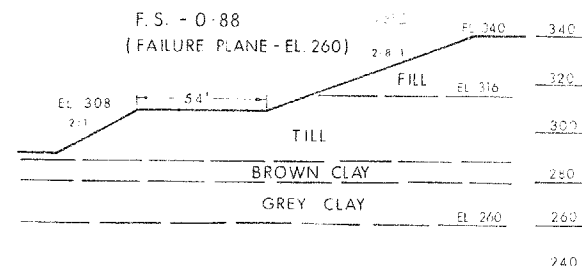
NORTH SLOPE OF EXCAVATION



NORTH SLOPE OF EXCAVATION



SECTION THROUGH WEST DYKE OF EXCAVATION



SECTION THROUGH EAST DYKE OF EXCAVATION

# EFFECTIVE STRESS ANALYSIS - SLIDING BLOCK METHOD

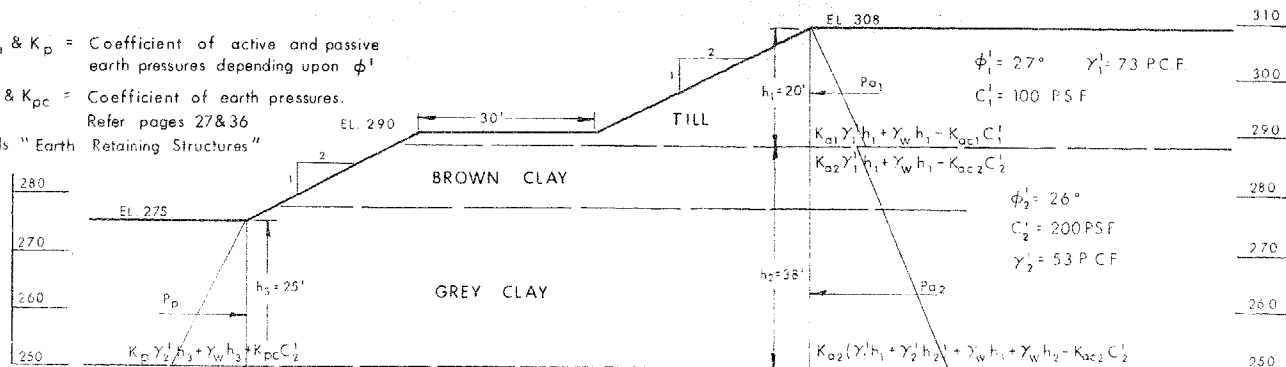
## LEGEND

$K_a$  &  $K_p$  = Coefficient of active and passive earth pressures depending upon  $\phi^1$

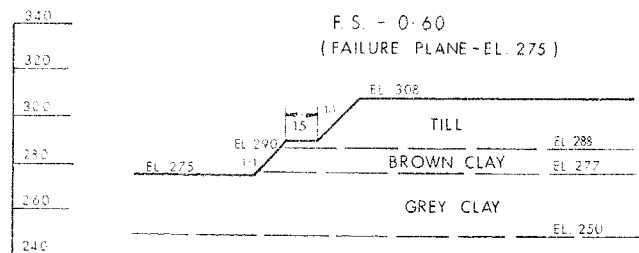
$K_{ac}$  &  $K_{pc}$  = Coefficient of earth pressures.

Refer pages 27 & 36

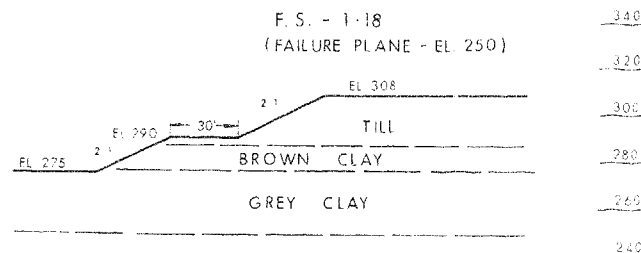
British code of Standards "Earth Retaining Structures"



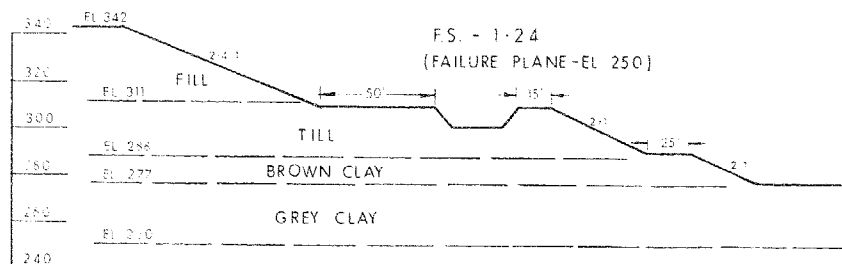
TYPICAL SECTION



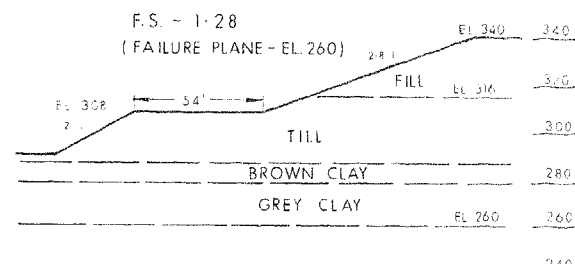
NORTH SLOPE OF EXCAVATION



NORTH SLOPE OF EXCAVATION



SECTION THROUGH WEST DYKE OF EXCAVATION



SECTION THROUGH EAST DYKE OF EXCAVATION

ADDENDUM

TO

INTERIM DESIGN REPORT

Slope Stability - Excavations in Canal Bed

W.J. 64-F-53-3

--

W.P. 444-64

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At the meeting of December 17, 1964, held at the Department of Highways, the decision was reached to cut back a 100-ft. length of the north side of the excavation to an average slope of 2:1, to give one additional case for study.

Effective and total stress analyses of the 2:1 slope cut were carried out using the program of the electronic computer. The results of these computations are shown on the attached Drawings No. 20 and 21. The soil characteristics used in the analyses are also presented on these drawings.

The minimum factor of safety in terms of total stresses is 1.15, while the effective stress analyses gave a minimum factor of safety of only 0.77. This would indicate that the slope should be unstable during the drawdown conditions that will take place next season after the canal is again emptied.

The drawdown condition to be simulated just before the filling of the canal in the spring of 1965 may not be as critical as the one in 1966 because of the limited time the water is to be left in the excavation prior to pumping.

Apparently, none of the test slope seems to be stable, depending on the type of analysis used. The choice of soil parameters, of course, is yet another source of possible error. The only slope

cont'd. /2 ...

that was found to be stable under all conditions (end of construction and the rapid drawdown case) is the one called "Proposed Slope" which has a 30-ft. berm at elev. 290 and 2:1 slopes. However, the test sections may show that this slope is too conservative and some other cross section may be finally adopted.

AGS/MdeF

*A. G. Stermac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER



# NORTH SLOPE OF EXCAVATION (TEST SECTION)

## TOTAL STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET

FS = 1.15

R = 96

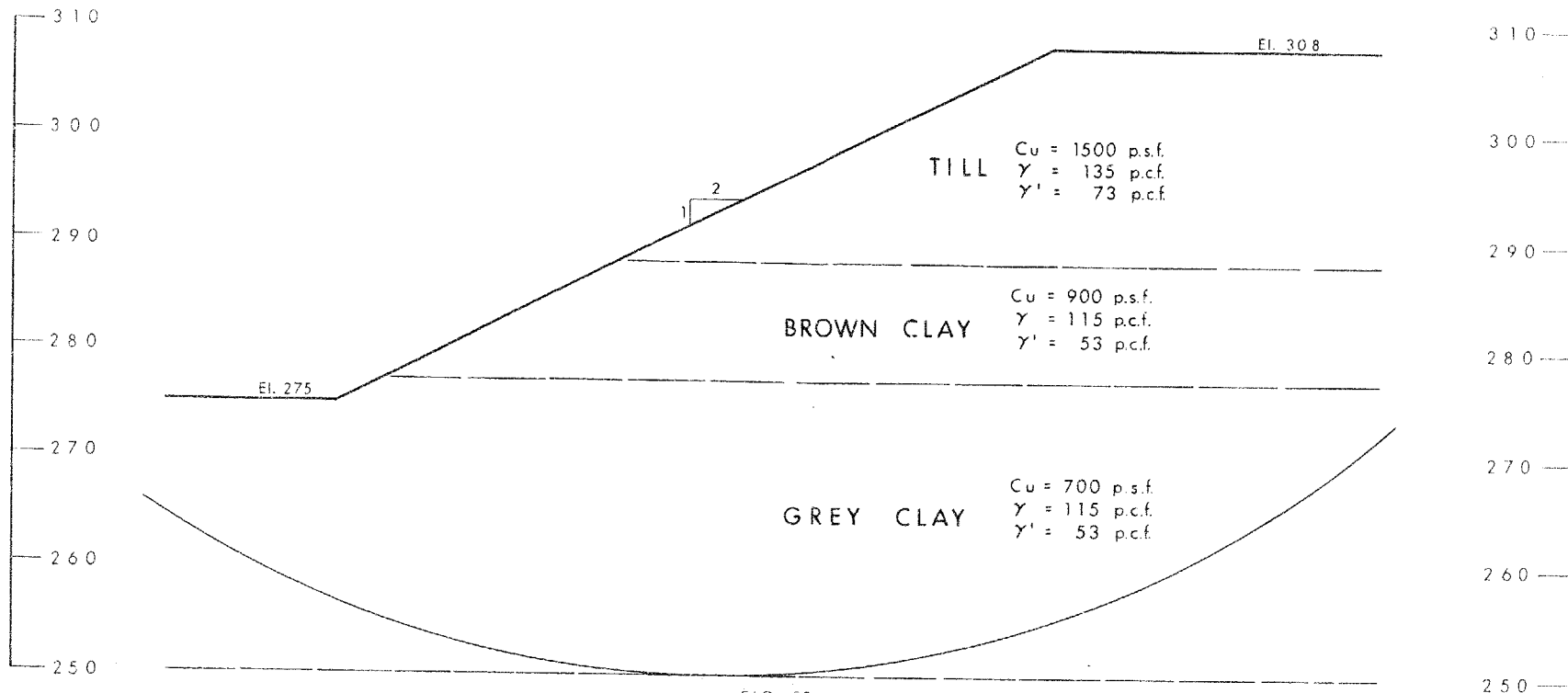


FIG. 20

64-F-53-3

# NORTH SLOPE OF EXCAVATION (TEST SECTION)

## EFFECTIVE STRESS ANALYSIS

SCALE: 1 INCH = 10 FEET

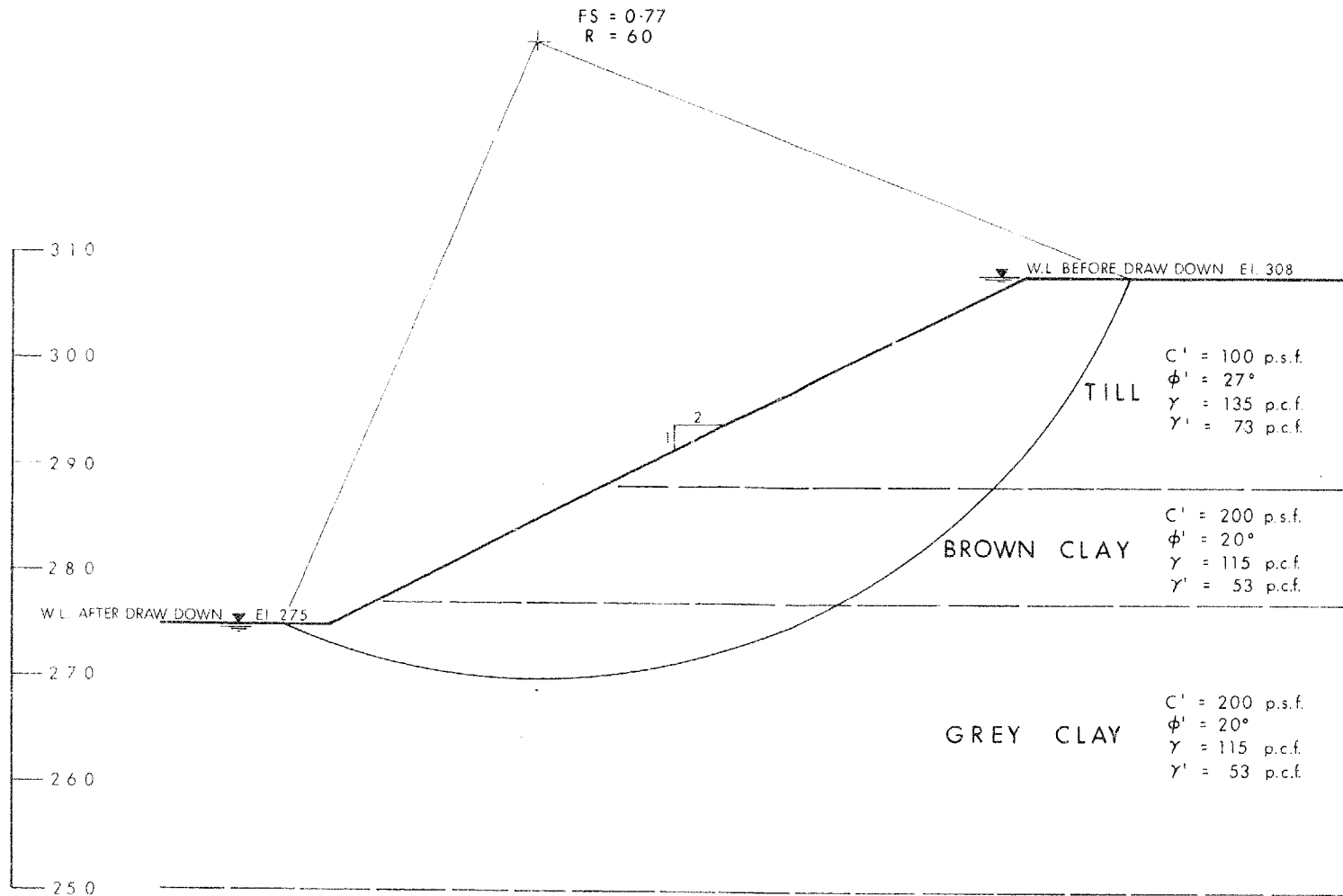


FIG. 21

64-F-53-3

## MEMORANDUM

To: Mr. B. R. Davis,  
Bridge Engineer,  
Bridge Division.

FROM: Foundation Section,  
Materials and Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. F. I. Hewson,  
Consultant Liaison  
Engr.

DATE: May 18, 1966

OUR FILE REF.

IN REPLY TO

SUBJECT:

## REPORT ON PUMPING TEST

At

CARLTON STREET TUNNEL  
St. Catharines, Ontario,  
District #4 (Hamilton)

W.J. 64-F-53-4 -- W.P. 444-64

Enclosed, please find our report on the pumping test recently carried out at the site of the Carlton Street Tunnel.

We believe that the factual information and recommendations contained in the report will be sufficient for your purposes.

Should additional information be required, or should any points in the report require further clarification, please feel free to call our Office.

AGS/MdeF  
Attach.

cc: Messrs. B. R. Davis (2)  
H. A. Tregaskes  
D. W. Farren  
G. K. Hunter (2)  
H. Greenland  
T. J. Kovich

*A. G. Stermac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

W. Melinyshyn  
General Engineering Co. (GECO) (2)  
Foundations Office  
Gen. Files

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  3. PIEZOMETER INSTALLATION.
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  5. TEST RESULTS.
  6. SUMMARY.
  7. MISCELLANEOUS.
-

REPORT ON PUMPING TEST  
At  
CARLTON STREET TUNNEL  
St. Catharines, Ontario,  
District #4 (Hamilton).  
W.J. 64-F-53-4 -- W.P. 444-64

1. INTRODUCTION:

Following the 1965 navigation season and the subsequent drawdown of the Welland Canal at the site of the Carlton St. Tunnel in St. Catharines, certain field investigations were carried out by the Foundation Section. As a result of these investigations new information relating to groundwater conditions within the stratified granular deposits located some 10 feet below the tunnel subgrade at the lowest point, was obtained. In the light of this new information which showed the permeability of the stratified deposits to be very much higher than was previously thought, it was decided that a dewatering scheme would be necessary to ensure the stability of the tunnel subgrade during construction work in the 1966-67 drawdown period.

In order that the dewatering scheme can be fully operative at the commencement of construction, it was decided to install it as soon as possible during the present (1965-66) drawdown period. Accordingly, a contract was awarded to Wellpoint Dewatering Corporation to supply and install a number of eductor wells, together with all necessary pumping equipment. Work commenced on February 16, 1966, and by March 14, 1966, the installation was essentially complete. A comprehensive account of the events leading up to the decision to install the eductor wells is contained in a letter from Mr. F. Sutcliffe, G.E.C.O. Ltd. to Mr. H. W. Adcock, Assistant Deputy Minister (Engineering), D.H.O.

In order to obtain information on the pumping time required to lower the groundwater level in the stratified zone to

a particular level, a full-scale pumping test was commenced on March 14, 1966, and was completed on April 21, 1966. This report contains a description of the test, together with conclusions and recommendations for the future dewatering operations based on the test results.

## 2. DEWATERING INSTALLATION:

A total of 52 eductor wells was installed at the site. Ten of these were installed on the south side of the trench and 42 on the north side. The initial phase of the installation consisted of searching for the most permeable zones by means of trial wells since it was realized that great variations in permeability existed among the irregularly stratified granular layers, partly because of discontinuity and partly because of varying grain size. The most productive water-bearing strata in the area outside the excavation, were found to be on the north side of the trench between the approximate mid point and the diverted canal drainage ditch. On the south side of the trench the areal extent of the strata which produced water in reasonably large quantities, was relatively small and was confined to a zone near the southwest corner. In order that the dewatering could be carried out whilst the canal was full, all pumping equipment was installed on the west dyke, and when the system was operating, water was discharged into the canal.

The layout and details of the eductor well system are shown on Drawing #64-F-53D which is contained in the Appendix of this report.

## 3. PIEZOMETER INSTALLATION:

Eight piezometers were installed in the stratified zone at various locations around the trench. The tips of these

3. PIEZOMETER INSTALLATION: (cont'd.) ...

piezometers were located at different elevations within the stratified granular layers. Three of the piezometers were installed in the canal bed, but were fitted with a remote control measuring device which enabled them to be read by means of pressure gauges located on the west dyke. The remainder of the piezometers were located on the east and west dykes. By means of this installation, direct observations could be carried out - even with the canal full of water.

The location and elevations of the piezometers are shown on Drawing #64-P-53D.

4. PUMPING TEST:

Pumping commenced on March 14, 1966, and was terminated on April 21, 1966. During this time frequent observations of piezometric level and rate of discharge were made. Since the termination of pumping operations, observations of the piezometers have been continued up to the present time. Fig. I of the Appendix shows a plot of piezometric head versus time for the period March 14 to May 13, 1966, and rate of discharge versus time for the full period of pumping operations.

5. TEST RESULTS:

An examination of the test results as plotted on Fig. I, shows that most piezometers located in the upper portion of the stratified deposit - i.e., above elev. 254.0, indicated a drop in head of about 12.0 feet during the pumping operations. Two piezometers, #50P-A and 50P-C which were located at elev. 240.9 and elev. 254.6, respectively, indicated a drop in head of about 26 feet. It is known from the experience of the eductor well installation, that the most permeable layers are located around

cont'd. /4 ...

5. TEST RESULTS: (cont'd.) ...

elev. 240, and it is therefore to be expected that the water head in these layers would be lowered much more rapidly than the less permeable layers above. However, it is necessary to lower the water level to a safe level in the entire stratum to ensure stability of the excavation base during the future construction operations, and the problem must be considered in this light.

From the results of our investigation of the piezometric levels in the various layers as reported in Foundation Report #64-F-53-2, the average piezometric head at the base of the grey clay stratum is about elev. 296. It is estimated that the required safe head to ensure stability of the trench base, is about elev. 270. To achieve this, it is necessary to lower the head by about 26 feet, and since 37 days of pumping were required to lower the level 12 feet, it appears that about  $2\frac{1}{2}$  months of pumping using the same installation, would be required.

Based on the foregoing, the following steps are recommended for the future construction operations:

(1) Dewatering operations should be started on or about the beginning of November, 1966.

(2) As soon as possible after the canal drawdown, additional piezometers should be installed close to the affected area to determine the piezometric head over the whole zone. At that time it can be decided whether or not additional eductors are necessary.

(3) A separate dewatering scheme will probably be necessary for the sump to be excavated in the trench base. This can be designed when the prevailing piezometric levels are established, just prior to construction of the sump.

cont'd. /5 ...



6. SUMMARY:

A pumping test recently carried out at the site of the Carlton Street Tunnel, St. Catharines, is reported.

The object of the test was to determine the piezometric water levels prevailing in the stratified granular layers below elev. 260.0, whilst the layers were being dewatered by means of an eductor well system.

The test results showed that after 37 days of pumping the piezometric head was lowered about 12 feet in the upper part of the stratified zone, and about 26 feet in the lower part. It is estimated that about  $2\frac{1}{2}$  months of pumping using the same installation would be required to lower the head by 26 feet throughout the entire stratum.

Recommendations pertaining to future dewatering operations are contained in the foregoing section headed 'Test Results'.

7. MISCELLANEOUS:

The pumping test was carried out during the period March 14 to April 21, 1966, by Wellpoint Dewatering Corporation Ltd. The piezometer installation was carried out by Canadian Longyear Ltd., under the supervision of Mr. Paul Payer, Project Foundation Engineer, who also carried out the field test observations. This report was prepared by Mr. K. G. Selby, Supervising Foundation Engineer, with the assistance of Mr. Payer.

May 1966

APPENDIX I

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - 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>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FCIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

# ABBREVIATIONS 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
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 = $\frac{e}{\Delta e}$
$c_v$	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

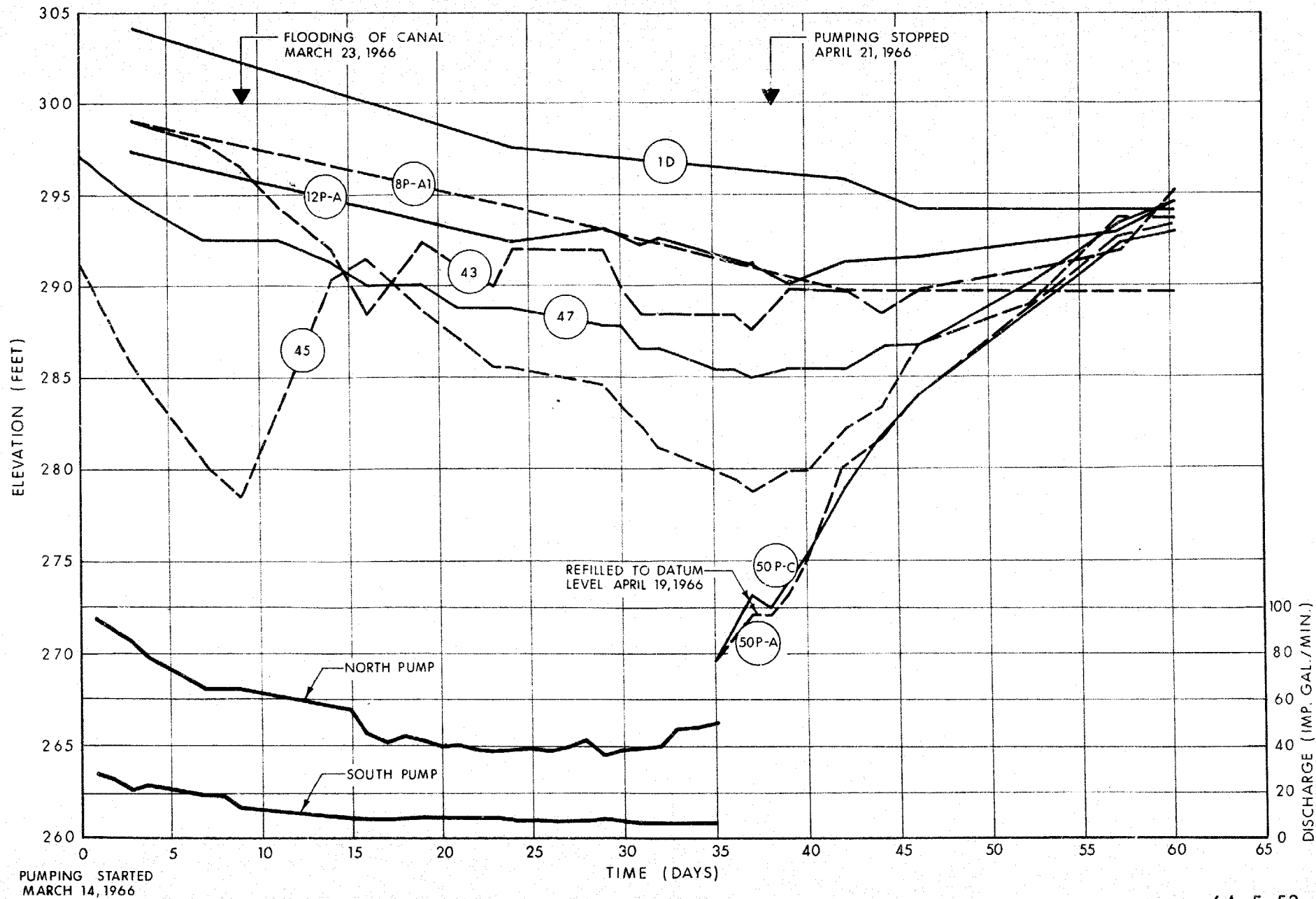
## FOUNDATIONS

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

# PIEZOMETRIC HEAD AND DISCHARGE OF EDUCTOR WELLS VS TIME



Department of Highways Ontario

Copy for the information of

Mr. A. C. Stermac, Principal Foundation Engineer, Room 107, Lab. Building

Bridge Division,  
Downsview, Ontario,  
September 30, 1966

St. Lawrence Seaway Authority,  
Box 592,  
ST. CATHARINES, Ontario.

Attention: W. A. O'Neill, P.Eng.

RE: Carlton St. Tunnel,  
W.P. 444-64

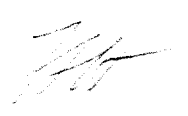
Dear Sir:

Last May Wellpoint Dewatering were told that it was unlikely we would require this installation at Carlton Street but that it could not be removed during the navigation season.

Since the bridge at Lock 2 is to be twinned, it appears unlikely a tunnel will be built at this location.

We would like to advise Wellpoint Dewatering when they can begin to salvage their equipment and thus put an end to the \$39.00 per day standby charge.

Yours truly,



FIH/pr

E. I. Hewson,  
Sr. Bridge Liaison Engineer

cc. R. Loughney  
A. G. Stermac  
J. Walter  
H. Greenland

64-F-57  
Copy File

Department of Highways Ontario

Copy for the information of  
Mr. A.G. Stermac, Principal Foundation Engineer,  
Room 107, Lab. Building

Bridge Division,  
Downsview, Ontario, Canada,  
May 30, 1966

Wellpoint Dewatering of Canada Limited,  
881 East 141st Street,  
Bronx,  
New York 10454, U.S.A.

Attention: Mr. R.W. Loughney

RE: Carlton St. Tunnel  
Contract 66-57, District No. 4

Dear Sirs:

The Federal Government has announced that the Welland Canal may be shifted to a new location almost a mile east of its present position. Such a shift would probably make our tunnel useless, so the call for bids has been cancelled until a decision has been made on the future of the Canal.

Since construction sequence depends on winter operations, there will be at least a full year's delay if the contract is called again. Consequently, it is unlikely that any pumping will be required next winter. If the decision is made to eliminate this tunnel, you might be able to salvage your plant while the Canal is dewatered next winter.

If you have any questions, please do not hesitate to contact us.

Yours truly,

F.I. Hewson, P. Eng.,  
Senior Bridge Liaison Engineer.

FIH:rd

c.c. A.G. Stermac  
F.H. Sutcliffe  
W.A. O'Neill  
H. Greenland

#64-F-53-1

W.P.#444-64

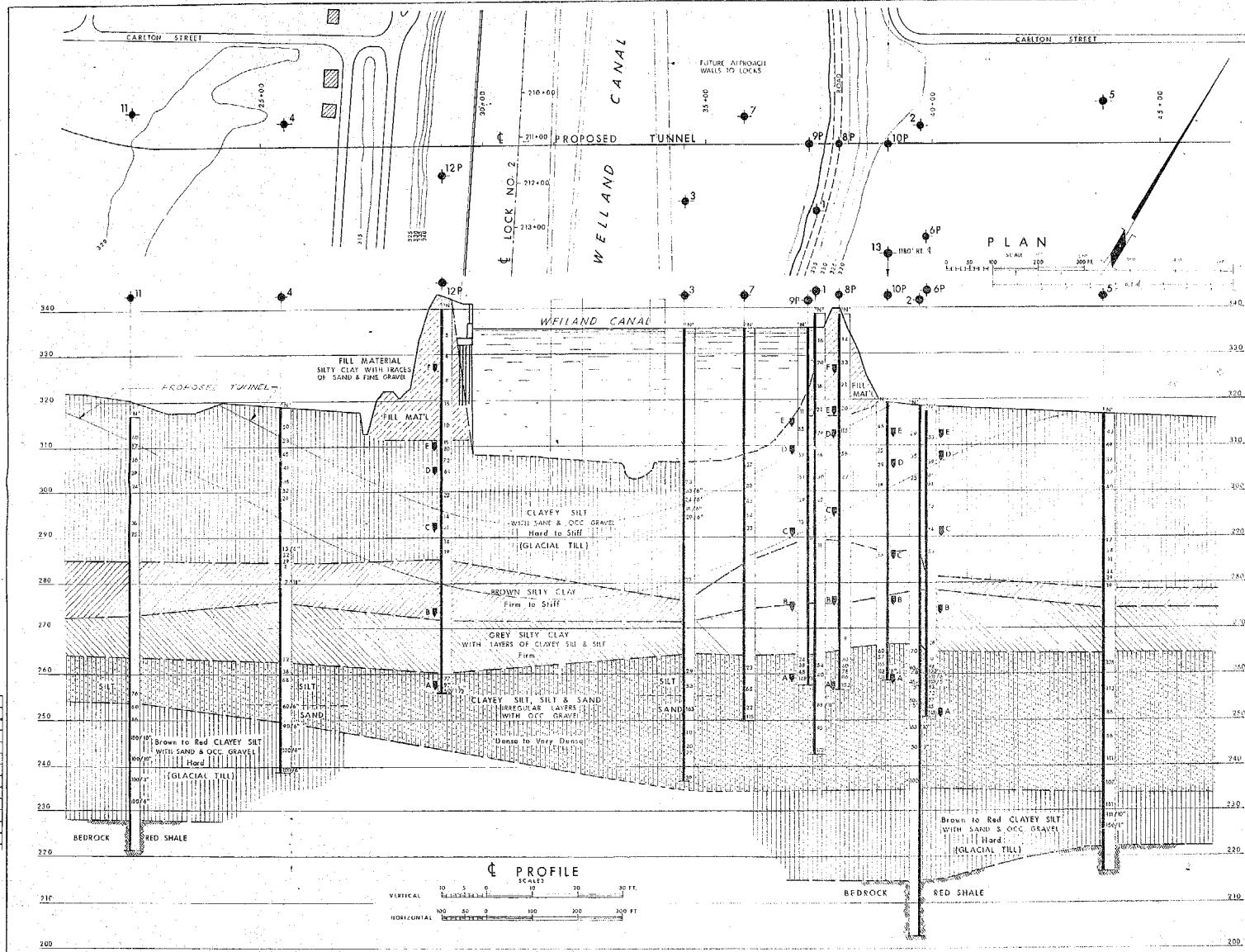
W.P.#444-65

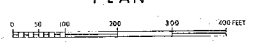
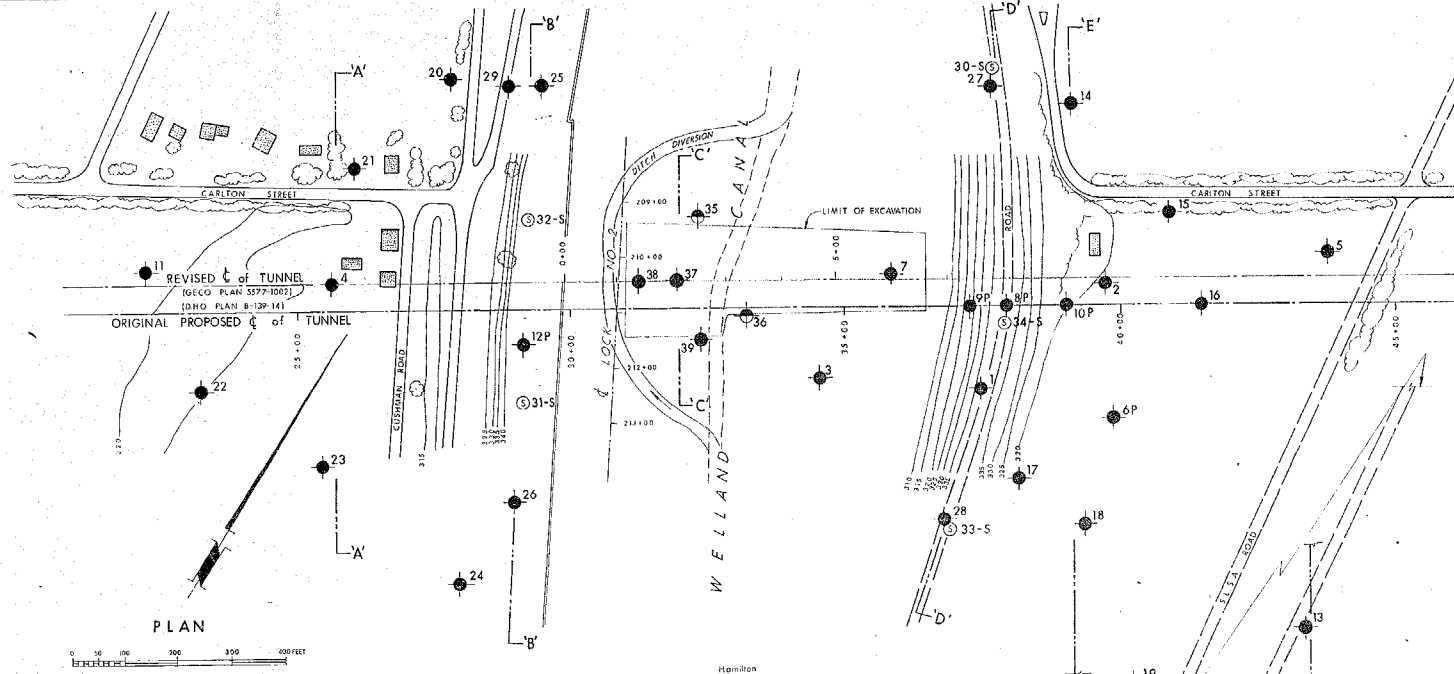
CARLTON ST.

TUNNEL

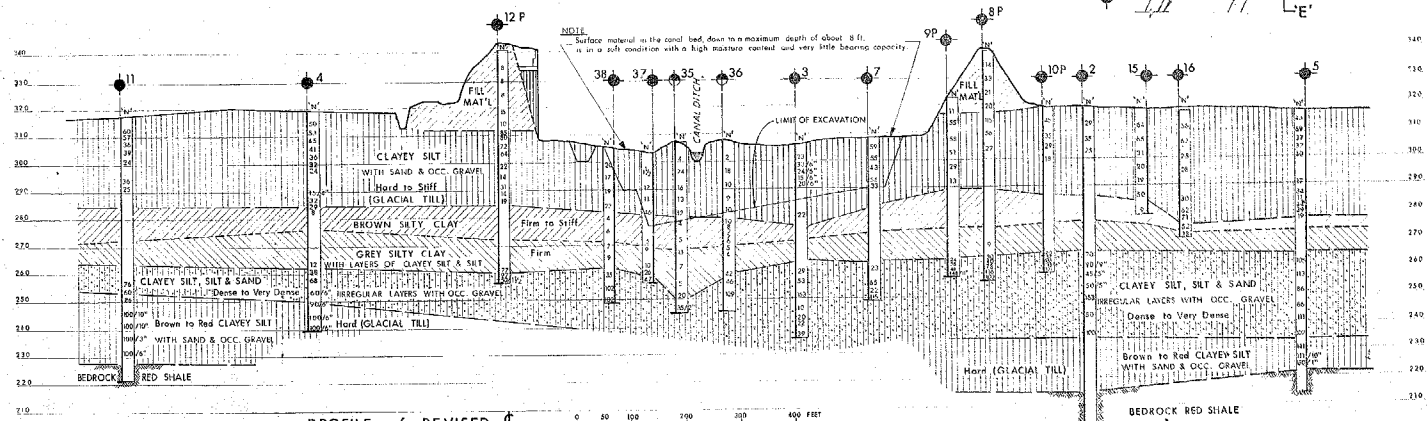
WELLAND CANAL



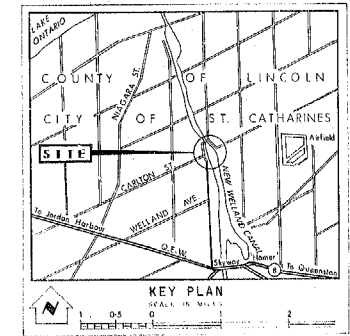
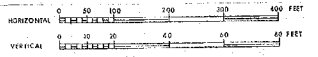




PLAN



PROFILE of REVISED C  
(AFTER CANAL DRAW DOWN)



**LEGEND**

- Bore Hole
- Bore & Slope Indicator Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation
- Ⓢ Slope Indicator
- P Piezometer

**SLOPE INDICATOR DATA**

WELL NO.	STATION	OFFSET	TOP ORIGINAL ELEVATION	TIP ELEVATION	LENGTH
30	27+70	430' LT	242.5	270.0	72.5'
31	29+12	170' RT	243.3	263.2	80.1'
32	29+20	165' LT	242.0	262.8	79.2'
33	26+90	400' RT	240.2	258.0	82.2'
44	27+90	300' RT	240.8	261.8	79.0'
35	32+25	165' LT	210.0	246.5	63.5'
26	33+25	15' RT	210.0	246.5	63.5'

Refer to Dwg 64-F-53C for Bore Hole Data & Sections A-A, B-B, C-C, D-D, E-E

**NOTE**

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

**DEPARTMENT OF HIGHWAYS - ONTARIO**  
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

**CARLTON STREET PROPOSED TUNNEL**  
WELLAND CANAL

KING'S HIGHWAY NO. \_\_\_\_\_ DIST. NO. 4  
CO. LINCOLN CITY OF ST. CATHARINES  
TWP. \_\_\_\_\_ LOT \_\_\_\_\_ CON. \_\_\_\_\_

**BORE HOLE LOCATIONS & SOIL STRATA**

DESIGNED BY: \_\_\_\_\_ DRAWN BY: \_\_\_\_\_  
CHECKED BY: \_\_\_\_\_ DATE: FEB. 5, 1965  
APPROVED BY: \_\_\_\_\_



#64-F-53-2

W.P. # 444-64

WP # 444-65

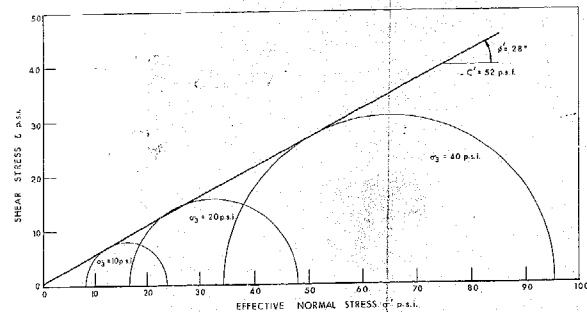
CARLTON ST.

TUNNEL

WELLAND

CANAL

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS  
WITH PORE PRESSURE MEASUREMENTS



FILL

BORE HOLE NO. .... 27  
SAMPLE NO. .... 2  
DEPTH ..... 6'-11" - 7'-4"  
BULK DENSITY ..... 128.0 %  
LIQUID LIMIT ..... 34.5 %  
PLASTIC LIMIT ..... 21.2 %  
INITIAL MOISTURE CONTENT ..... 26.5 %  
FINAL MOISTURE CONTENT ..... 18.6 %  
DEGREE OF SATURATION ..... 100 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

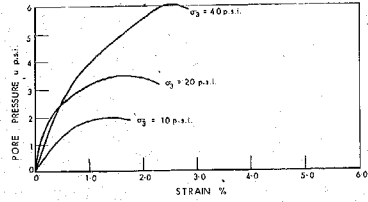
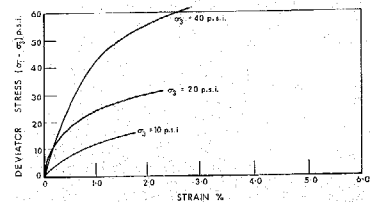
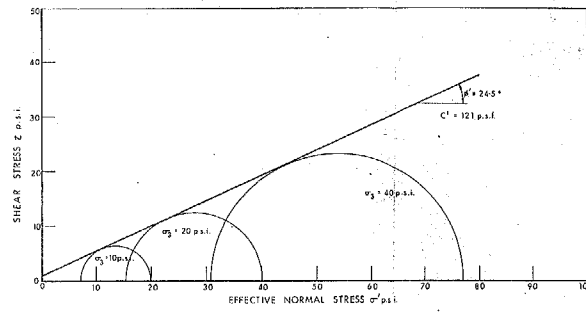


FIG. 12 (a)

JOB 64-F-53

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS  
WITH PORE PRESSURE MEASUREMENTS



FILL

BORE HOLE NO. .... 28  
SAMPLE NO. .... 6  
DEPTH ..... 18'-8" - 19'-2"  
BULK DENSITY ..... 122.0 p.c.f.  
LIQUID LIMIT ..... 49.1 %  
PLASTIC LIMIT ..... 23.9 %  
INITIAL MOISTURE CONTENT ..... 27.6 %  
FINAL MOISTURE CONTENT ..... 22.8 %  
DEGREE OF SATURATION ..... 98.0 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

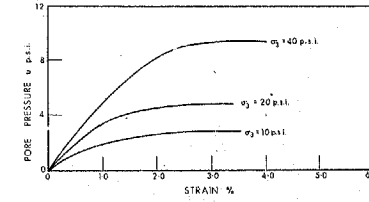
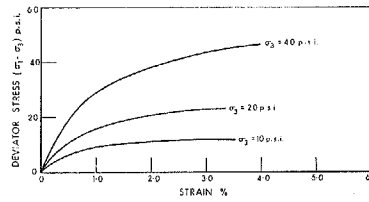
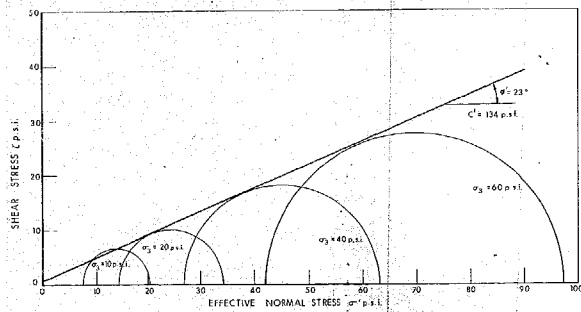


FIG. 12 (c)

JOB 64-F-53

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS  
WITH PORE PRESSURE MEASUREMENTS



FILL

BORE HOLE NO. .... 12  
SAMPLE NO. .... 10  
DEPTH ..... 8'-7" - 8'-11"  
BULK DENSITY ..... 124.5 p.c.f.  
LIQUID LIMIT ..... 44.8 %  
PLASTIC LIMIT ..... 21.9 %  
INITIAL MOISTURE CONTENT ..... 27.3 %  
FINAL MOISTURE CONTENT ..... 23.8 %  
DEGREE OF SATURATION ..... 100 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

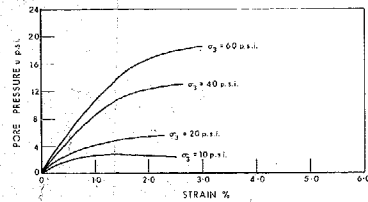
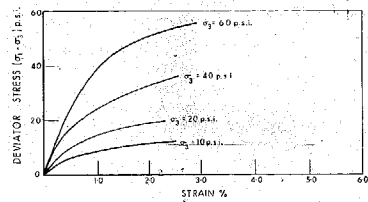
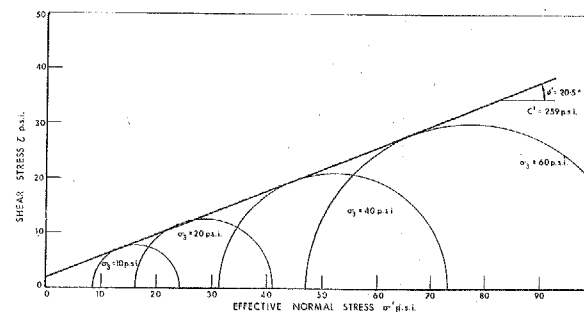


FIG. 12 (b)

JOB 64-F-53

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS  
WITH PORE PRESSURE MEASUREMENTS



FILL

BORE HOLE NO. .... 26  
SAMPLE NO. .... 6  
DEPTH ..... 19'-1" - 19'-6"  
BULK DENSITY ..... 124.0 p.c.f.  
LIQUID LIMIT ..... 52.8 %  
PLASTIC LIMIT ..... 23.5 %  
INITIAL MOISTURE CONTENT ..... 25.0 %  
FINAL MOISTURE CONTENT ..... 22.6 %  
DEGREE OF SATURATION ..... 98.4 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

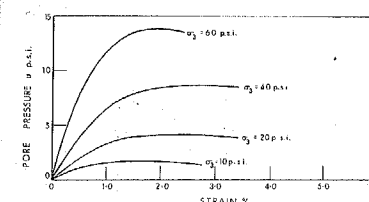
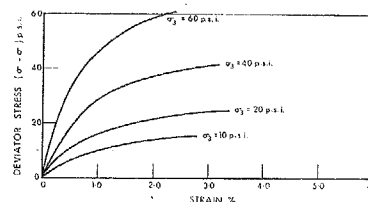


FIG. 12 (d)

JOB 64-F-53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

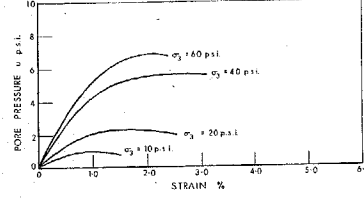
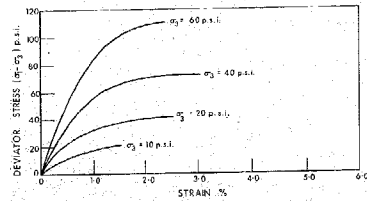
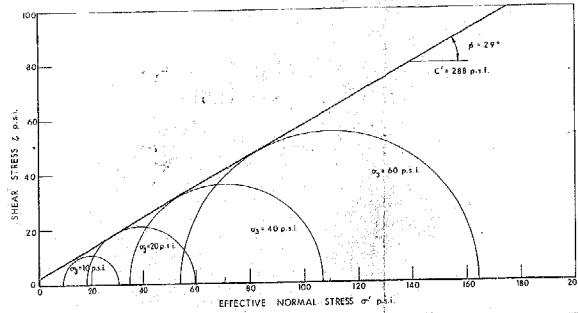


FIG. 13(a)

JOB - 64 - F - 53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

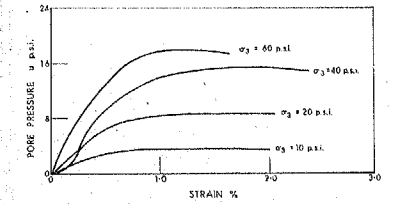
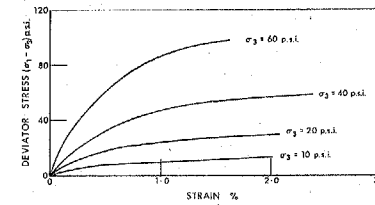
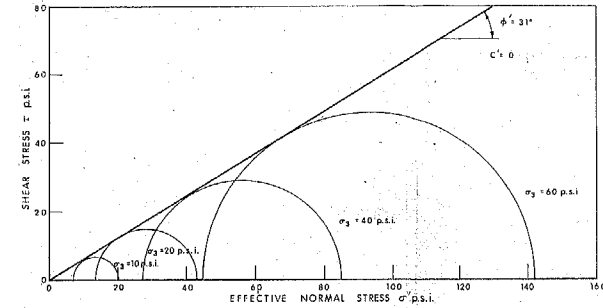


FIG. 13(c) (Rev. Fig. 9)

JOB - 64 - F - 53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

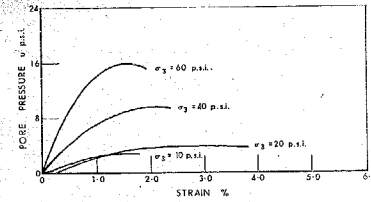
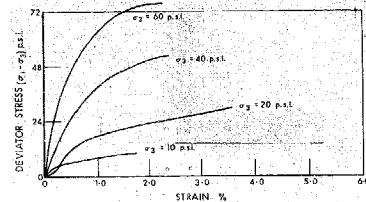
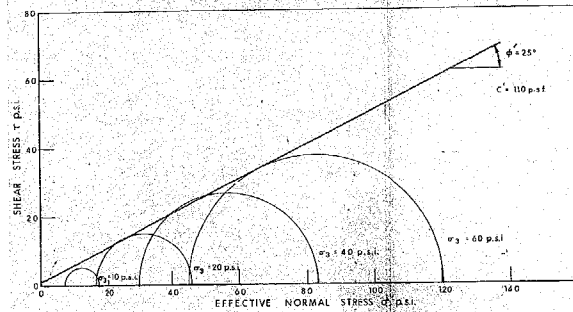
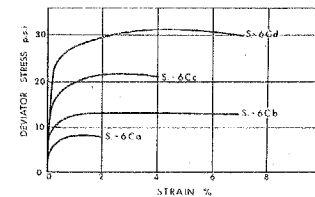
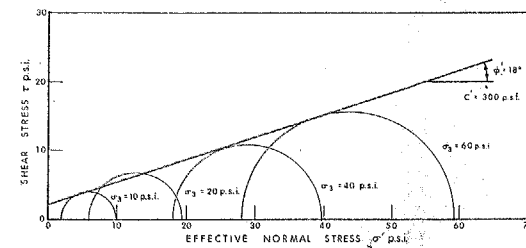


FIG. 13(b) (Rev. Fig. 8)

JOB - 64 - F - 53

# CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TESTS

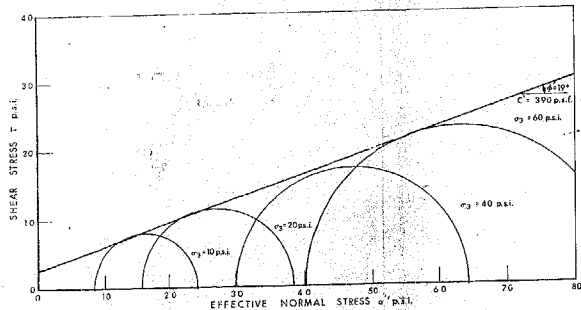


NOTE: (1)  $\sigma_3$  MAINTAINED CONSTANT WITH  $\sigma_1$  DECREASING.  
(2) SHEAR STRENGTH DEFINED AS THE MAXIMUM PRINCIPAL STRESS DIFFERENCE  $\sigma_1 - \sigma_3$

FIG. 14(a) (Rev. Fig. 10)

JOB - 64 - F - 53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS



## BROWN CLAY

BORE HOLE NO. .... 15D  
SAMPLE NO. .... 15D  
DEPTH ..... 54'-9"-55'-2"  
BULK DENSITY ..... 119.8 p.c.f.  
LIQUID LIMIT ..... 42.6 %  
PLASTIC LIMIT ..... 20.7 %  
INITIAL MOISTURE CONTENT ..... 32.1 %  
FINAL MOISTURE CONTENT ..... 27.4 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

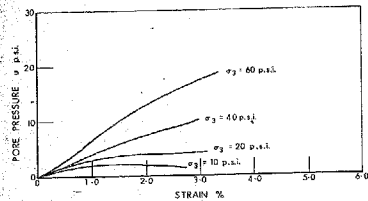
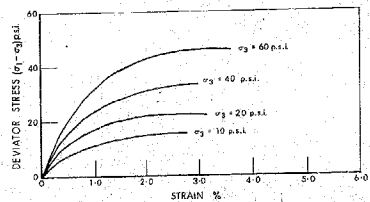
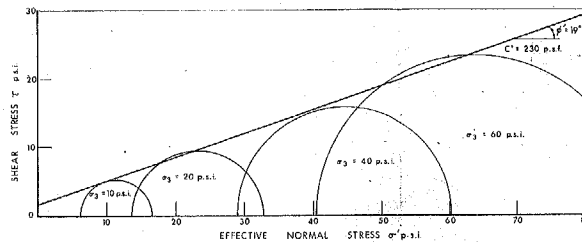


FIG. 14 (b)

JOB - 64 - F - 53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS



## GREY CLAY

BORE HOLE NO. .... 9P  
SAMPLE NO. .... 11  
DEPTH ..... 55'-2"-55'-6  
BULK DENSITY ..... 122.5  
LIQUID LIMIT ..... 32.6 %  
PLASTIC LIMIT ..... 18.6 %  
INITIAL MOISTURE CONTENT ..... 28.0 %  
FINAL MOISTURE CONTENT ..... 24.3 %  
 $\sigma_3$  CONSTANT  
 $\sigma_1$  INCREASING

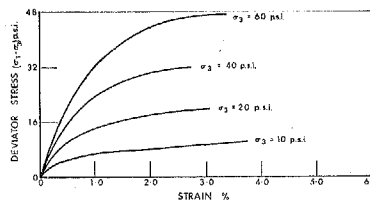
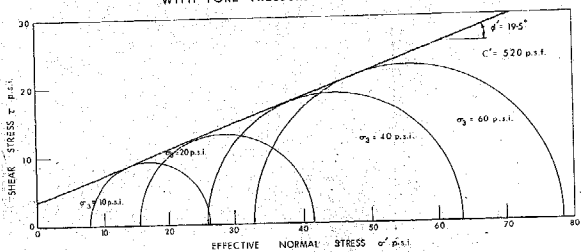


FIG. 15 (b)

JOB - 64 - F - 53

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS



## GREY CLAY

BORE HOLE NO. .... 9P  
SAMPLE NO. .... 11  
DEPTH ..... 54'-8"-55'-3"  
BULK DENSITY ..... 124.3 p.c.f.  
LIQUID LIMIT ..... 32.6 %  
PLASTIC LIMIT ..... 18.6 %  
INITIAL MOISTURE CONTENT ..... 30.3 %  
FINAL MOISTURE CONTENT ..... 24.2 %  
 $\sigma_3$  CONSTANT,  $\sigma_1$  INCREASING

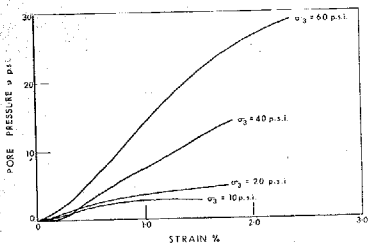
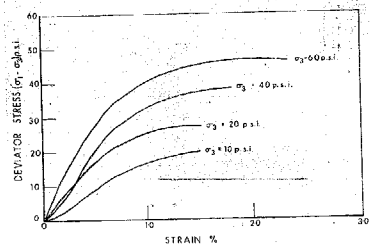
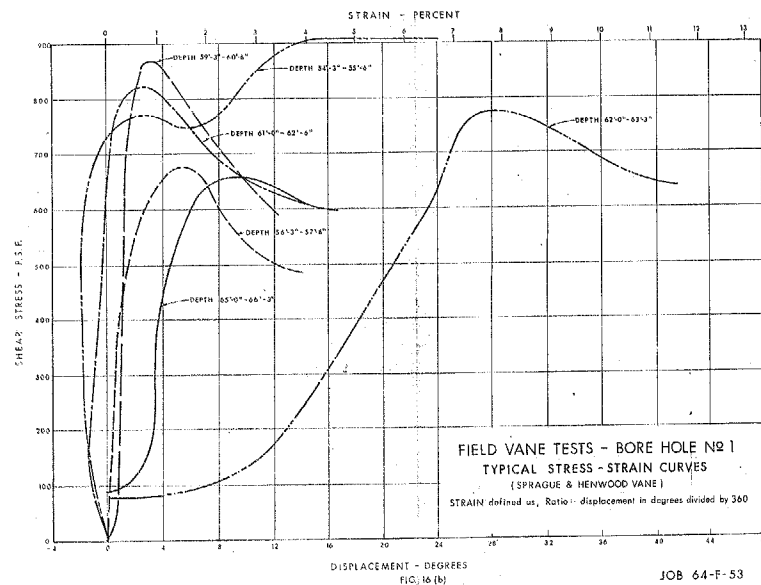


FIG. 15 (b)

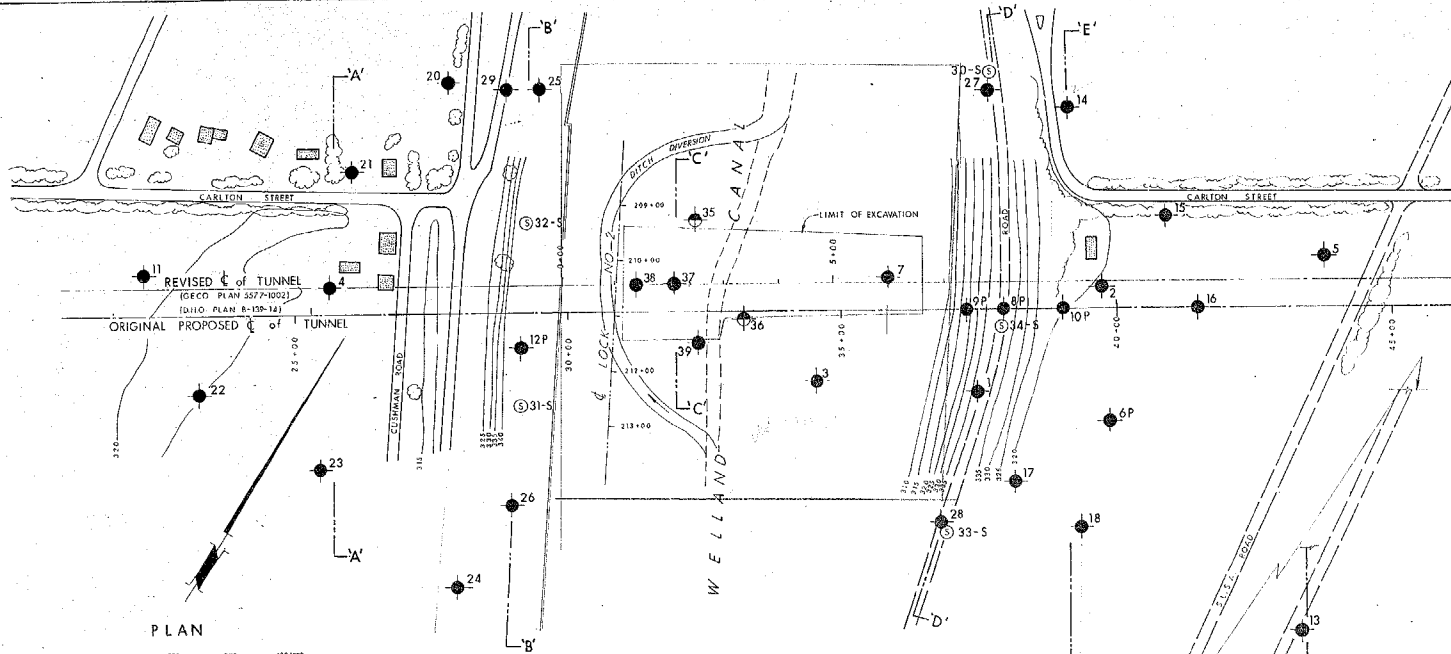
JOB - 64 - F - 53



FIELD VANE TESTS - BORE HOLE NO. 1  
TYPICAL STRESS - STRAIN CURVES  
(SPRAGUE & HENWOOD VANE)  
STRAIN defined as, Ratio - displacement in degrees divided by 360

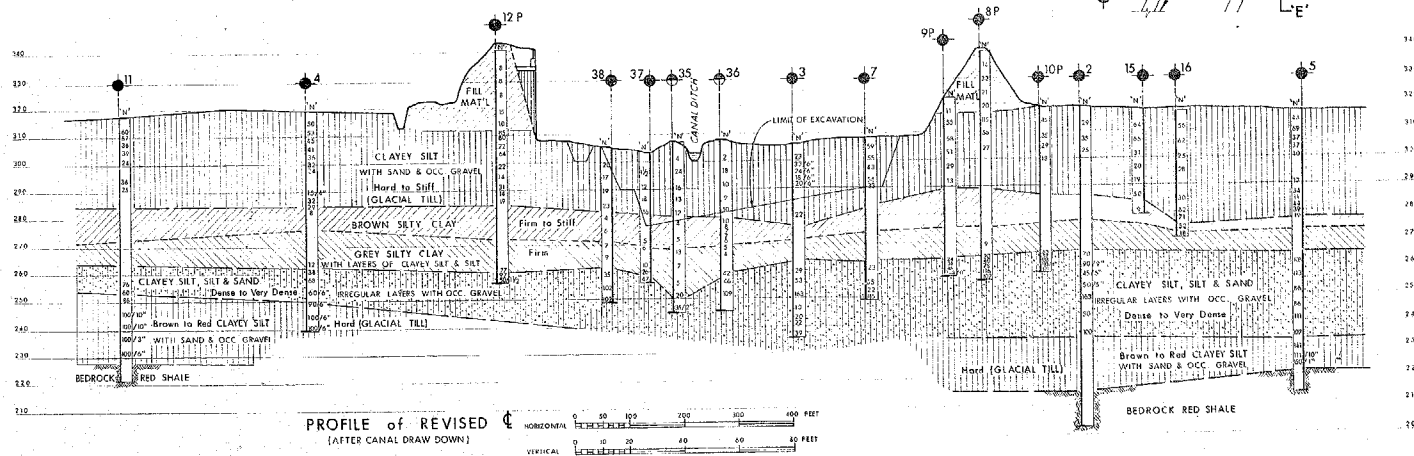
FIG. 16 (b)

JOB 64-F-53



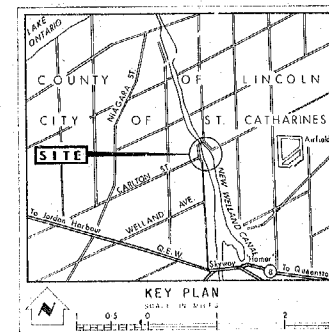
0 50 100 200 300 400 FEET

PLAN



PROFILE of REVISED T  
(AFTER CANAL DRAW DOWN)

HORIZONTAL 0 50 100 200 300 400 FEET  
VERTICAL 0 10 20 30 40 50 60 FEET



LEGEND

- Bore Hole
- Bore & Slope Indicator Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation
- ⑤ Slope Indicator
- P Piezometer

SLOPE INDICATOR DATA

WELL NO.	STATION	OFFSET ORIGINAL	TOP ELEVATION	TIP ELEVATION	LENGTH
30	37+70	430' LT	242.5	270.0	72'-5"
31	29+12	170' RT	343.3	263.2	80'-1"
32	29+22	165' LT	342.0	262.8	79'-2"
33	36+90	100' RT	340.2	238.0	82'-2"
34	37+90	30' RT	340.8	281.8	79'-0"
35	32+35	165' LT	310.0	246.5	63'-5"
36	35+25	15' RT	210.0	246.5	63'-5"

Refer to Dwg 64-F-53C for Bore Hole Data & Sections A-A, B-B, C-C, D-D, E-E

NOTE

The boundaries between soil strata have been established only at bore hole locations. Between bore holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION - FLUORINATION SECTION

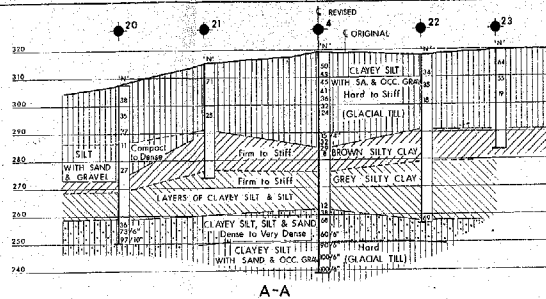
**CARLTON STREET PROPOSED TUNNEL**  
WELLAND CANAL

KING'S HIGHWAY NO. \_\_\_\_\_ DIST NO. 4  
CO. LINCOLN (CITY OF ST. CATHARINES)  
TWP. \_\_\_\_\_ LOT \_\_\_\_\_ CON. \_\_\_\_\_

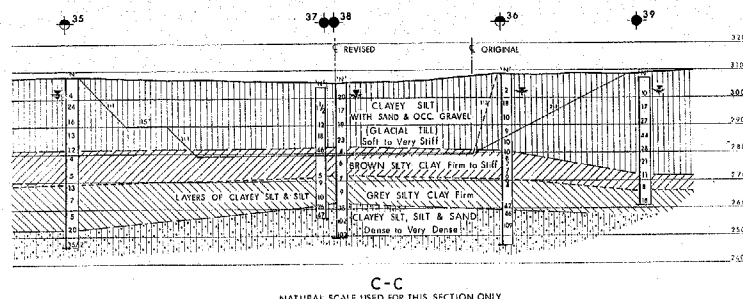
**BORE HOLE LOCATIONS & SOIL STRATA**

DESIGNED BY: \_\_\_\_\_ CHECKED BY: \_\_\_\_\_ W.P. NO. 444-64-B-65, 64-F-53B  
DRAWN BY: \_\_\_\_\_ CHECKED BY: \_\_\_\_\_ JOB NO. 64-F-53  
DATE: FEB 8, 1965 SITE NO. \_\_\_\_\_ DRILLING COMPANY NO. \_\_\_\_\_  
APPROVED BY: \_\_\_\_\_ (SIGNATURE) (PRINT NAME)  
BY: \_\_\_\_\_ (SIGNATURE) (PRINT NAME)

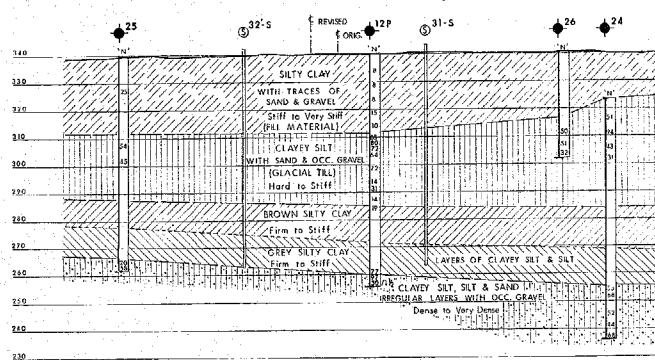




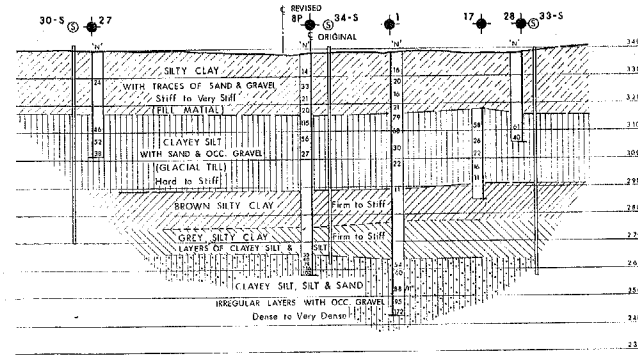
A-A



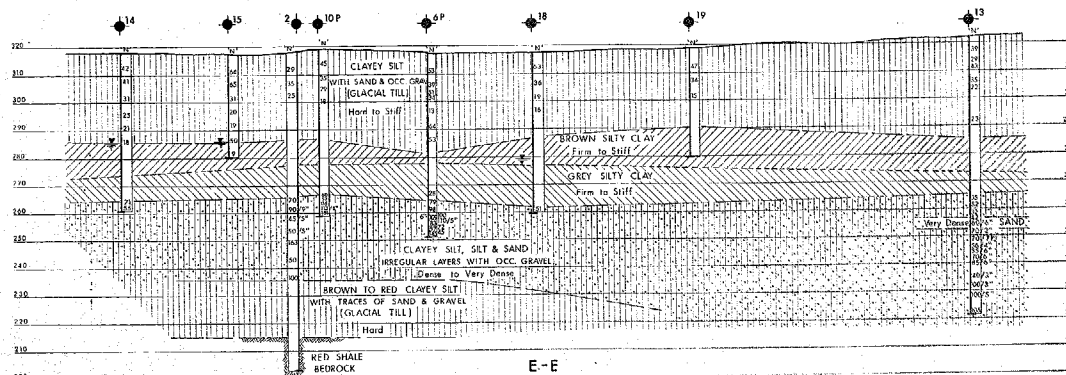
C-C  
NATURAL SCALE USED FOR THIS SECTION ONLY



B-B  
ALONG WEST DIKE



D-D  
ALONG EAST DIKE



E-E

# LEGEND

- Bore Hole
- Bore & Slope Indicator Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation
- ⑤ Slope Indicator

P	Piezometer	GROUND ELEVATION	STATION	OFFSET
1	338-9	37+43	142' RT.	
2	318-6	39+73	41' LT.	
3	306-0	34+51	127' RT.	
4	318-7	28+56	51' LT.	
5	316-7	43+75	00' LT.	
6P	317-48	39+85	250' RT.	
7	307-1	35+86	64' LT.	
6P	358-48	37+92	€	
9P	321-1	37+29	€	
10P	319-5	39+00	€	
11	317-2	22+14	74' LT.	
12P	339-9	29+10	66' RT.	
13	322-6	39+00	1180' RT.	
14	318-5	29+09	363' LT.	
15	317-3	40+84	163' LT.	
16	316-7	41+42	€	
17	317-4	28+13	313' RT.	
18	317-7	39+34	396' RT.	
19	319-3	29+83	683' RT.	
20	307-0	27+78	420' LT.	
21	314-5	26+00	240' LT.	
22	319-4	23+28	145' RT.	
23	319-2	25+45	280' RT.	
24	323-0	27+94	497' RT.	
25	338-9	29+45	405' LT.	
26	340-3	28+94	347' RT.	
27	339-9	37+68	395' LT.	
28	338-4	36+78	293' RT.	
29	324-8	24+45	405' LT.	
30-34	SEE SLOPE INDICATOR DATA (64-F-53B)			
35	308-0	32+35	165' LT.	
36	308-0	33+25	15' RT.	
37	305-7	32+00	32' LT.	
38	303-0	31+27	50' LT.	
39	306-2	32+41	55' RT.	

## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DATE	BY	DESCRIPTION

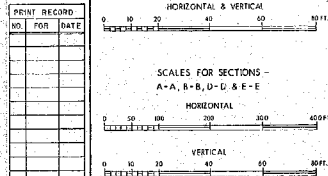
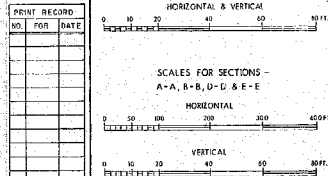
DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

**CARLTON STREET PROPOSED TUNNEL**  
WELLAND CANAL

KING'S HIGHWAY NO. \_\_\_\_\_ DIST. NO. 4  
CO. \_\_\_\_\_ (CITY OF ST. CATHARINES)  
TWP. \_\_\_\_\_ LOT \_\_\_\_\_ CON. \_\_\_\_\_

**SOIL STRATIGRAPHY AT SECTIONS**

SECTION NO. 64-F-53 DRAWING NO. 64-F-53  
DATE MARCH 9, 1965 SITE NO. \_\_\_\_\_ BRIDGE DRAWING NO. \_\_\_\_\_  
APPROVED [Signature] CHECKED [Signature] DATE MARCH 9, 1965





#64-F-53-3

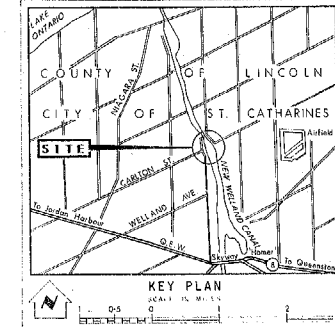
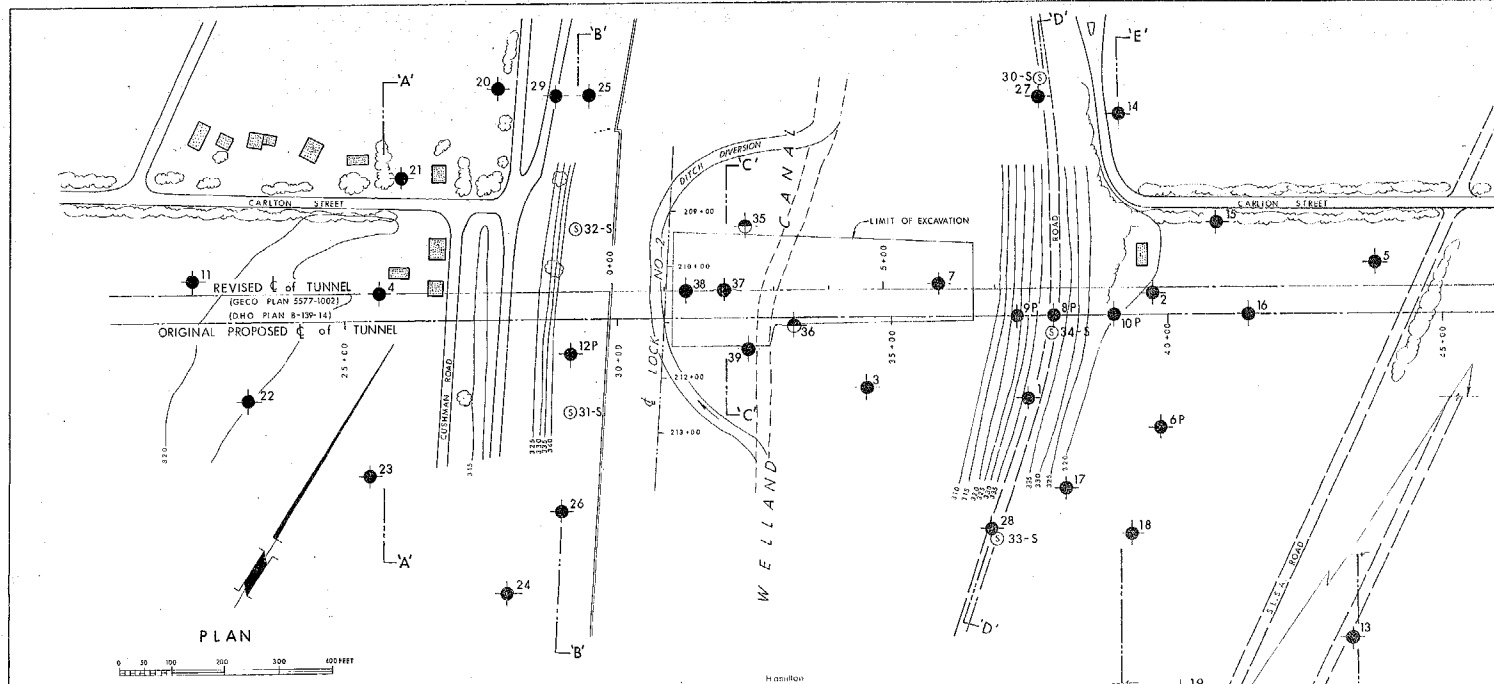
W.P. #444-64

W.P. #444-65

CARLTON ST.

TUNNEL

WELLAND CANAL



LEGEND	
	Bore Hole
	Bore & Slope Indicator Hole
	Bore in Case Presentation Hole
	Water Level Indicated at Time of Field Investigation
	Slope Indicator
	Piezometer

SLOPE INDICATOR DATA					
WELL NO.	STATION	OFFSET ORIGINAL	TOP ELEVATION	TIP ELEVATION	LENGTH
30	37+70	430' LT.	342.5	270.0	72.5'
31	29+12	170' RT.	343.2	263.2	80.0'
32	29+22	165' LT.	342.0	262.8	79.2'
33	36+90	400' RT.	240.2	258.0	82.2'
34	37+90	30' RT.	240.8	261.9	79.0'
35	32+35	165' LT.	319.0	246.5	83.5'
36	33+25	15' RT.	310.0	246.5	63.5'

**NOTE**  
The boundaries between soil strata have been established only of Bore Hole locations. Between Bore Holes, the boundaries are assumed from geological evidence and may be subject to considerable error.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

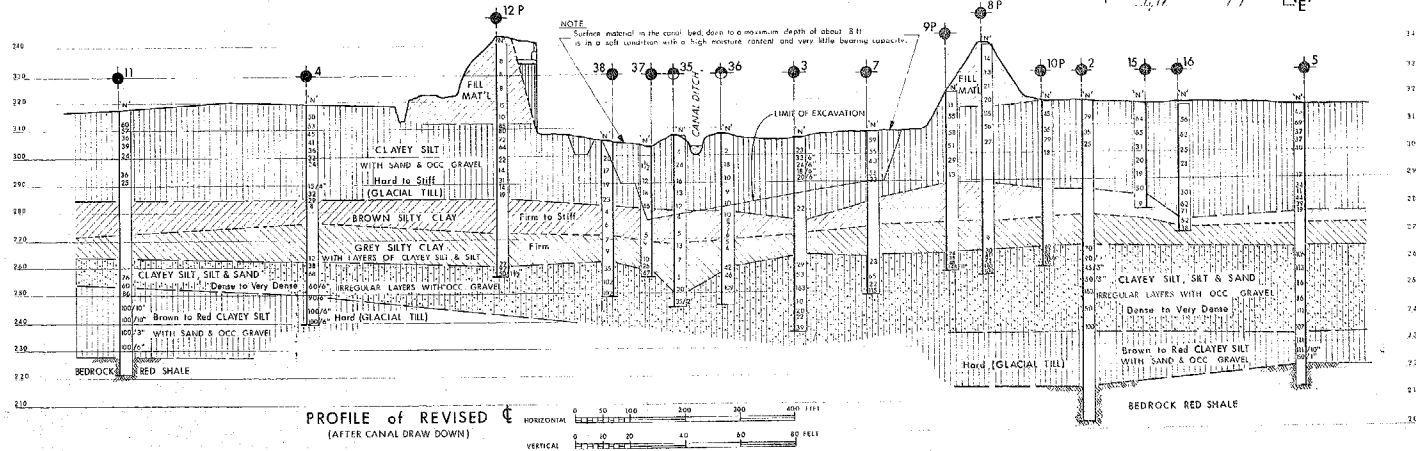
**CARLTON STREET PROPOSED TUNNEL**  
WELLAND CANAL

KING'S HIGHWAY NO. \_\_\_\_\_ DIST. NO. 4  
CO. LINCOLN (CITY OF ST. CATHARINES)  
TWP. \_\_\_\_\_ LOT \_\_\_\_\_ CON. \_\_\_\_\_

**BORE HOLE LOCATIONS & SOIL STRATA**

SUB'D. K.S. CHECKED BY \_\_\_\_\_ W.P. NO. 444-845.65  
DRAWN BY \_\_\_\_\_ JOB NO. 64-F-53  
DATE FEB. 8, 1965 SITE NO. \_\_\_\_\_ BRIDGE DRAWING NO. \_\_\_\_\_  
APPROVED BY \_\_\_\_\_ CONT. NO. \_\_\_\_\_

PRINT RECORD	NO.	FOR	DATE



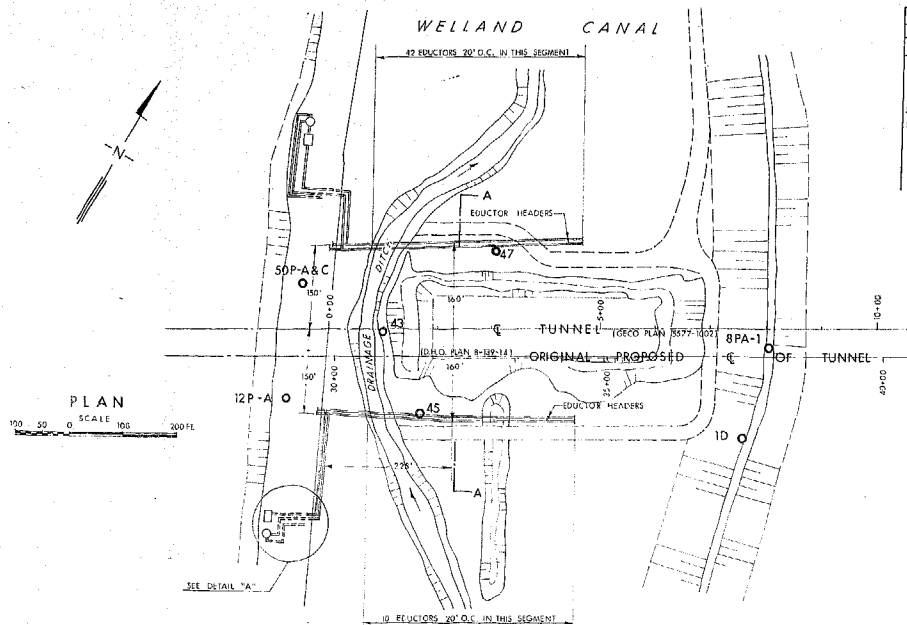
#64-F-53-4

W.P.#444-64

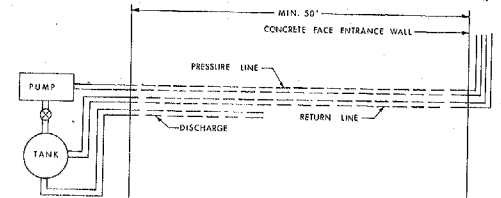
CARLTON ST.

TUNNEL

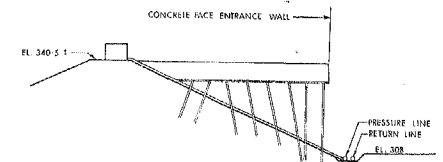
ST. CATHARINES



PIEZOMETER DATA				
NO.	Piezometer	TID ELEVATION	ORIG. STATIONS	OFFSET
ID		260.7	37+43	150' RT.
8PA-1		253.2	37+92	14' LT.
12P-A		252.0	29+10	69' RT.
50P-A		240.9	29+40	132' LT.
50P-C		244.6	29+40	132' LT.
43		257.6	30+87	45' LT.
45		257.0	31+53	105' RT.
47		252.1	33+00	203' LT.

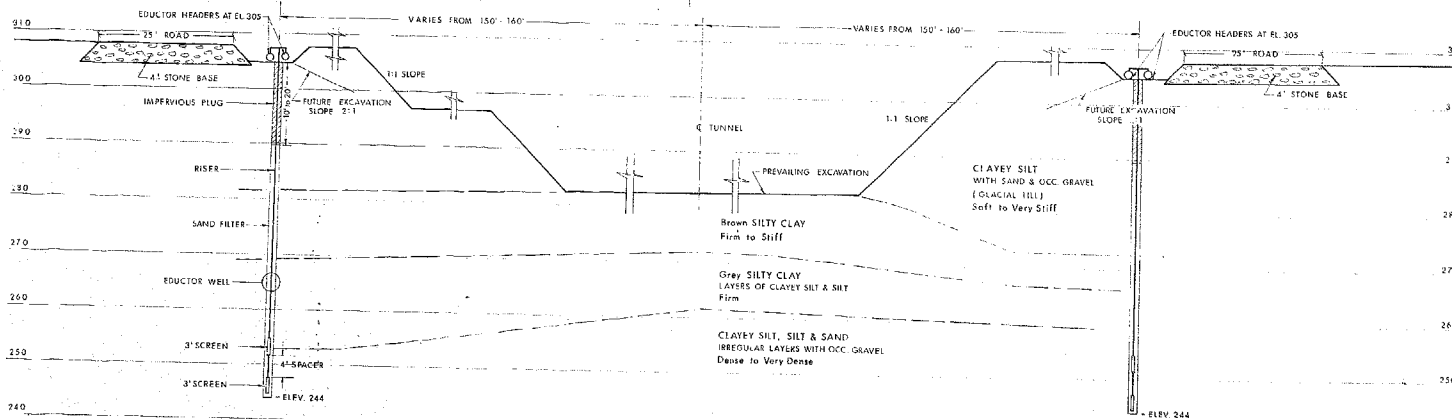


PLAN  
SCALE 1/8" = 1'-0"



ELEVATION  
SCALE 1" = 20"

DETAIL "A"



SECTION A-A  
10 5 0 SCALE 10 20 FT.

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & TESTING DIVISION - FOUNDATION SECTION			
PROPOSED EDUCTOR DEWATERING SYSTEM			
CARLTON STREET TUNNEL			
KING'S HIGHWAY NO. _____		DIST. NO. <u>4</u>	
CO. <u>LINCOLN</u>		ST. <u>CATHARINES</u>	
TWP. _____		LOT _____ CON. _____	
EDUCTOR HEADER & PIEZOMETER LOCATIONS			
SUBMITT. K.S. _____	CHECKED _____	REP. NO. _____	MAP. DRAWING NO. _____
DRAWN S.O. _____	CHECKED _____	JOB NO. <u>64-F-53</u>	<b>64-F-53 D</b>
DATE <u>18 MAY 1966</u>		SHEET NO. _____	
APPROVED <u>[Signature]</u>		SHEET NO. _____	