56-F-228C

HUNTSUILLE BY-PASS

B.A.50B
23-58

RACEY, MacCALLUM AND ASSOCIATES
LIMITED

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REPORT NO: S-500/T-233

310 Odeon Building,
20 Carlton Street,
Toronto, Ontario.

29 March 1956.

Department of Highways of Ontario,
Room 1422, East Block,
Parliament Buildings,
Toronto, Ontario.

C.N.R. Overhead Bridge

Attention: Mr.F.C.Brownridge.

RE: FOUNDATION INVESTIGATION AND STABILITY ANALYSES FOR THE EMBANKMENT APPROACHES, STA.28+00 HUNTSVILLE BY-PASS, ONTARIO.

Dear Sirs:

56-F-228C

We have completed the foundation investigation and stability analyses for the embankment approaches to the railroad overpass at station 28+00 on the proposed Huntsville By-Pass section of Highway No.11; our report is enclosed herewith.

Analyses of the stability of the embankment were first carried out using the total strength of the underlying soil. This method considers only the apparent cohesion of the soil and is sometimes called the ϕ=0 method of analysis. Subsequently, an analysis of the stability was carried out utilising the effective strength of the subsoil. This method permits the evaluation of the increase in strength due to consolidation of the subsoil. The possibility of displacing the soft subsoil with the weight of the fill was also considered.

Our conclusions and recommendations are as follows:-

1. It would be highly impractical to build the embankment by the conventional total strength method of design. The quantity of fill required would be many times that which would be required for a design based on effective strength, and even somewhat more than if the embankment was built successfully by the displacement method. Side slopes of 6:1 proved to be unsafe by this analysis. Therefore, if the embankment was constructed quickly, the side slopes would have to be in excess of 6:1.

29 March 1956.

2. The embankment could be constructed by displacing the subsoil with the fill. In some locations charges may have to be exploded in the subsoil to effect this. This method would allow the work to proceed without delay, but if the embankment should fail by spreading, the quantities of fill required would be increased considerably. Since compaction of the fill would be difficult, if not impossible, and since the ever present danger of trapping soft pockets under parts of the embankment could lead to differential settlement of a fairly high order, this method can be recommended only as a very poor second choice.

3. Analysis by the effective stress method indicates that the embankment can be built in increments over a maximum period of 400 days. This is based on a $1\frac{1}{2}$ to 1 slope for the fill. If this slope is increased, the maximum allowable pore pressures will be increased and the time required decreased. Similarly, if piezometers are installed for the direct measurement of pore pressures, it will probably be found that they dissipate considerably faster than estimated from the consolidation tests. It was considered more economical to use the very low factor of safety of 1.2 and risk some isolated failures, than to use a higher factor of safety and unduly prolong construction. For estimating the quantity of fill required, it is recommended that an allowance be made for the displacement of up to ten feet of the subsoil under the highest part of the embankment.

Mr. B.F.Welsh wrote the enclosed report. I have reviewed it and am in accord with it.

Yours very truly,
RACEY, MacCALLUM AND ASSOCIATES LIMITED

W. Trow

W.A.Trow,P.Eng.

WAT/MD

FOUNDATION INVESTIGATION AND STABILITY ANALYSES FOR THE EMBANKMENT APPROACHES, STATION 28+00, HUNTSVILLE BY-PASS, ONTARIO.

Report No. S-500/T-233

Racey, MacCallum & Associates Ltd.

29 March, 1956

CONTENTS

ENCLOSURES

29 March 1956.

FOUNDATION INVESTIGATION AND STABILITY ANALYSES FOR THE EMBANKMENT APPROACHES, STA.23+00 HUNTSVILLE BY-PASS, ONTARIO.

SCOPE

This report deals with the foundation investigation carried out to determine the stability of a proposed highway embankment south west of Huntsville, Ontario.

I corporated in the report are the results of the drilling work, field tests and laboratory tests on representative and undisturbed samples, along with the analysis of these results. The stability was investigated by both total strength, $\phi=0$ and effective stress analyses.

The foundation conditions over the site were extremely variable and, for this reason, simplyfing assumptions were made wherever the writer felt they were justified. It was felt that a rigorous analysis which accounted for every minor change in the strength of the soil, would tend to leave the reader with the impression that a higher degree of accuracy has been achieved than is actually the case. The recommendations made in this report are based on a comprehensive study of the field investigation and the laboratory test results utilising, where applicable, the most recently published advancements in soil mechanics theory.

LOCATION OF THE SITE AND BOREHOLES

The site is located approximately half a mile south east of Huntsville, Ontario, on the proposed Huntsville By-Pass section of Highway No.11, as shown on enclosure no.1. The area investigated was on both sides of the C.N.R.right-of-way and consists of swampy low lying ground. A small stream flows more or less parallel to, and west of, the centreline on the south side of the railroad. This stream turns sharply west after passing through a culvert under the railroad and crosses the centre line just north of the railroad right-of-way.

Boreholes nos. 1 and 2 were located on the north side of the railroad, on the proposed centre line at stations 30+00 and 31+50 respectively, while boreholes nos. 3 and 4 were located to the south of the railroad on the centre line at stations 25+00 and 22+00 respectively, as shown on enclosure no.2.

THE SUBSOIL CONDITIONS

The soil underlying the site consists of a top layer of very soft dark grey organic clay, varying in thickness from 6.5 feet in hole 1 to 3.0 feet in hole 2. Below this is a layer of soil, ranging from grey silty clay to grey clayey silt, with silt seams and layers up to several inches in thickness. A definite silt layer, approximately 4 feet thick, was detected between elevations 914.9 and 918.9 in hole no.4. The clay layer, as encountered in the boreholes, ranges in thickness from a maximum of 28.5 feet in hole 1, to a minimum of 13.5 feet in hole 4. The above materials are underlain by a layer of loose to medium dense grey silt. The soil profiles are shown in detail on the engineering data sheets (enclosures nos. 3, 4, 5 and 6) and on the centre line profile (enclosure no.9). The water table was established to be at El.931.0 feet. The borings were not taken below the grey silt because our previous investigation for the associated Railway Overpass indicated the presence of dense sand, gravel and boulders at greater depths just above bedrock. These granular deposits would have no influence on the stability of the approach embankments.

The topmost organic clay layer is of little importance. It should be displaced by the weight of the embankment. However, in all of the stability analyses, the fill was assumed to rest directly on the ground surface. This simplyfing assumption is used because it is impossible to accurately estimate the depth to which the natural soil will be displaced. However, the error introduced is small, providing no more than 10 feet of soil is displaced,and is on the side of safety. The error will be offset to some extent by the assumption that the organic layer has the same strength and unit weight as the underlying clay layer.

The second layer, which will be referred to as the clay layer in the following pages, in an effort to simplify the nomenclature, exhibits a variation in strength between the different holes. Holes 2 and 3 show considerably higher shear strengths by the field vane than holes 1 and 4, although the unconfined compression tests failed to show this variation. However, in all holes a noticeable increase in strength is apparent below El.910 feet.

The sensitivity, which was also measured by the field vane, was found to be considerably higher in holes 2 and 4 than in holes 1 and 3. No significant variation in the properties of this layer from hole to hole is noticeable from the Atterberg Limit tests. The deposit appears to be normally loaded, by the consistent manner in which the moisture content was found to be in excess of the liquid limit and by the generally soft nature of the soil, but consolidation tests do not entirely support this belief. The consolidation test on the sample taken from 22½ feet to 24½ feet in hole 1, indicates some preconsolidation at this level. This is borne out by the increase in strength generally found below El.910.0 feet. The preconsolidation of the lower portion of this clay stratum is probably due to a heavier loading, or to dessication at sometime in its past history.

The average unit weights of the clay layer are 112.5, 121, 118 and 113 p.c.f. in holes 1, 2, 3 and 4 respectively. These values are based both on direct measurements and on calculations of the unit weight from moisture content measurements, using an estimated specific gravity of 2.7. For the purposes of calculations, the unit weight of the layer was assumed to be 121.5, as determined in hole 1, and the submerged unit weight is therefore approximately 60 p.c.f.

Definite varving was found in some samples from this clay layer, after they had been split and partially dried in the laboratory, indicating in all probability a lacustrine deposit. The coarser portions of the varved clay and the definite silt layers will probably allow horizontal drainage of the soil and thereby accelerate consolidation as the embankment is constructed. The results of consolidation tests performed on two undisturbed samples from hole 1, taken in this layer and shown on enclosures nos. 7 and 8, indicate for the top part of the layer, a coefficient of consolidation of 0.1 sq.ft. per day for the approximate range of loading anticipated, and a compression index Cc of 0.165. The results of consolidation tests are valid for vertical drainage only, and if horizontal drainage occurs in the soil layer, the actual rate of consolidation will be greater than that estimated from the tests. Therefore, the rate of consolidation, as estimated from consolidation tests, should be regarded in this case as an outside limit and showing only the maximum time required to consolidate.

The layer underlying the clay was composed predominantly of medium-dense grey silt. It was judged, by visual inspection, to be sufficiently permeable to form a lower drainage boundary during the consolidation of the clay layer.

THE DESIGN AND CONSTRUCTION OF THE EMBANKMENT

The problem of designing and constructing the embankment can be subdivided under three headings. Each represents a distinct method of designing or constructing the embankment. These are:- 1. Total Strength Analysis, 2. Displacement Method, and 3. Effective Stress Analysis.

The first is the conventional method of analysing stability problems. Only the apparent cohesion of the clay layer is considered in this method and the angle of internal friction is considered to be nil. This is sometimes referred to as the $\phi=0$ method. The displacement method is a construction technique. The object in this method is to squeeze out the soft clay layer under the weight of the fill. The third method is a design procedure, based on the effective strength of the soil and the pore water pressure. The effective strength of a clay is the true strength as measured by slow drained shear tests.

1. Total Strength Analysis

The centre line profile of the proposed embankment is shown on enclosure no.9. The embankment rises on a vertical curve to a maximum height of 36 feet above the existing ground surface. For purposes of the analysis, the fill was assumed to be a granular material which, in view of the high water table and the anticipated displacement of the subsoil, is the most desirable type of fill. It was also assumed that the fill would be capable of standing on a 1½ horizontal to a 1 vertical slope. Assuming no cohesive strength and an angle of internal friction ∅ approximately equal to the angle of repose of the embankment, ∅ would be equal to 33° - 44'. This assumption is conservative. The angle of internal friction could be considerably higher, depending on the nature of the granular fill and the degree of compaction it was subjected to. Generally speaking, the angle of internal friction is equal to the angle of repose of a granular material only in its loosest state. The unit weight of this fill, when compacted, was assumed to be approximately 130 and 80 p.c.f. in air and water respectively.

The non-uniform soil conditions over the site make it difficult to cover all cases with one analysis. Furthermore, no definite or established pattern in the variability of the strength of the soil over the length of the site could be established, with the possible exception of the immediate vicinity of the railroad right-of-way, where the strength was greater.* It was decided therefore, to base the analysis on an embankment height of 35 feet and assign a cohesion of 200 p.s.f. to the first twenty feet of subsoil and 400 p.s.f. to the remaining portion of the clay layer. These are total strength values and are in line with the average value determined from field vane tests in hole 1. In soft saturated clay soils of this type, it is felt that the field vane yields a more reliable measure of shear strength than does the unconfined compression test. From a preliminary study of the conditions, it was apparent that a slip circle failure would not likely occur in the deeper silt layer and, accordingly, no assumptions of its strength were required.

In the slip circle analysis, a minimum factor of safety of 1.25 was sought, and if any trial circle fell below this value, the proposed design being analysed was discarded. The object of the initial trial, as illustrated in enclosure 10, fig.1, was to determine the severity of the problem. When a value of 0.4 for the factor of safety was determined, it was obvious that a very serious stability problem existed.

An attempt design the embankment, utilising counter balancing side berms 15 feet high, as shown on enclosure 10, fig.2, was analysed and a factor of safety of 0.89 was obtained from the first trial circle. It was subsequently found that the 15 foot high berm was itself unstable. It would therefore be necessary, if this method of design was followed, to have side berms counter balancing side berms, almost ad infinitum. The same effect could be achieved more simply by using flatter side slopes.

Trials were made with side slopes of 4 to 1, 5 to 1, and 6 to 1, as shown on enclosure 11, Figs. 1, 2 and 3, for which the factors of safety were F=0.82, 1.22 and 0.67 respectively. It is interesting to note that in the 4 to 1 and 5 to 1 analyses the circles which were used were not by any means the most critical, as shown by the factor of safety of 0.67 determined with a 6 to 1 slope. A slope greater than 6 to 1 was considered to be economically unreasonable.

The possibility of excavating to 20 feet below the ground surface to remove the comparatively soft soil and replacing it with granular fill, was investigated. Two of several trials of this method are shown on

* See report of Foundation Investigation for the Railway Overpass at Sta.28+00, Huntsville By-Pass, Ontario. 22 February 1956.

enclosure no.12. Although the factors of safety shown here are close to 1, they are not the most critical circles that could be obtained. However, they do illustrate the weakness of this approach. The existing soil would have to be replaced to the 20 foot depth for a considerable distance past the toe of the slope, to prevent a critical circle from passing outside it. From the preliminary investigation, this procedure was obviously uneconomical. The cost of removing the soft material to a depth of 20 feet or more and then backfilling with granular material, over the width that would be required, would be high.

C. Displacement Method

If the embankment is designed on the basis of the conventional total strength analysis, the cost of obtaining a safe embankment will be extremely high. It may be possible to achieve some saving in the quantity of fill required, by using the weight of the embankment to displace the soft clay.

Unfortunately, it is impossible to estimate with any degree of accuracy, the quantity of fill that would be required to build a stable embankment by the displacement method. It is the writer's belief however, that some saving could be achieved if the embankment did not fail during construction. The success of this method depends on the development of a mud wave, either with or without the use of explosives; that is, the embankment would settle vertically more or less intact, displacing the subsoil only directly below it. It is obvious from enclosure no.12 that considerably more than the top 20 feet of the clay will have to be displaced if a stable embankment is to be achieved. It may be necessary in this connection to explode charges of dynamite in the subsoil below the embankment, to facilitate this displacement.

Some of the subsoil will be displaced regardless of how the embankment is built, but under the sudden loading required to displace the clay layer to sufficient depth, there is the danger that the embankment itself may fail by spreading. This is particularly true if any of the silt seams encountered in the clay layer are not continuous. If a general embankment failure by spreading occurred, the quantity of fill required to build up the embankment would be considerably greater than if the embankment was built on the basis of a total strength analysis. Again, there is a strong possibility that soft pockets will be trapped under the fill, giving rise to very substantial differential settlement. Although the displacement method would permit relatively rapid construction of the embankment, the quantity of fill required to displace the subsoil to a sufficient depth and the risk of failure of the embankment, are considered too great to recommend it.

If this method should be followed, it is extremely important to strip off every vestige of the meadow-mat over the full width of the right-of-way, before placing the fill. Cases have been recorded where fills have been supported on a meadow-mat for a considerable time before suddenly failing. The effective stress method of analysis considered under the next heading provides an alternative solution to this construction problem.

3. Effective Stress Analysis

A study of the stability of these embankments, was also carried out using an effective stress analysis*. By utilising the effective strength of the subsoil and taking into account the pore water pressures generated during construction, it was found that the embankment could be built up in increments over a maximum period of 400 days.

From the sliding block analysis, enclosure no.13, and the shear diagram, enclosure no.14, it is possible to establish the pore water pressure at failure as 1380 p.s.f. The clay layer is assumed to have no cohesive strength and an angle of internal friction of 30 degrees; this is in line with results obtained from slow drained triaxial tests on similar soils. But a factor of safety was required and, in selecting it, consideration was given to the economical factors involved. It was felt that it would be justified to work to a low factor of safety and to risk a few failures in weaker areas during construction, rather than to use a higher factor of safety and unduly prolong the period of construction. Therefore, a factor of safety of 1.2 was arbitrarily applied in the shear stress diagram, to yield a maximum permissible pore water pressure of 980 p.s.f.

A coefficient of consolidation of 0.1 sq.ft. per day was obtained from the curve on enclosure no.7, which shows the result of a consolidation test performed on an undisturbed sample from the upper part of the clay layer. From this, the curve shown on enclosure no.15 was constructed to show the rate of consolidation of a 15 foot thick layer of clay under a constant load. The 15 foot thickness was derived by assuming that approximately 10 feet of the compressible layer would be displaced by the weight of the fill. Even if the full 10 feet are not displaced, the estimated rate should still be conservative because the numerous silt layers in the material will permit horizontal drainage and thereby accelerate the consolidation. It is impossible to take these layers into account in the consolidation calculations. The true rate of consolidation will probably be much greater than is shown by the curve.

By assuming an initial period of 80 days and subsequent periods of 40 days for the placing of the fill, and considering that each increment of fill is addded uniformly over the period, the residual pore water pressure at the end of each period can be obtained, as shown in Table I **. The maximum pore pressures are slightly above the permissible 980 p.s.f., however they are not sufficiently high to endanger the embankment. The curves on enclosure no.16, showing the time vs. the safe embankment height and pore water pressures, were constructed using the values shown in Table I. The height of the embankment referred to in the table and on the curves is the height above the existing ground level; it was found that displacement of the subsoil under the embankment by the fill will tend to increase the factor of safety, but since the amount of displacement could not be accurately estimated, it was neglected.

* A.W.Skempton and A.W.Bishop "Gain in stability due to pore pressure dissipation in a soft clay" Fifth International Commission on Large Dams 1955.
** Karl Terzaghi "Theoretical Soil Mechanics" Page 287.

TABLE I

TIME DAYS (1)	HEIGHT OF FILL (2)	TOTAL WEIGHT OF FILL, P.S.F. (3)	TOTAL PORE PRESSURE DISSIPATED, P.S.F. AFTER TIME IN (1) (4)	NET PORE PRESSURE P.S.F. (5)
80	8	955	287	668
120	12	1375	494	881
160	15	1656	702	954
200	18	1913	930	983
240	21	2155	1142	1013
280	24	2363	1363	1000
320	27	2545	1542	1003
360	30	2705	1744	961
400	35	2968	1983	985

The embankment could be constructed at the rate shown on enclosure no.16, but due to horizontal drainage, the pore pressures should dissipate more rapidly than estimated from the laboratory tests. Therefore, the estimate of 400 days should be regarded as a maximum.

If piezometers * are installed to obtain direct measurements of the pore pressure, it would be possible to control the rate of loading, to keep the pore pressure below the critical value while building the embankment at the maximum possible rate.

As a check on the sliding block analysis, the slip circle shown on enclosure no.17, was analysed by the effective stress method, as proposed by Bishop **, using a factor of safety of 1.2 and solving for the pore pressure as shown in appendix A. The permissible pore pressure obtained by this method was 924 p.s.f. and serves as a check on the value obtained from the sliding block analysis.

The disadvantage of this method of constructing the embankment is the length of time it will take. This is particularly true in the later stages, where the volume for each increment of height will be comparatively small. The time could probably be reduced by increasing the side slopes to 2:1 or more and thereby increasing the maximum permissible pore pressure. Settlement of the embankment would be approximately 2.0 feet, based on an average compression index as obtained from the test results shown on enclosures nos. 7 and 8, of 0.35, and a compressible layer approximately 15 feet thick. However, approximately 65 percent of this settlement or 16 inches, will occur during construction.

* A. Casagrande - Soil Mechanics in the Design and Construction of the Logan Airport. Journal Boston Society of Civil Engineers, April 1949, pp.192-222.

** A.A.Bishop - The use of the Slip Circle in the Stability Analysis of Slopes. Geotechnique, March 1955.

29 March 1956.

The main advantages as opposed to the displacement method, are that it provides a scientific method of constructing the embankment using much less fill. Also, the fill can be compacted to the degree desired and borrow quantities can be estimated with a higher degree of accuracy than can be done by the displacement method.

CONCLUSIONS AND RECOMMENDATIONS

1. It would be highly impractical to build the embankment by the conventional total strength method of design. The quantity of fill required would be many times that which would be required for a design based on effective strength, and even somewhat more than if the embankment was built successfully by the displacement method. Side slopes of 6:1 proved to be unsafe by this analysis. Therefore, if the embankment was constructed quickly, the side slopes would have to be in excess of 6:1.

2. The embankment could be constructed by displacing the subsoil with the fill. In some locations charges may have to be exploded in the subsoil to effect this. This method would allow the work to proceed without delay, but if the embankment should fail by spreading, the quantities of fill required would be increased considerably. Since compaction of the fill would be difficult, if not impossible, and since the ever present danger of trapping soft pockets under parts of the embankment could lead to differential settlement of a fairly high order, this method can be recommended only as a very poor second choice.

3. Analysis by the effective stress method indicates that the embankment can be built in increments over a maximum period of 400 days. This is based on a $1\frac{1}{2}$ to 1 slope for the fill. If this slope is increased, the maximum allowable pore pressures will be increased and the time required decreased. Similarly, if piezometers are installed for the direct measurement of pore pressures, it will probably be found that they dissipate considerably faster than estimated from the consolidation tests. It was considered more economical to use the very low factor of safety of 1.2 and risk some isolated failures, than to use a higher factor of safety and unduly prolong construction. For estimating the quantity of fill required, it is recommended that an allowance be made for the displacement of up to ten feet of the subsoil under the highest part of the embankment.

B.F.Welsh, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
B. F. WELSH
PROVINCE OF ONTARIO

BFW/MD

Original and 1 copy - Dept. of Highways of Ontario, Attention: Mr.F.C.Brownridge.
c.c's. 1 - Dept. of Highways of Ontario, Attention: Mr.H.C.Bernier, Huntsville.
1 - Racey, MacCallum and Associates Ltd., Montreal.
2 - Soils Engineers.

APPENDIX A

SLIP CIRCLE ANALYSIS BY BISHOP'S METHOD

(SEE ENCL.NO.17)

SLICE	W	α	SINα	W SINα	(1) $W(1-\bar{B})\tan\phi'$	Sec α	Tanα	(2) $\dfrac{\sec\alpha}{1+\dfrac{\tan\phi'\tan\alpha}{F}}$ F = 1.2	(1) x (2) F = 1.2
1	41,000	52	.785	32,200	27,300	1.62	1.28	.922	25,200
2	48,500	42	.669	32,400	39,000$(1-\bar{B})$	1.35	0.90	.941	36,700 $(1-\bar{B})$
3	46,800	29	.485	22,700	27,100$(1-\bar{B})$	1.14	0.554	.901	24,400 $(1-\bar{B})$
4	40,200	17	.282	11,300	23,200$(1-\bar{B})$	1.045	0.292	.946	22,000 $(1-\bar{B})$
5	35,500	6	.105	3,700	20,500$(1-\bar{B})$	1.008	0.145	.943	19,350 $(1-\bar{B})$
6	26,500	-6	-.105	-2,800	15,300$(1-\bar{B})$	1.008	-0.145	1.081	16,390 $(1-\bar{B})$
7	11,800	-18	-.309	-3,600	6,800$(1-\bar{B})$	1.05	-0.309	1.232	8,250 $(1-\bar{B})$
8	7,800	-30	-.500	-3,900	4,500$(1-\bar{B})$	1.155	-0.500	1.521	6,840 $(1-\bar{B})$
9	3,000	-42	-.669	-2,000	1,700$(1-\bar{B})$	1.35	-0.670	1.980	3,370 $(1-\bar{B})$
	261,100			90,000					162,500 - 137,300$\bar{B}$

The length of the arc L = 95.1 feet

Total pore pressure developed = $\dfrac{220,100}{L} = \dfrac{220,100}{95.1} = 2.310$

$$\text{Now } F = \frac{W(1-\bar{B})\tan\phi'\left\{\dfrac{\sec\alpha}{1+\dfrac{\tan\phi'\tan\alpha}{F}}\right\}}{W\sin\alpha}$$

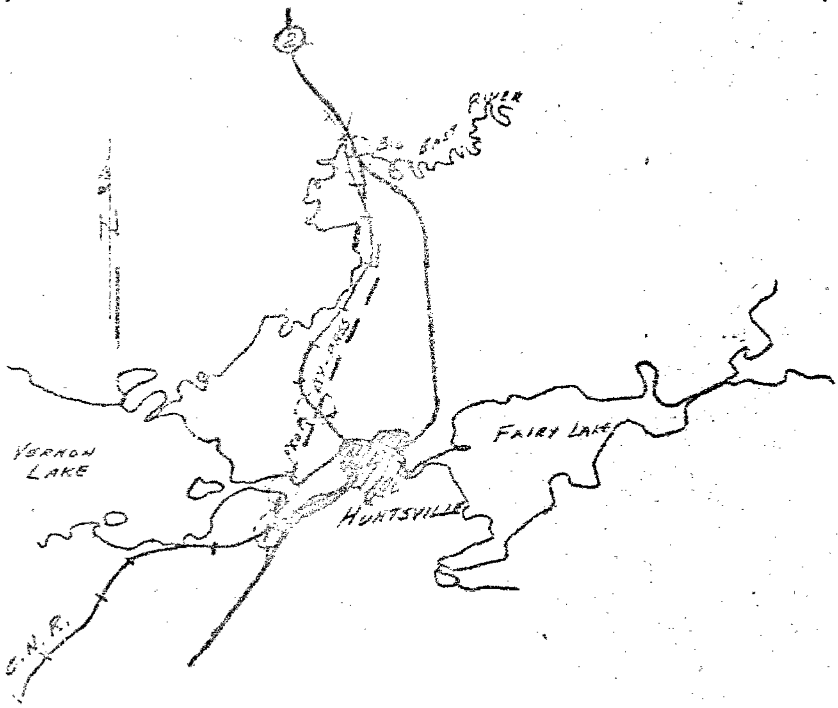
Therefore where F = 1.2 we have $1.2 = \dfrac{162,500 - 137,300\,\bar{B}}{90,000}$

and solving for the fraction of pore pressure permissible $\bar{B} = 0.4$
and the permissible pore pressure = 0.4 x 2,310 = 924

ENCL. No. 1

PREP. & DRAWN BY
B.F.W.

S-500-501/55/T-170, T-192, T-193, T-194 & S-500/55/3

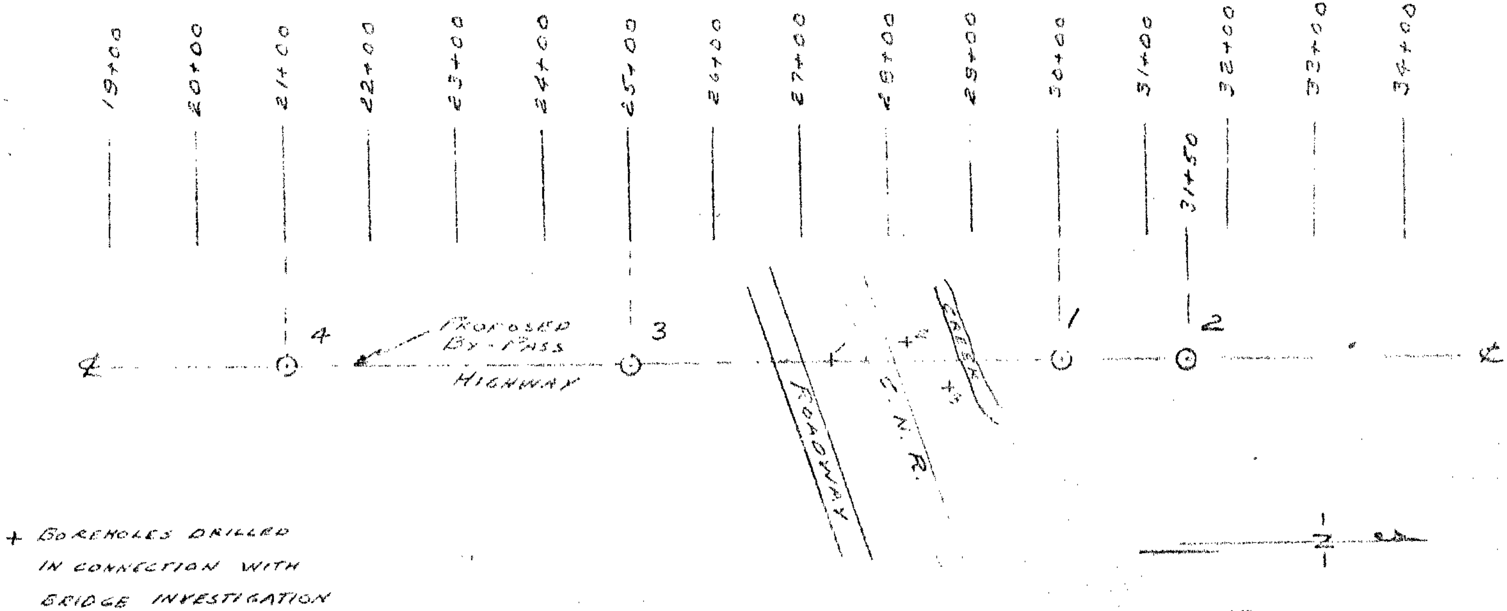


LOCATION PLAN

SCALE 1 IN = 2 MI.

RACEY McCALLUM & ASSOCS. LTD.

NOV. 8, 1955



LOCATION OF BOREHOLES

SCALE 1 IN. = 200 FT.

Order No. [illegible]

Enclosure No. 3

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

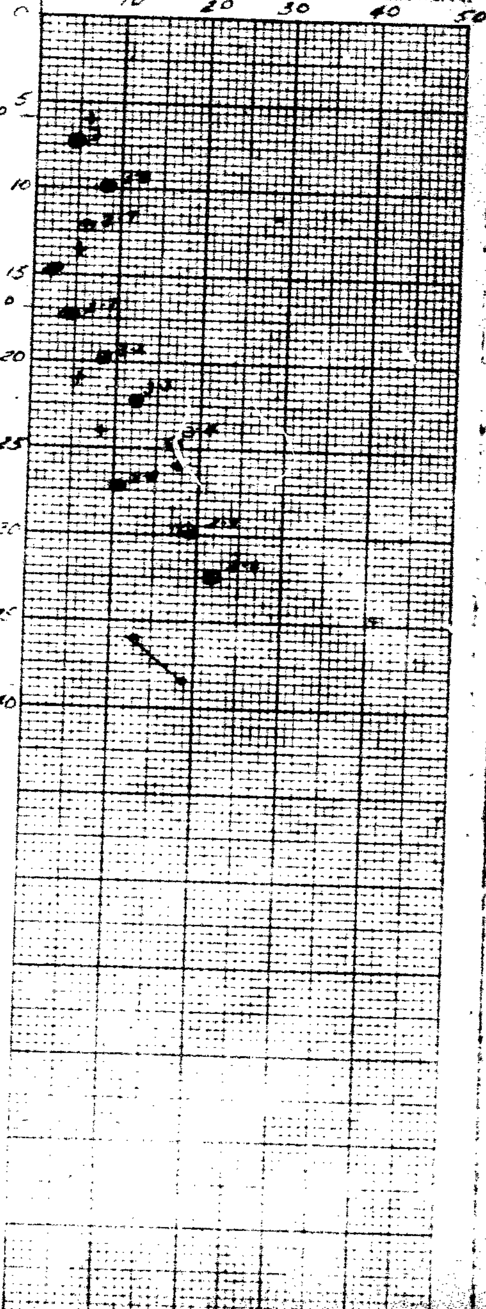
Project: F R OVERPASS APPROACHES - HUNTSVILLE BY-PASS
Location: [illegible] 1/2 MI S E OF HUNTSVILLE, ONT
Hole Location AS SHOWN ON ATTACHED PLAN
Hole Elevation and Datum: 931.0' ([illegible])
Field Work Begun FEBRUARY 2, Ended FEBRUARY 7 1956

Field Supervision: C. O.
Driller: L. [illegible]
Prep.: B. F. W.
Checked: W. T.
Date: 9/2/56

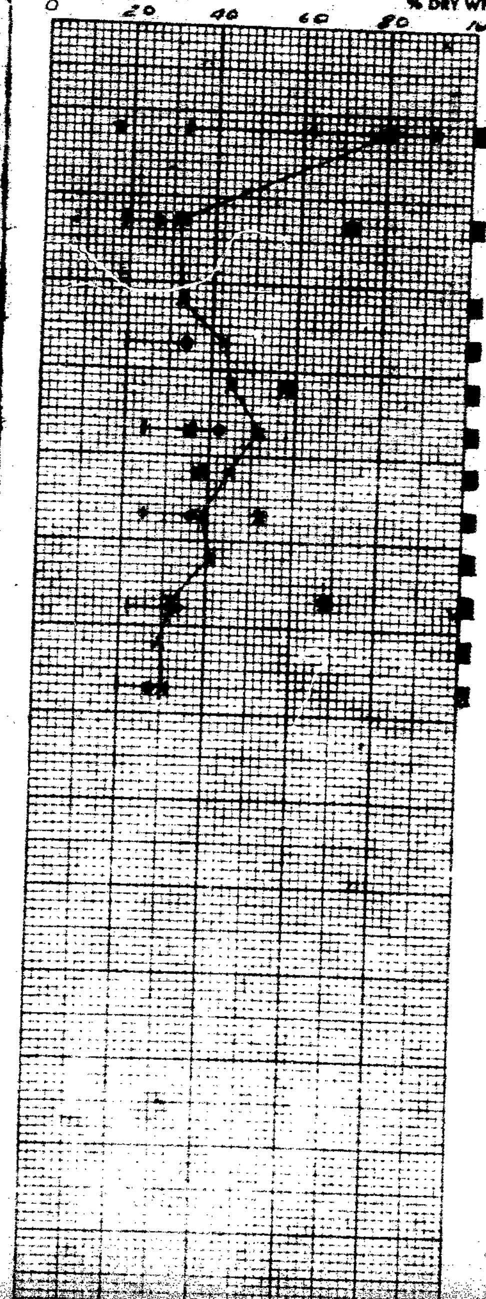
LEGEND
Sampling Method
2" Dia. split tube
2" Shelby tube
Penetration Resistance
2" Split tube
2" Dia. Cone
Casing
Strength
Unconfined compression +
Vane test and sensitivity
Consistency
Natural moisture
Liquid limit
Plastic limit
Natural Unit Weight

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	GROUND SURFACE	931.0	0
	VERY SOFT DARK GREY TO BLACK ORGANIC CLAY	925.0	6.0
	SOFT GREY SILTY CLAY ALTERNATING WITH SOFT GREY CLAYEY SILT AND SOME SILT LAYERS AND SAND [illegible]	914.0	17.0
	MEDIUM STIFF GREY SILTY CLAY WITH SOME SILT SEAMS	896.0	35
	[illegible] DENSE [illegible] MEDIUM CLAY [illegible]		
	END OF BOREHOLE	892.0	39

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY AND UNIT WEIGHT



SAMPLE NO.
TW 1
TW 2
TW 3
TW 4
TW 5
TW 6
TW 7
TW 8
TW 9
TW 10
SS 11
SS 12

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

Order No. [illegible] Enclosure No. 4

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Project: [illegible] OVERPASS APPROACHES HUNTSVILLE BY-PASS Field Supervision: [illegible]
Location: [illegible] ½ MI S.E. OF HUNTSVILLE, ONT. Driller: L. BELL
Hole Location: AS SHOWN ON ATTACHED PLAN Prep.: B.F.W.
Hole Elevation and Datum: 931.6 (C.H.O.) Checked: W.T.
Field Work Begun: FEB. 7 Ended: FEB. 8 Date: 29/2/56

LEGEND

Sampling method
2" Split spoon tube
2" Shelby tube

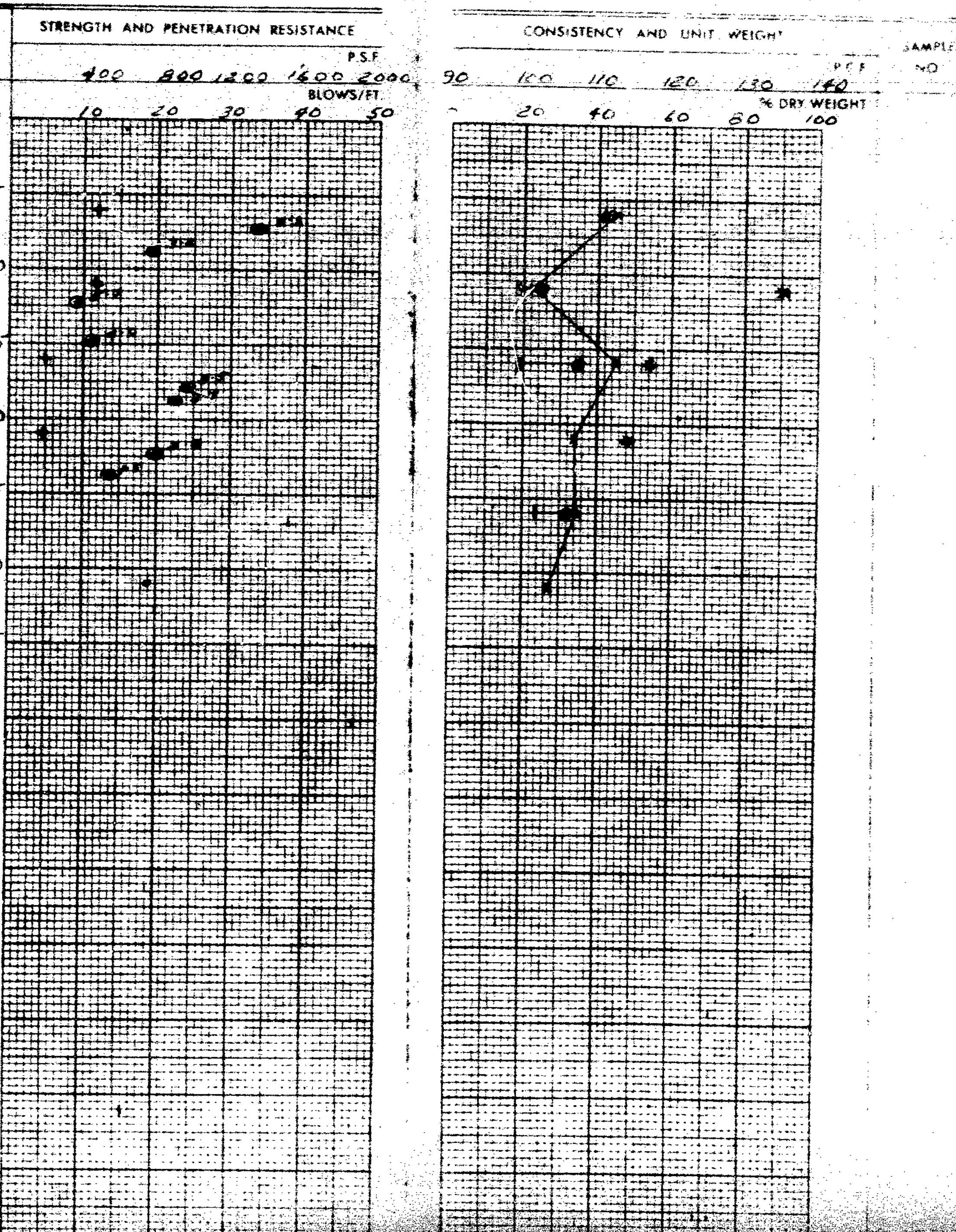
Penetration Resistance
2" Split tube
2" Dia. cone
Casing

Strength
Unconfined compression
Vane test

Consistency
Natural moisture
Liquid limit
Plastic limit

Natural Unit Weight

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	GROUND SURFACE	931.6	0
	VERY SOFT DARK GREY ORGANIC CLAY	927.6	4.0
	MEDIUM STIFF TO SOFT GREY SILTY CLAY AND CLAYEY SILT WITH SILT SEAMS AND SAND PARTINGS	906.1	25.5
	MEDIUM DENSE GREY SILT		
	END OF BOREHOLE	900.1	31.5



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

Order No. S 500/56/T-239

Enclosure No. 5

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

LEGEND

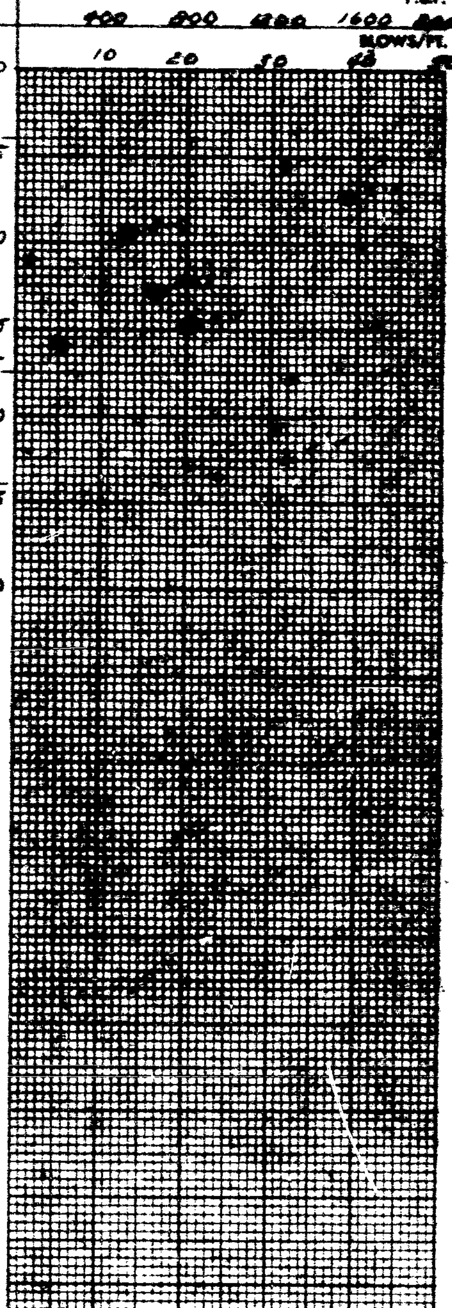
- Sampling Method
  - 2" Dia split tube
  - 2 Shelby tube
- Penetration Resistance
  - 2" Split tube
  - 2" Dia Cone
  - Casing
- Strength
  - Unconfined compression
  - Vane test and sensitivity
- Consistency
  - Natural moisture
  - Liquid limit
  - Plastic limit
- Natural Unit Weight

Project: R.R. OVERPASS APPROACHS - HUNTSVILLE BY-PASS
Field Supervision: C.O.
Location: APPR. 1/2 MI. S.E. OF HUNTSVILLE, ONT.
Driller: L. BELLEY
Hole Location AS SHOWN ON ATTACHED PLAN
Prep.: B.F.W.
Hole Elevation and Datum: 932.0
Checked: W.T.
Field Work Begun FEB. 9 Ended FEB. 10
Date: 1/3/56

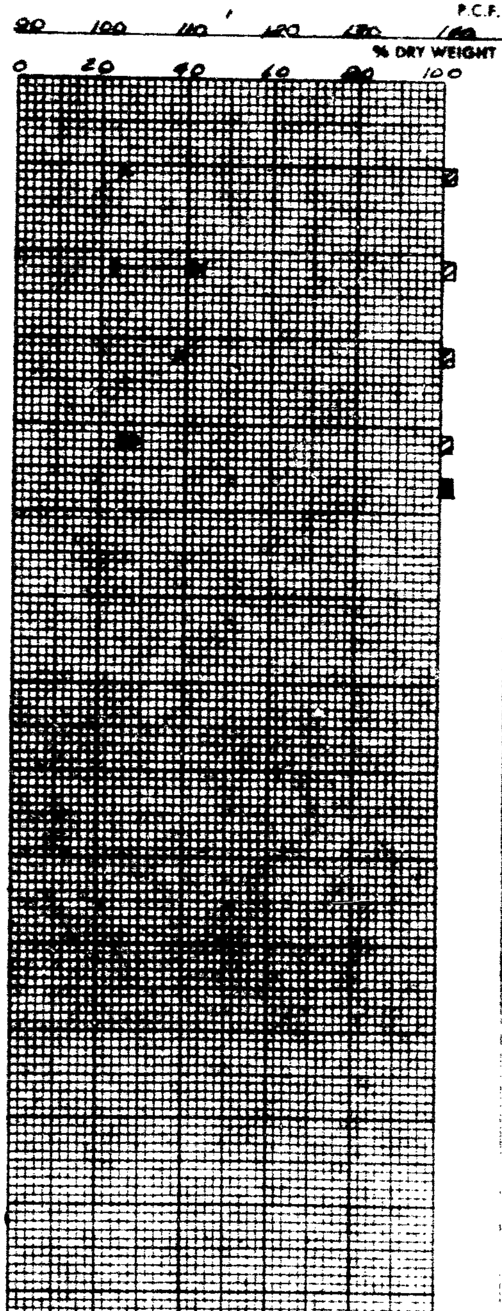
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE NO.
	GROUND SURFACE	932.0	0	
	DARK GREY ORGANIC CLAY WITH DECAYED VEGETATION VERY SOFT		4.0	
	MEDIUM-STIFF GREY SILTY CLAY AND CLAYEY SILT WITH SILT LAYERS			TW1, TW2, TW3
	MEDIUM-DENSE GREY FINE TO MEDIUM SILT	914.5	17.5	TW4, SS LAST
	END OF BOREHOLE			

Depth scale (feet): 0, 5, 10, 15, 20, 25, 30

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY AND UNIT WEIGHT



DE... IN NEGATIVE DUE TO ... OF ORIGINAL DOCUMENT

Order No. S-SQA/55/T-233

Enclosure No. 6

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 4

Project: R R Overpass Approachs - Huntsville By Pass
Location: Appr ½ mi S.E of Huntsville Ont
Hole Location As shown on attached plan
Hole Elevation and Datum: 930.9 (D.H.O.)
Field Work Begun Feb. 11 Ended Feb 12

Field Supervision: C. O.
Driller: L Belley
Prep.: B. F. W.
Checked: W. T.
Date: 1/3/56

LEGEND

Sampling Method
- 2" Dia split tube
- 2" Shelby tube

Penetration Resistance
- 2" Split tube
- 2" Dia. Cone
- Casing

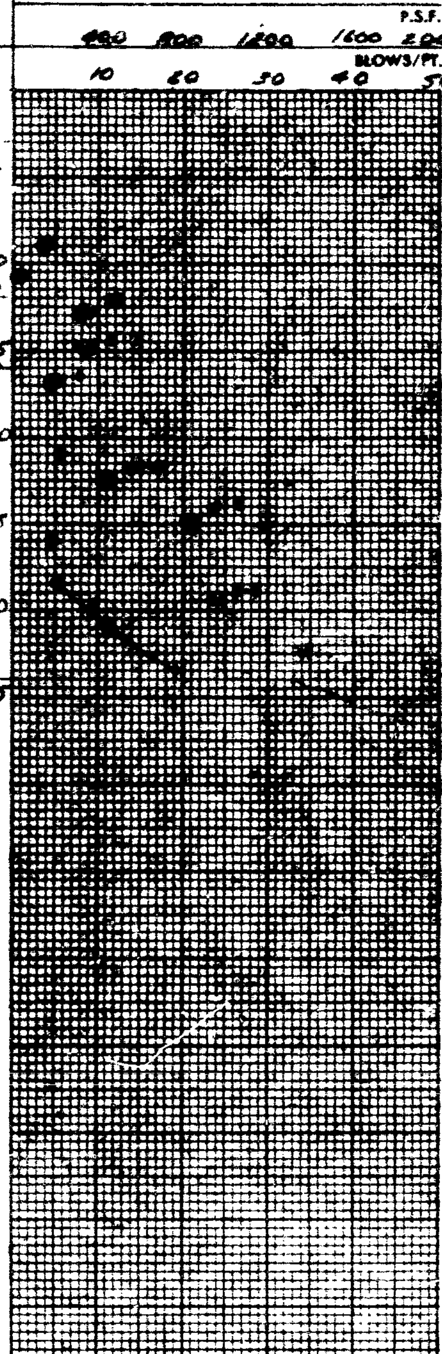
Strength
- Unconfined compression
- Vane test and sensitivity

Consistency
- Natural moisture
- Liquid limit
- Plastic limit

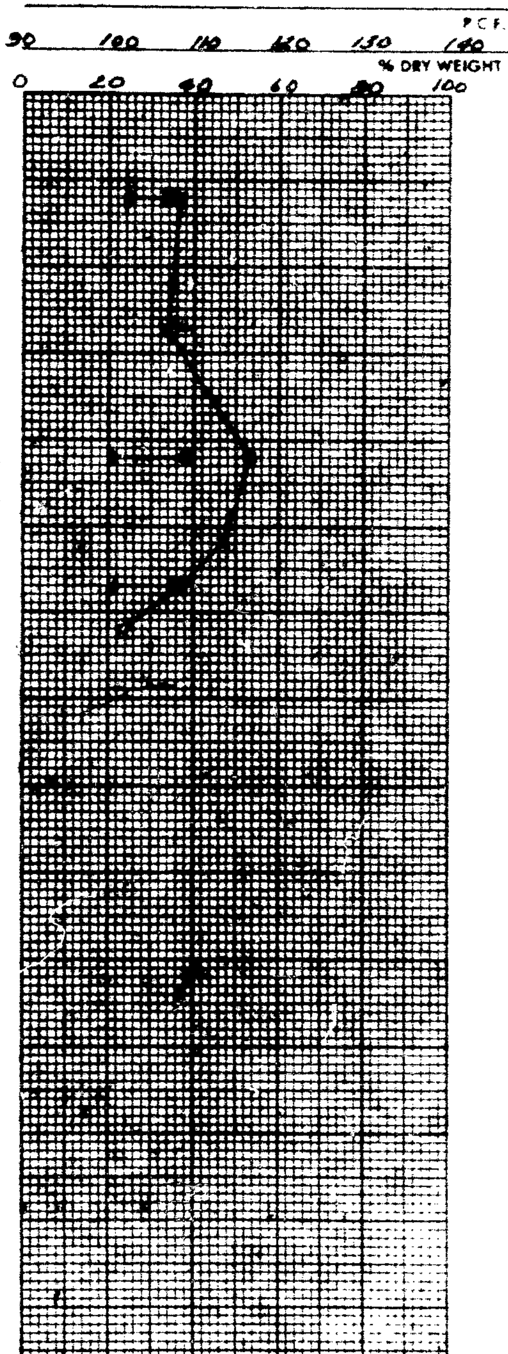
Natural Unit Weight

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground Surface	930.9	0
	Dark Brown Organic clay	927.9	3.0
	Soft grey clayey fine silt	918.9	12.0
	Loose grey fine to medium silt	914.9	16
	Grey silty clay and clayey silt with silt seems medium-stiff to stiff	900.9	30
	Loose to medium dense grey silt — End of Borehole	896.9	34.0

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY AND UNIT WEIGHT

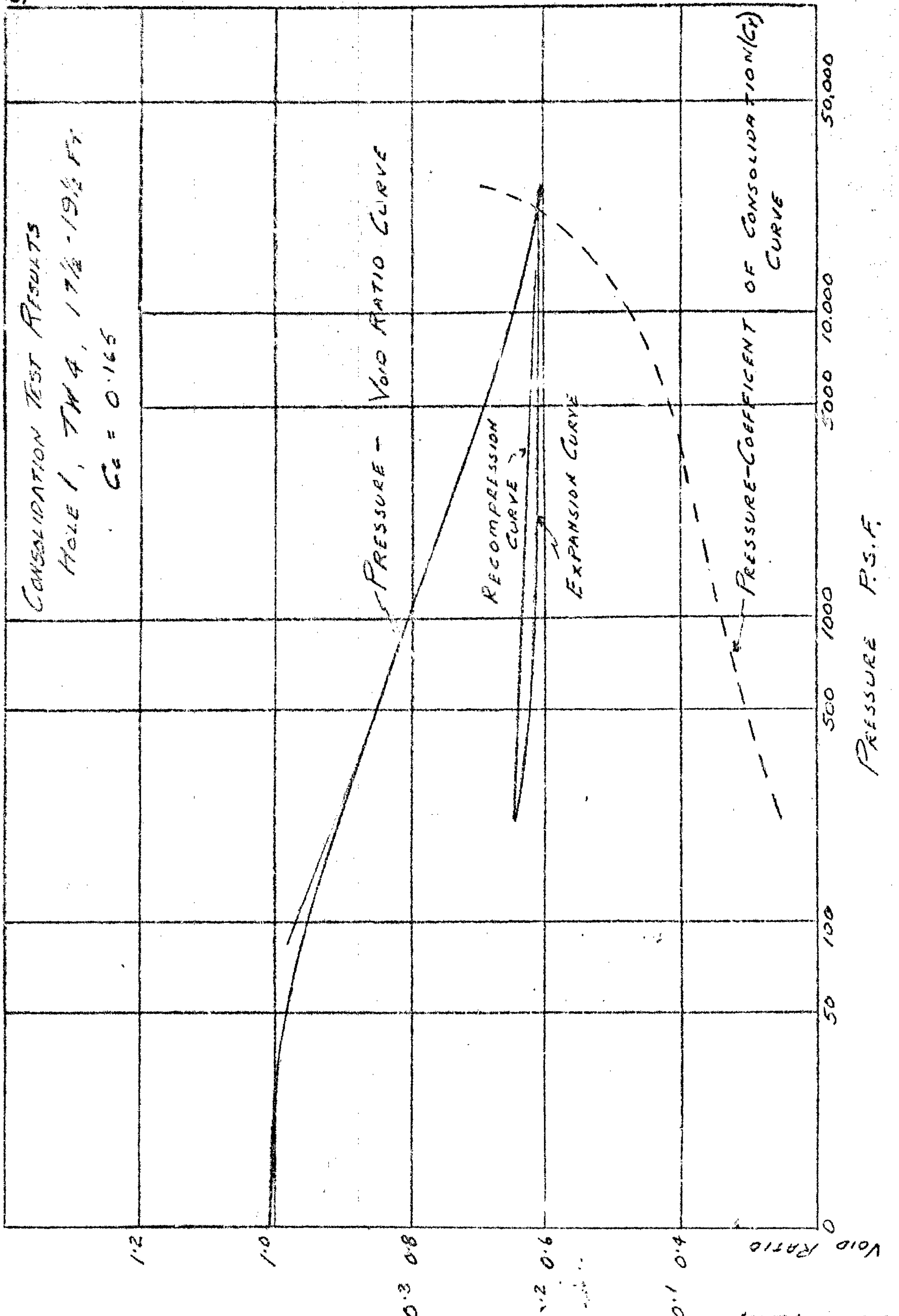


SAMPLE NO

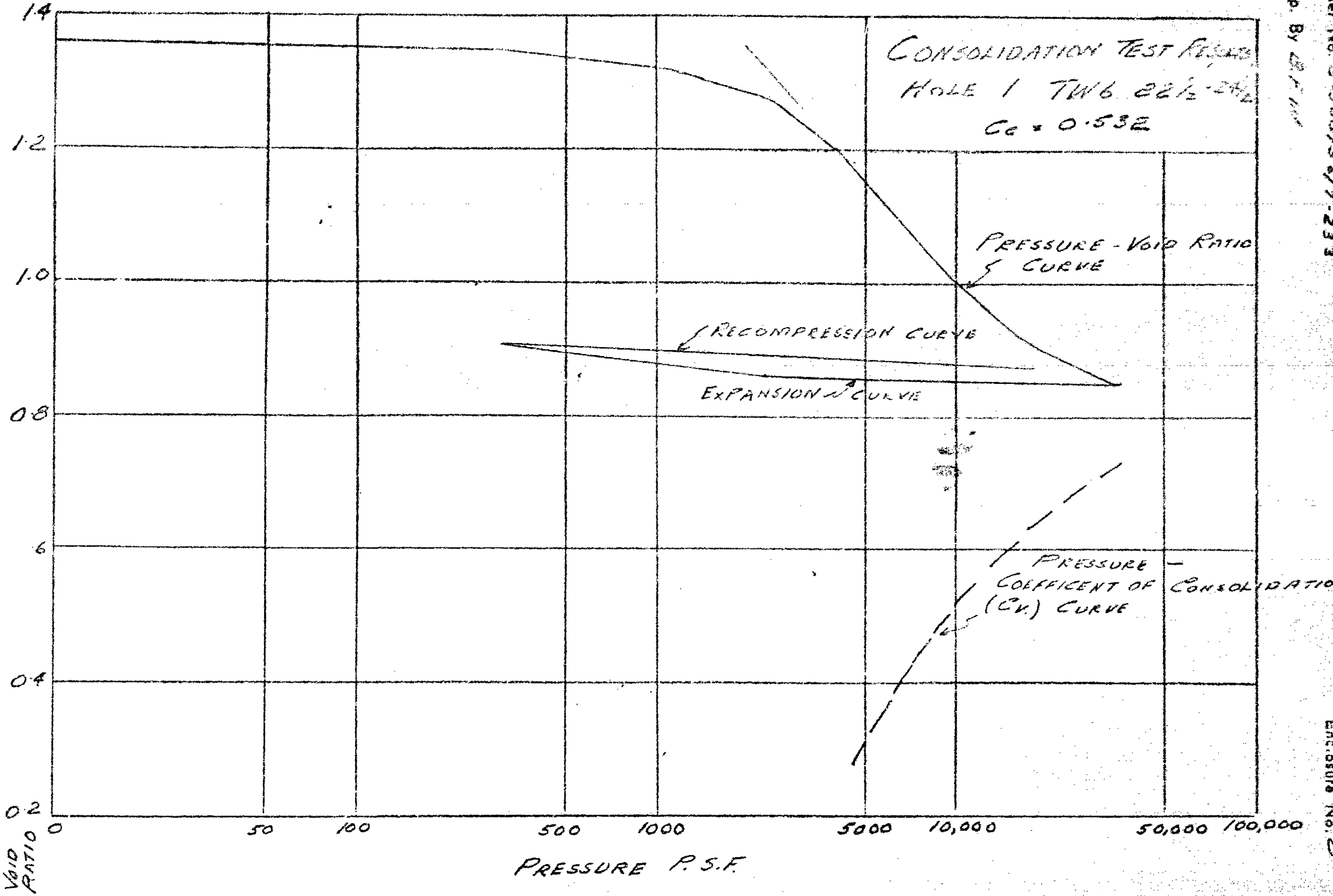
Order No. 5-E00/SC/T-233

Enclosure No. 7

Prep. By B.F.W.



Racey, MacCallum & Associates Ltd.



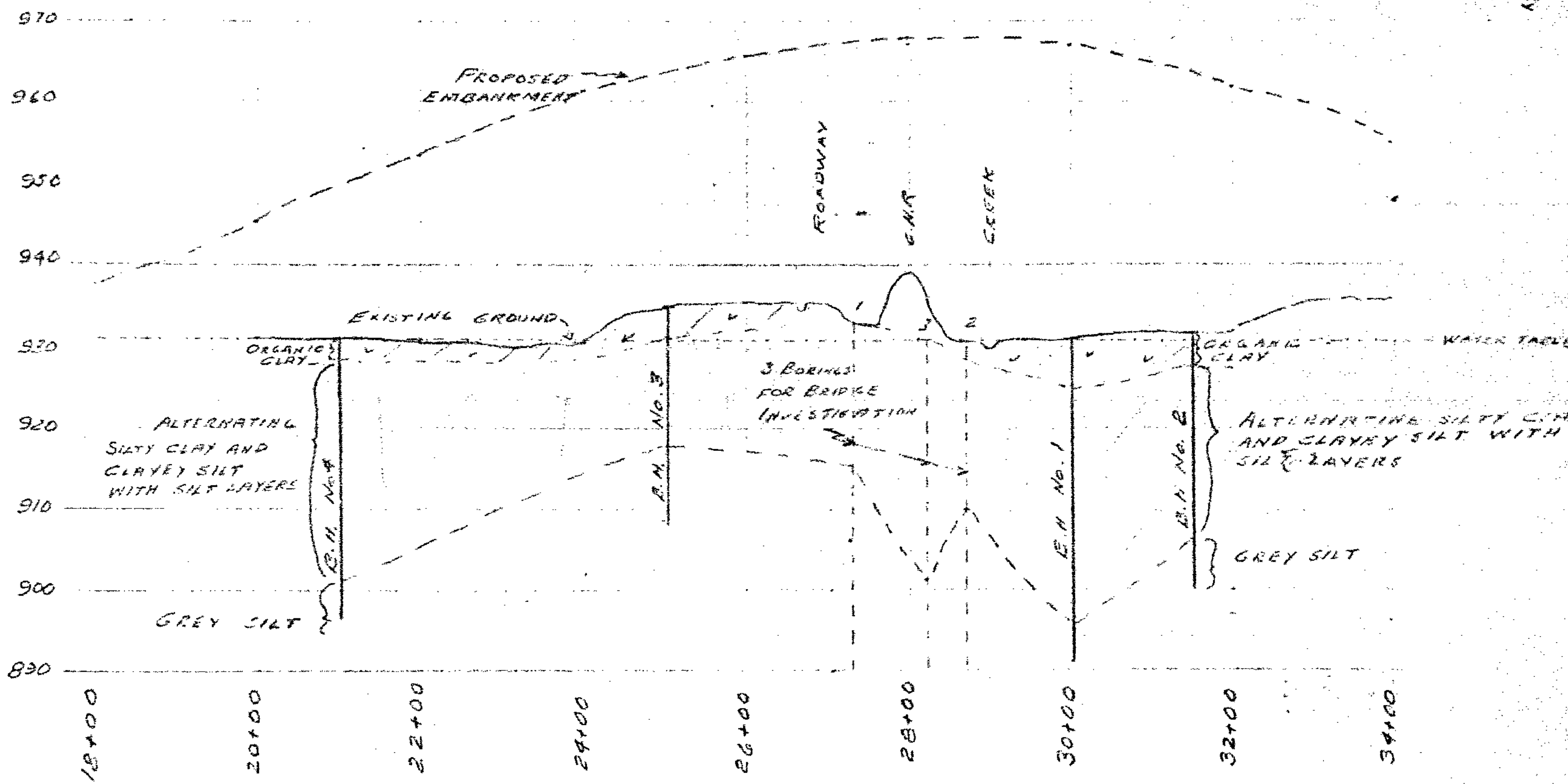
Order No. 5-500/56/T-233
Prep. By

Enclosure No. 8

Racey, MacCallum & Associates Ltd.

Order No. S-500/56/T.233

Prep. By



℄ PROFILE AT SITE

SCALES: 1" = 20' VERT.
1" = 200' HORIZ.

Enclosure No. 9

Prep. By R.F.W.

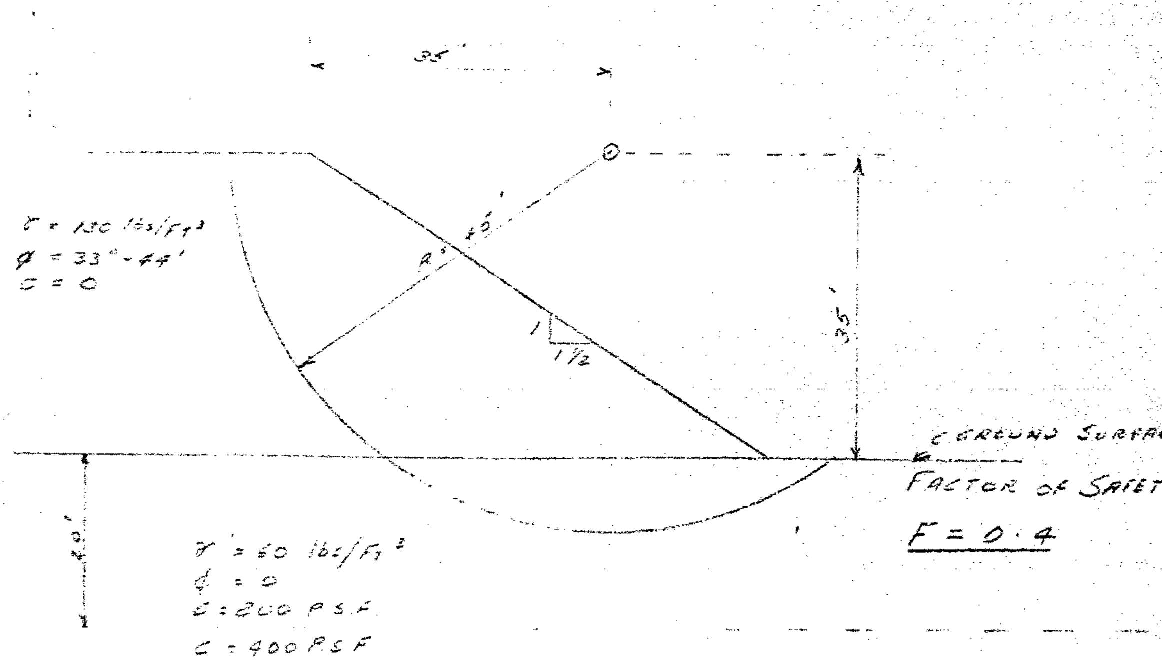


FIG. 1

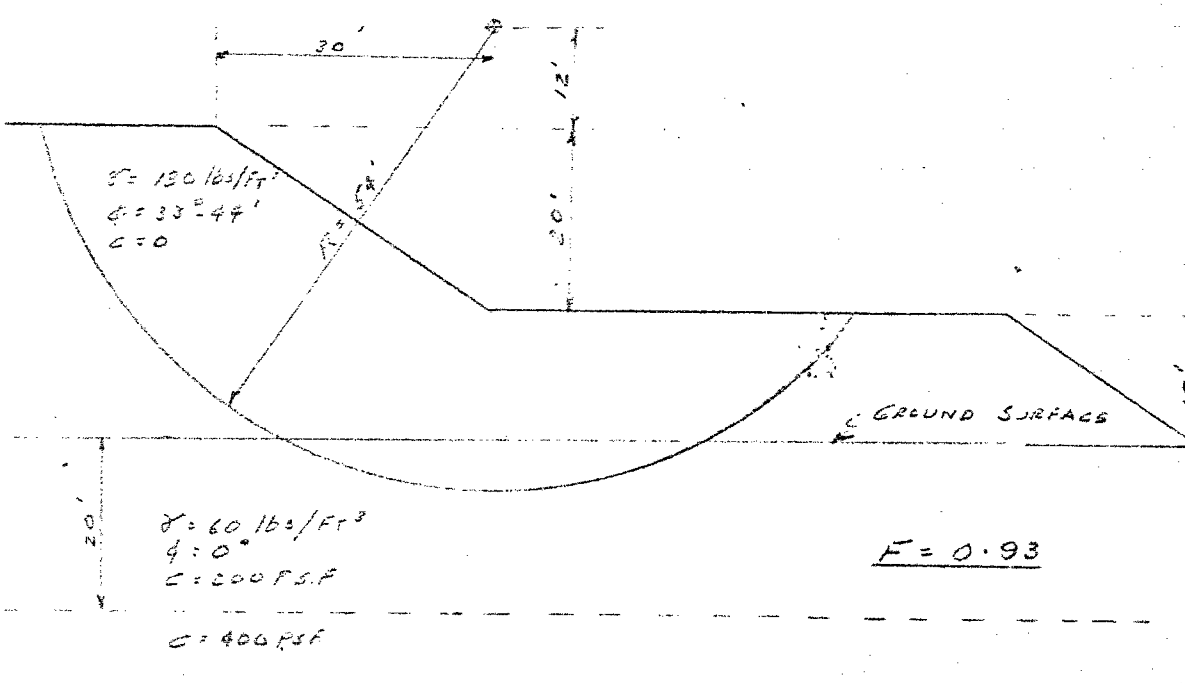


FIG. 2

Order No. S-500/5-6/T-233 Enclosure No. 11

Prep. By R. F. W.

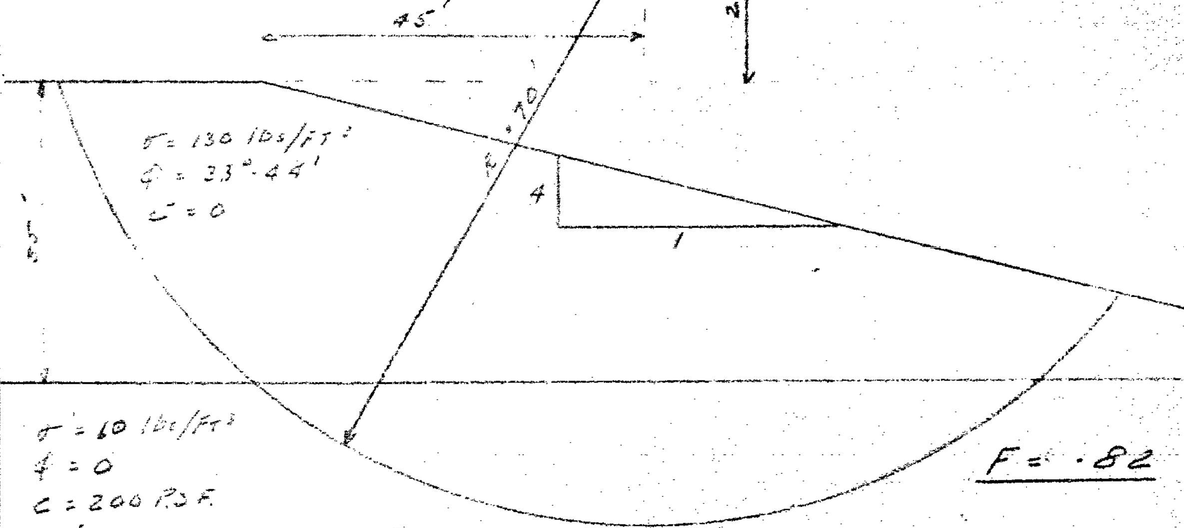


FIG. 1 SCALE 1" = 20'

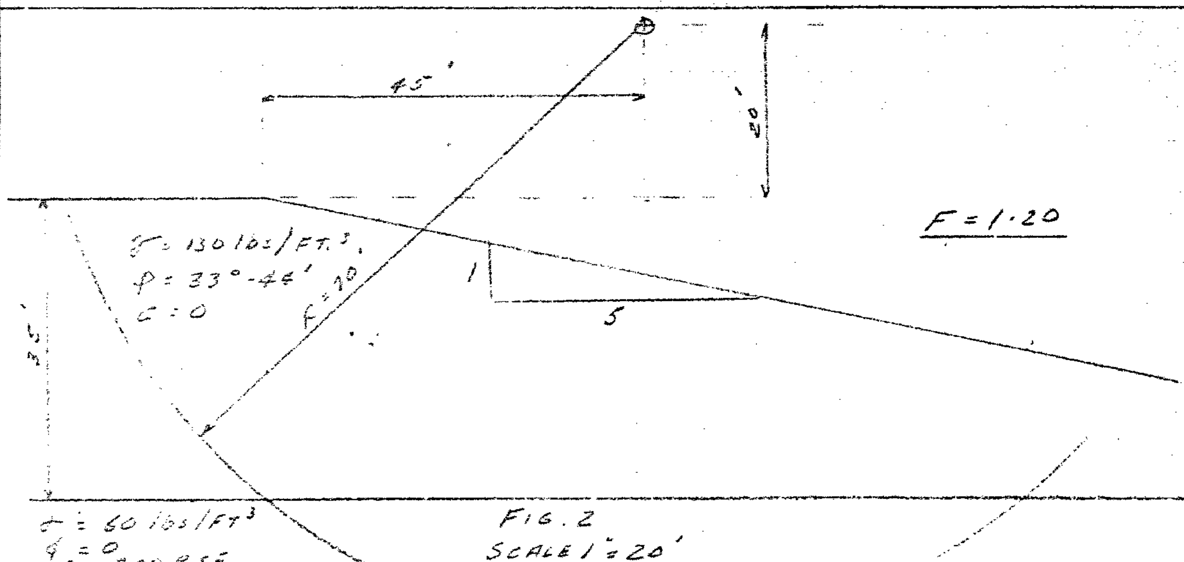


FIG. 2
SCALE 1" = 20'

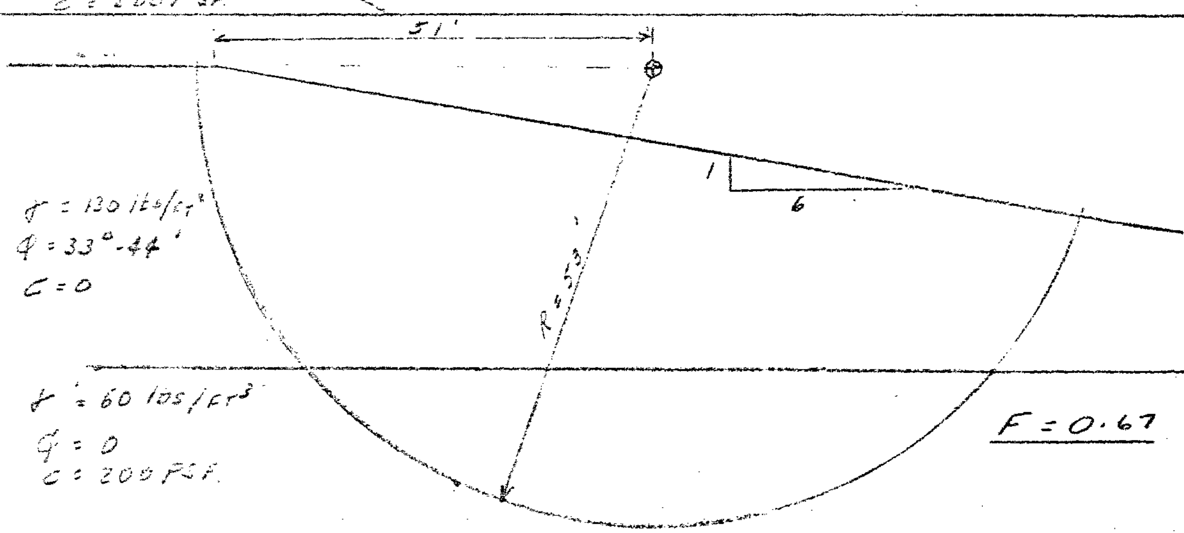


 FIG. 3 SCALE 1" = 20'

Order No. 5-500/56/T-233 . Enclosure No. 12

Prep. By B.F.W.

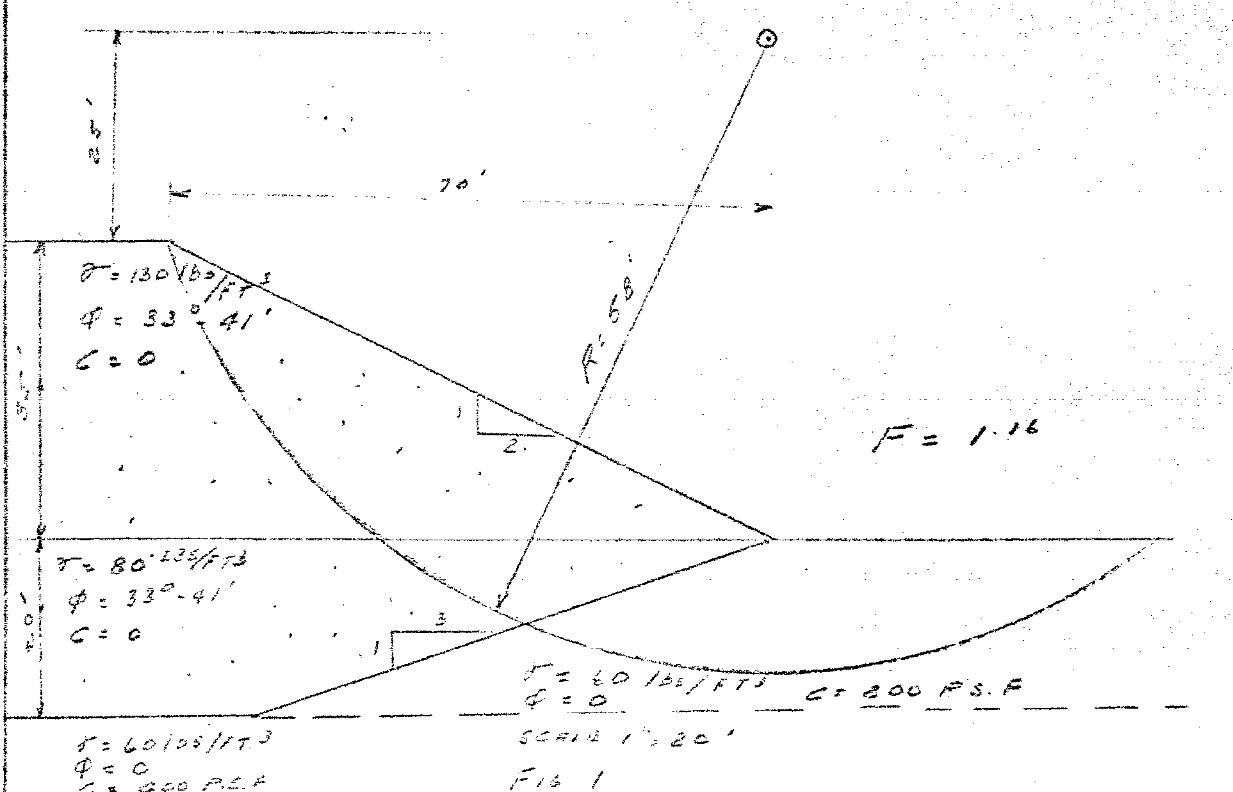


FIG 1

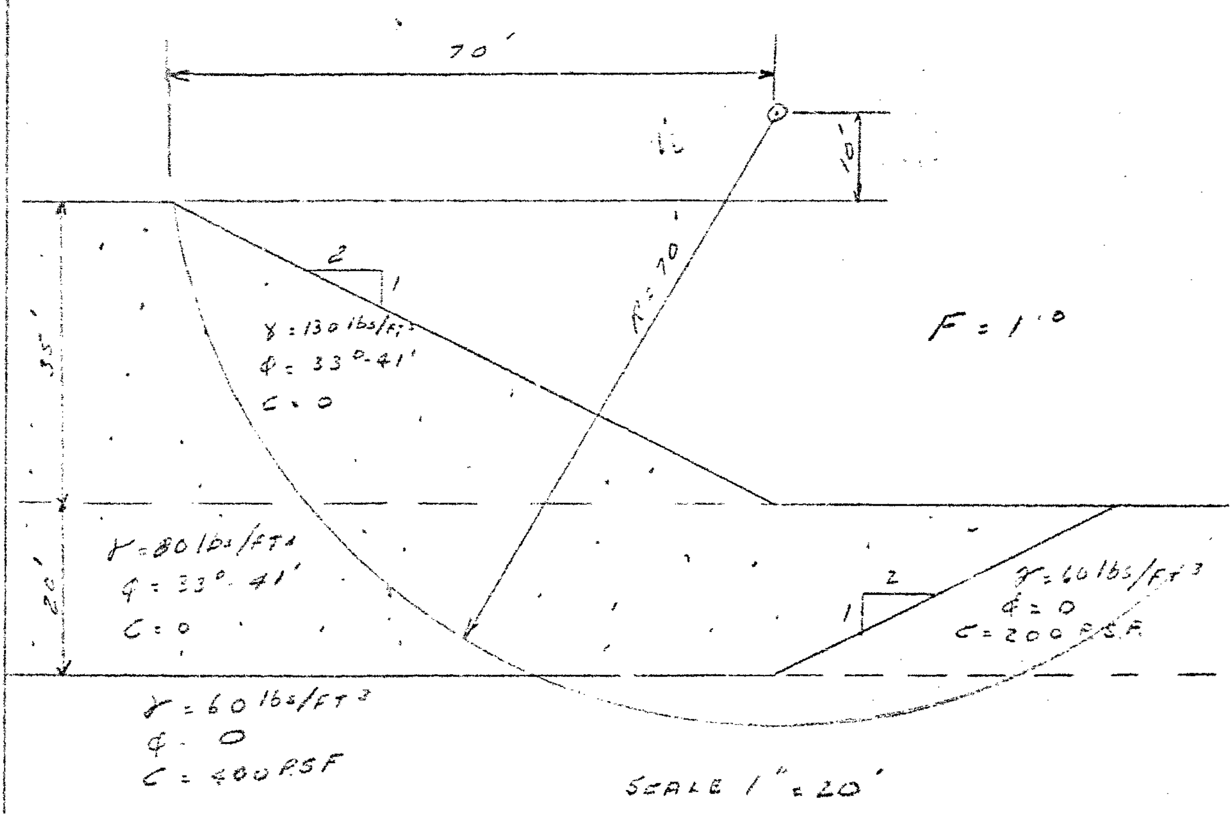


FIG. 2

Racey, MacCallum & Associates Ltd.

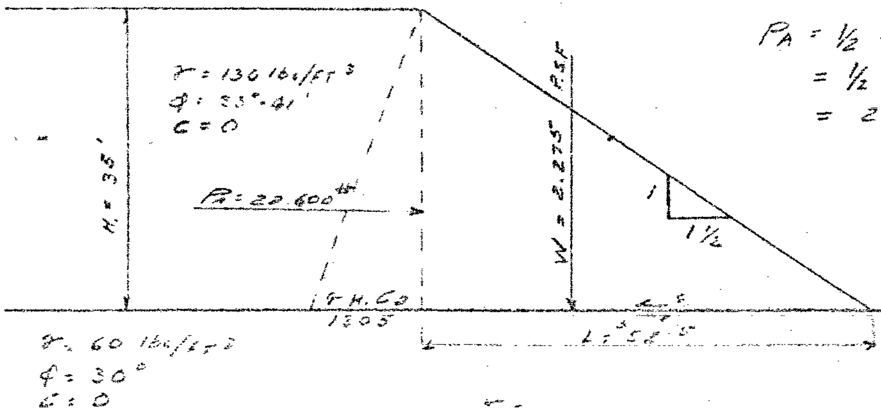
Order No. S-506/56/T-237
Prep. By D.F.W

$C_A = \tan^2\left(45 - \frac{\phi}{2}\right)$
$= .2864$

$P_A = \frac{1}{2}\gamma H_1^2 C_A$
$= \frac{1}{2} \times 130 \times (35)^2 \times .2864$
$= 22,600\#$

$P_P = 0$

$W = \frac{\frac{1}{2} \times 35 \times 52.5}{52.5} \times 130 = 2,275$ P.S.F.



SHEAR STRENGTH $\bar{S} = P_A - P_P = 22,600$
UNIT SHEAR STRENGTH $s = \frac{S}{L} = \frac{22,600}{52.5} = 431$ P.S.F.

$1.2\ s = 517$ P.S.F. WHERE 1.2 = FACTOR OF SAFTEY

SLIDING BLOCK ANALYSIS

SCALE :- 1" = 20'

Enclosure No. 13

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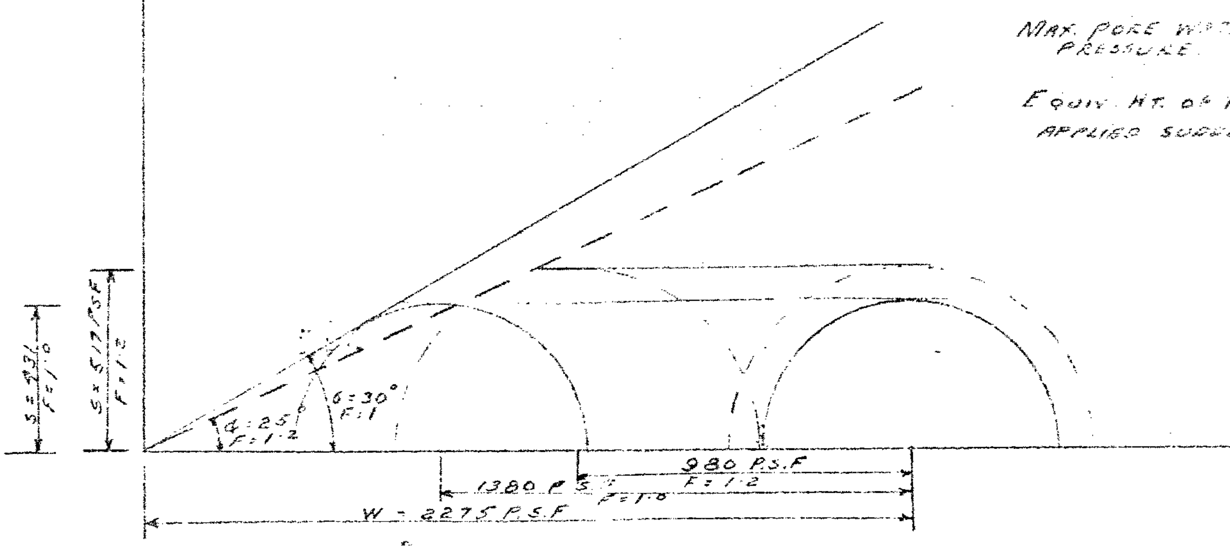
Order No. 5-500/56/T-233

Prep. By R.F.K.

ASSUME EFFECTIVE STRENGTH
OF CLAY $\phi = 30°$ $C = 0$

	F = 1.0	F = 1.2
MAX. PORE WATER PRESSURE	1380 P.S.F.	980 P.S.F.
EQUIV. HT. OF FILL APPLIED SUDDENLY	10.6 FT.	7.5 FT.



SLIDING BLOCK ANALYSIS

SHEAR DIAGRAM

SCALE 1" = 500#

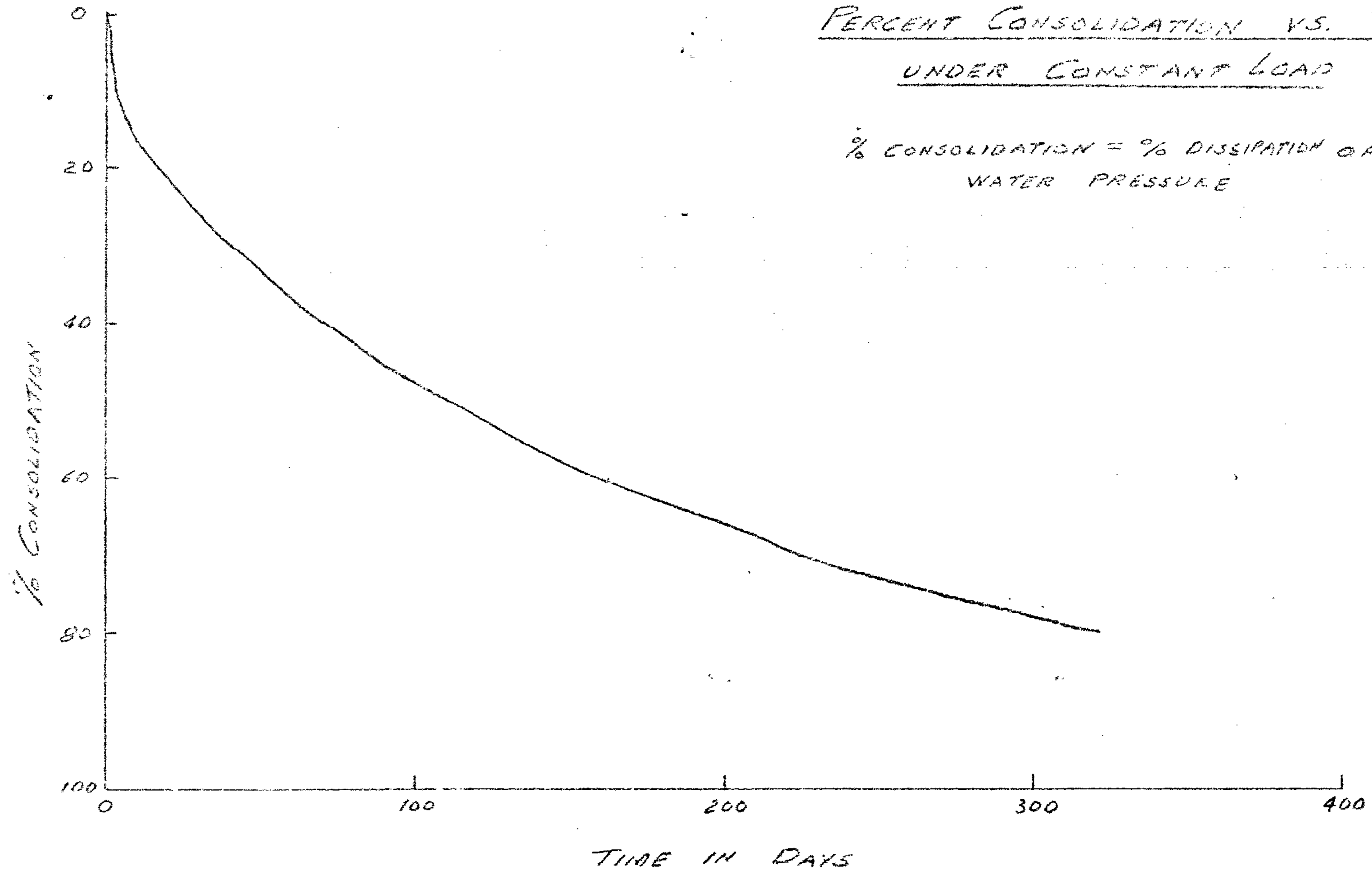
Enclosure No. 14

Order No. 5 500/58/T-233

Prep. By B.J.W

PERCENT CONSOLIDATION VS. TIME UNDER CONSTANT LOAD

% CONSOLIDATION = % DISSIPATION OF PORE WATER PRESSURE



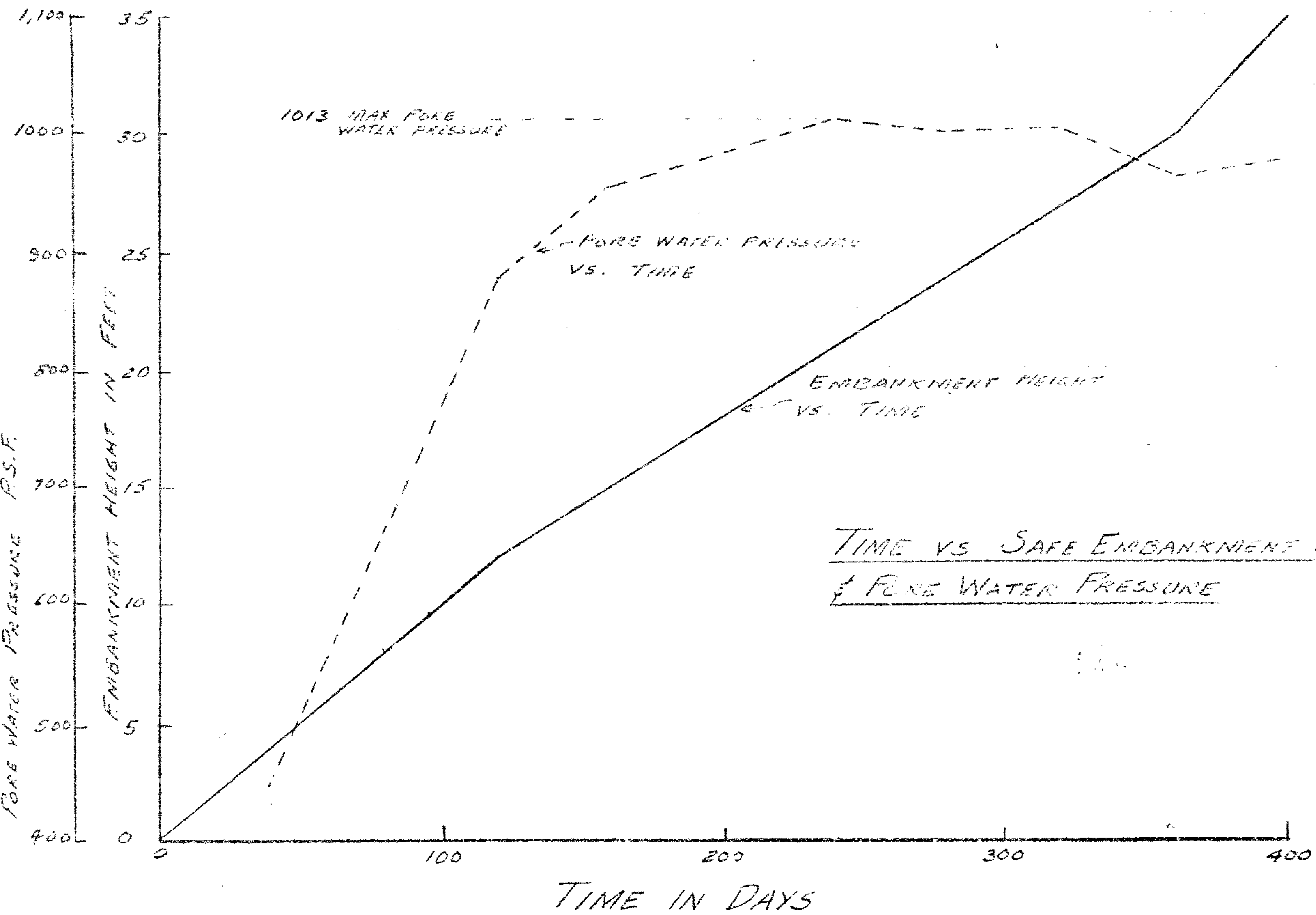
Enclosure No. 15

Racey, MacCallum & Associates Ltd.

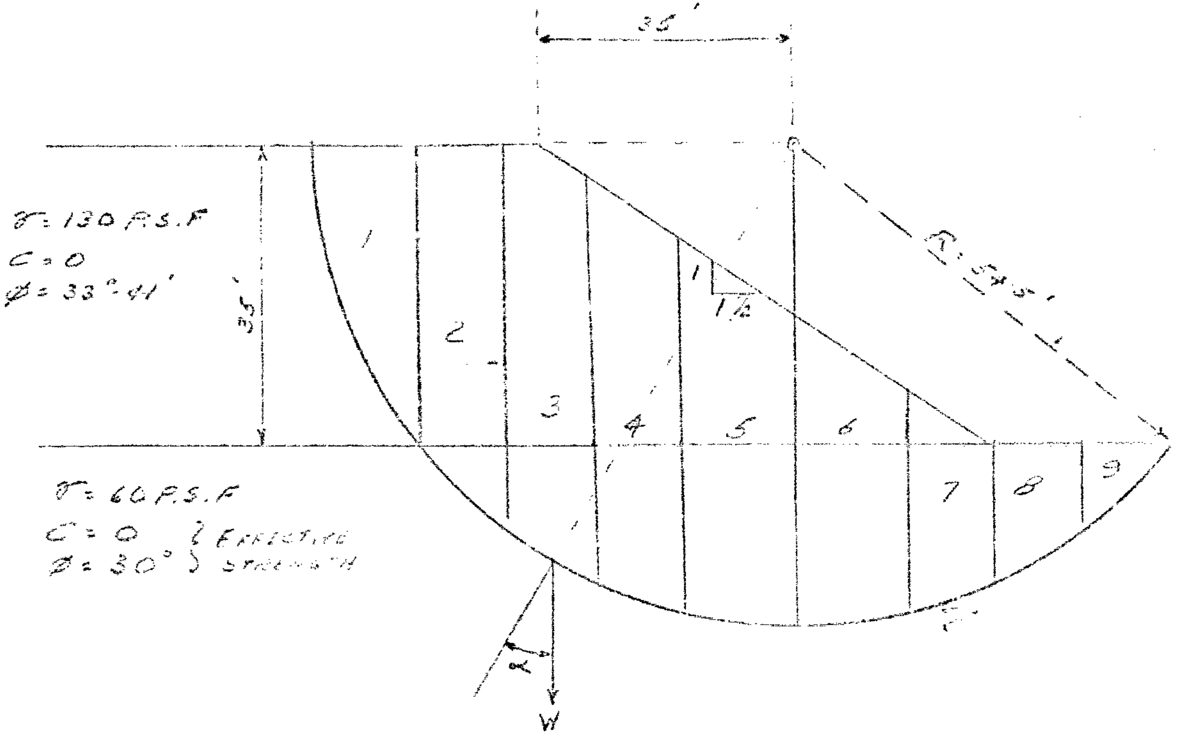
Racey, MacCallum & Associates Ltd.

Order No. 5-500/5C/7-233

Prep. By B.F.W.



Enclosure No. 16



SLIP CIRCLE FOR STABILITY ANALYSIS BY

BISHOP'S METHOD

SCALE :- 1" = 20'