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HWY 11

OVER C.N.R.

CALLANDER



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57F201C

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**LIMITED**

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27 CARLTON STREET

REPORT NO: S-500/T-633

3 May 1957.

A.M. Ioye, Esq.,  
Bridge Engineer,  
Department of Highways of Ontario,  
290 Davenport Road,  
TORONTO, Ontario.

Attention: Mr. S. McCombie

**RE: FOUNDATION INVESTIGATION FOR THE  
PROPOSED RAILWAY OVERPASS AND  
APPROACH EMBANKMENT ON HIGHWAY NO. 11,  
OVER THE C.N.R. TRACKS, NEAR  
CALLANDER, ONTARIO.**

Dear Sirs:

Enclosed herewith is our report covering the results of a field investigation and associated analysis of the subsoil competence for the support of the above-noted railway overpass.

This report was prepared by our Mr. P.E.M. Monk, and the comments and opinions presented in it can be briefly summarised as follows:-

1. The field vane measurements in the sensitive clay stratum underlying the proposed railway crossing are believed to provide a more realistic indication of shear strength, than do laboratory tests. This is because loss of strength due to disturbance is kept to a minimum in this test. The results of slow drained direct shear tests and of consolidation tests appear to confirm this view.

2. Analyses show that a 3:1 embankment slope can be constructed safely, and that 2:1 slopes can be used if stage construction is instituted. A 2:1 slope with 10 x 32 berms would also be acceptable.



3. Very high stresses will be exerted in the clay underlying the abutments, because of the great weight of the approach embankments. In order to maintain stable conditions adjacent to the railway therefore, either an open-type bridge structure or a one span bridge, incorporating the use of batter piles and sheet piling driven to a depth of forty eight feet, will be required. Sheet piling through the clay stratum will tend to control any twisting of the bridge due to the introduction of eccentric horizontal stresses in the clay.

4. The estimated settlement of the embankment has been computed to be of the order of fifteen inches. Ninety percent of this movement should be completed in about two years.

We sincerely regret any inconvenience to you, due to the delay in issuing this report. We shall be pleased to discuss any problems not specifically covered in it, if you so require.

Yours very truly,  
 PACKY, MACCALLUM AND ASSOCIATES LIMITED

*W.A. Trow*  
 W.A. Trow, P. Eng.,  
Divisional Soils Engineer.

WAT/D  
In quadruplicate



Department of Highways Ontario,  
280 Davenport Road,  
Toronto, Ontario.

FOUNDATION INVESTIGATION  
FOR THE PROPOSED RAILWAY OVERPASS AND  
SPURRACH EMBANKMENT ON HIGHWAY NO. 11  
OVER THE C.N.R. TRACKS NEAR CALLANDER  
ONTARIO.

Reference: No. S-500/T-633      Racey, MacCallum and Associates Ltd.  
May 3rd, 1957.



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FOUNDATION INVESTIGATION  
FOR THE PROPOSED RAILWAY OVERPASS AND  
APPROACH EMBANKMENT ON HIGHWAY NO. 11  
OVER THE C.N.R. TRACKS, NEAR CALLANIER,  
ONTARIO.

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PURPOSE OF THE INVESTIGATION AND SCOPE OF THE REPORT

The investigation was undertaken following work carried out by the Materials and Research Branch, Foundation Section, of the Department of Highways of Ontario, which had indicated unstable conditions with respect to the approach embankments at the above-mentioned site. The purpose of the investigation was to ascertain whether a re-location of the centre line of the proposed crossing to a point three hundred feet north along the railway lines, would enable the approach embankment to be founded upon a stable subsoil. Rock outcrops in the area had suggested that such a relocation might prove useful. Failing the discovery of an obviously stable subsoil markedly different in character from that encountered during the investigation carried out by the Department of Highways of Ontario, further testing of the original line was to be undertaken to obtain sufficient combined information for an economic design. At the suggestion of Mr. S. McCombie, this investigation was to include the performance of 'in-situ' vane tests, since it was felt that this method of measuring the shearing resistance of the soil was more representative for the sensitive clays encountered.

The report covers:-

1. The analysis of the stability of the embankment under instantaneous load conditions, based upon a sliding block analysis.
2. Computations based upon an effective stress analysis, to indicate the saving of embankment material made possible by the use of stage construction.
3. The factors of safety desirable for various parts of the embankment and overpass.



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# PURPOSE OF THE INVESTIGATION AND SCOPE OF THE REPORT (Cont'd)

4. The problems associated with the contact of the approach embankment with the overpass abutments, including brief outline calculations to indicate some of the possible design alternatives for the overpass itself.

5. The bearing capacity of the subsoil with respect to the overpass.

6. The magnitude and rate of settlement of the embankment.

## LOCATION OF THE SITE AND BOREHOLES

The site lies at the north end of Callander, Ontario, approximately 300 yards east of the existing highway No. 11. A sketch plan of the area indicating the location of the boreholes and penetration tests carried out during the investigation is shown on enclosure No. 1.

## GEOLOGY OF THE AREA

The area consists of bedrock of Diorite Gneiss overlain by relatively recent deposits of loose sand and silty clay. The bedrock outcrops frequently and appears to be quite irregular in profile.

## FIELD INVESTIGATION AND DESCRIPTION OF THE SUBSOIL PROPERTIES

The field investigation was commenced on 26th February 1957, and completed on 12th March. Two boreholes and three penetration tests were carried on the suggested relocated line 300 feet north along the C.N.R. tracks, and are shown on enclosure No. 1. Borehole No. 1 which was to the west of the C.N.R. tracks on the relocated line was taken to bedrock at a depth of 45 feet through 10 feet of loose to medium dense fine



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**FIELD INVESTIGATION AND DESCRIPTION  
OF THE SUBSOIL PROPERTIES (Cont'd)**

grey sand. At a depth of 10 feet a very sensitive brown banded with grey, fatty clay with odd very small fine sand pockets was encountered which continued to a depth of 16 feet. Vane tests carried out at depths of 12 feet and 17 feet indicated undisturbed shear strengths of 675 and 920 lbs. per square foot respectively and a sensitivity of 7. Below the clay stratum fine grey sand was encountered which became coarse sand from a depth of 42 feet down to bedrock at 45 feet. The cone penetration tests reached refusal at a depth of from 22 to 25 feet which indicates a denser layer of sand a short distance below the clay stratum. Four unconfined compression tests were carried out on two Shelby samples taken from the clay stratum. Three of these tests showed an average unconfined compression strength of 940 lbs. per square foot, and the fourth an unconfined strength of 645 lbs. per square foot. Borehole No. 2 which was drilled on the alternative route location as shown on enclosure No. 1, indicated subsoil conditions similar to those found in Borehole No. 1. Unconfined compression tests on two samples from Borehole No. 2 gave values of unconfined strength of 870 and 880 lbs. per square foot. Two vane tests indicated shear strengths of 572 and 440 lbs. per square foot. The sensitivities as measured by the vane were somewhat lower than in Borehole No. 1 as were the shear strengths.

After discussion with Mr. S. McCombie, Bridge Planning and Design Engineer, of the Department of Highways Ontario, it was decided to reinvestigate the original crossing with special attention being given to in-situ vane tests.

Five boreholes were drilled on the original line, each to a depth just below the bottom of the clay stratum. Two Shelby samples were taken in each borehole with the exception of Borehole No. 7 where the clay stratum was



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### FIELD INVESTIGATION AND DESCRIPTION OF THE SUBSOIL PROPERTIES (Cont'd)

thinner than at the other locations tested. The clay stratum was similar to that already described except that in Boreholes 6 and 7 it had somewhat greater strength than was observed elsewhere. The sensitivity was again high and there is some scatter in the results. In general it appears that the clay has an undisturbed shear strength in excess of 500 lbs. per square foot. Some of the laboratory results have been discarded since it is felt that the samples were disturbed. This is particularly true of Borehole No. 3 where almost continuous sampling was attempted, and it is thought that the vane tests disturbed the shelly samples and 'vice versa'. The higher strengths obtained in check boring No. 5 appear to confirm this view.

A wide scatter in the moisture content results was obtained and this is thought to be due to the banded nature of the clay. The natural moisture content was found to be consistently greater than the liquid limit which further demonstrates the high sensitivity of the clay.

### DISCUSSION

The problems associated with the design of the proposed overpass and approach embankment are many. For convenience they will be dealt with under the following headings.

1. The stability of the approach embankment.
  - (a.) Stability with respect to a sliding block analysis.
  - (b) Long term stability with respect to an effective stress analysis and stage construction.
  - (c) The desirable factor of safety with respect to the possible damage resulting from a failure.



2. The stability of the overpass

- (a) Designed as an open multi-span bridge, allowing a safe natural slope to the approach embankment.
- (b) Designed as a single span bridge with full embankment earth pressures on the abutments.

Design calculations for the above considerations are shown at the end of this report.

3. Bearing capacity with respect to the overpass.

4. Anticipated settlement of the approach embankments.

1. The Stability of the Approach Embankments.

(a) Stability with respect to a sliding block analysis

From the calculations shown on the design sheets at the end of this report, it can be seen that with embankment slopes of 2:1, the underlying clay requires a shear strength of 550 lbs. per square foot for stability. Skempton (Geotechnique June 1955), states that in sensitive materials the values of shear strength, as obtained by means of the vane test, should be used in preference to those obtained by unconfined compression tests. In view of the sensitivity to disturbance of the clay at this site, both during sampling and in the subsequent transport to Toronto, this preference would appear reasonable. On this basis therefore, the values of shear as determined by the unconfined tests, have been ignored. The results from borehole no.3 have also been discarded, since they are felt to be low due to disturbance, caused by attempting to take continuous vane tests and Shelby tube samples. When the results of this boring indicated low values of shear strength, a second borehole no.5 was put down nearby, in which substantially higher shear strengths were encountered.



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The average of the two lowest shear strengths along the line of the proposed embankment, obtained by means of the vane test, is 560 lbs. per square foot. The results of two consolidation tests and one direct shear test appear to substantiate this value of shear strength. The value of preconsolidation pressure, as shown by the preconsolidation tests, is 2400 lbs. Assuming a value of 560 lbs. per square foot, as the shear strength, would give a value of c/p ratio of approximately 0.25, which is not unreasonable for a soil of this plasticity. The direct shear test indicates a shear strength prestress of the order of 500 lbs. per square foot.

Using the average lowest vane shear strengths, a 2:1 slope is marginally stable by a sliding block analysis. With a 3:1 slope the shear stress at the bottom of the clay layer is 420 lbs per square foot and, therefore, the factor of safety with respect to a sliding block analysis is 1.3.

#### (b) Stability with respect to an Effective Stress Analysis

The sliding block analysis gives the factor of safety with respect to stability under instantaneous loading. It may however, be possible to obtain a greater factor of safety by means of stage construction, and to ascertain whether this is possible an effective stress analysis must be carried out. The calculations for an effective stress analysis are shown on the design sheets at the end of this report, and the results may be summarised as follows. With a factor of safety of 1.2 applied to the angle of shearing resistance obtained from the slow drained direct shear test, and assuming an embankment slope of 2:1, an embankment lift of twenty one feet may be constructed. Based upon the time taken for the centre of the clay layer to reach the required state of consolidation, a time interval of two hundred and thirteen days must be allowed between the completed placement of the first lift and the start of the second lift. These calculations are based upon the assumption that the second lift would complete the embankment to full height.



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- (c) Factors of Safety desired with respect to the Embankment itself, and to the Embankment and the Overpass considered as a Unit.

Due to the consequences of failure of the overpass, which is over a main line of the C.N.R., the selection of a factor of safety close to this line must be considered carefully. It has been shown with a 2:1 slope that, on the basis of a sliding block analysis, the subsoil is in a state of marginal stability and that, with stage construction, has a factor of safety of 1.2 as applied to the effective stress analysis. It is immediately apparent therefore, that a 2:1 slope cannot be considered for any part of the embankment if it is built without stage construction and, further, it is thought that close to the overpass itself a lesser slope would be desirable, to ensure an adequate factor of safety with relationship to the damage which could result were a failure to occur. Based upon a 3:1 slope and the assumptions made in the appendix, the factor of safety with respect to a sliding block analysis is 1.3 at the completion of the embankment, assuming that no consolidation has taken place during construction. Consolidation of the clay layer will take place and will be quite rapid at the bottom of the clay, the level of greatest induced stress. When the clay has completely consolidated, the factor of safety at this level will have increased to approximately 1.8.

For the remainder of the embankment, away from the effects of the overpass itself, either a 3:1 slope or a 2:1 slope with stage construction, may be used. Use might be made of side berms to conserve material, if stage construction is considered undesirable. A slope of 2:1 with side berms 10m wide would be safe, with a factor of safety of 1.3 as for the 3:1 slope.

## 2. The Stability of the Overpass

### (a) Designed as an open Multi-span bridge

The overpass may be designed approximately as shown



in fig.1, with the reactions supported by piles upon bedrock.



The embankment material would then slope away from the railway tracks at a slope of 3:1, and no lateral thrusts due to the embankment would be placed upon the abutments. This design, however, means that the total span will be over two hundred and fifty feet, and the structure will prove expensive.

(b) Designed as a Single Span Bridge, with full Embankment earth pressures on the Abutments.

There are two main problems associated with the construction of an overpass having full earth pressures on the abutments, and they are:-

1. The stability of the underlying soft clay, with respect to a normal passive failure.
2. The stability of the bridge structure itself, with respect to the eccentric earth loads tending to rotate the structure or the soil beneath the structure.

1. From the calculations shown on the design sheets, it may be seen that a total length below existing ground surface of forty eight feet of sheet piling is required for the stability of the underlying soil, assuming that the lateral thrusts on the abutments are supported by batter piles, or fifty six feet of sheet piling if the abutment thrusts have not been accommodated in some other way. The sheet piling should be integral with the abutments. It may prove beneficial to brace the abutments, one against the other.



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2. The design sheets indicate that the subsoil is unstable with respect to rotation due to the large eccentric earth pressures. However, with the inclusion of sheet piling as required by the previous section, the rotational plane shear should not take place, due to the passive pressures set up behind the sheet piling.

It seems therefore, that if full abutment earth pressures are to be used, the sheet piling will have to be taken to bedrock.

### 3. Bearing Capacity of the Subsoil with respect to the Overpass

A recommendation with regard to the support of the bridge abutments, has been given in the report submitted by the Foundation Section of the Department of Highways of Ontario.

### 4. Settlement of the Embankment

The results of the consolidation tests are shown on enclosure no.5. Assuming an existing overburden pressure with respect to the centre of the clay layer of  $15 \times 60 = 900$  lbs., and a final overburden pressure of 4530 lbs., the change in relative void ratio as indicated by the consolidation curves, is  $1.12 - .86 = .26$  for hole no.2 sample 2, and .3 for hole no. 3 sample 1.

$$\text{Probable settlement} = \frac{e_1 - e_2}{1 + e_1} \times D$$

Where  $e_1$  = initial void ratio

$e_2$  = final void ratio

D = thickness of clay stratum.

$$\text{Settlement} = \frac{.26}{2.12} \times 10 \times 12 = 14.2 \text{ inches}$$

Thus, the probable settlement of the embankment is of the order of fifteen inches. Approximately ninety percent. of this settlement should have taken place within two years.



## CONCLUSIONS

The comments presented in the foregoing discussion can be briefly summarised as follows:-

1. The subsoil at the site consists of loose to medium fine grey sand, in a saturated condition for a depth of approximately ten feet, underlain by sensitive banded grey and brown clay for a depth of approximately ten feet. This clay overlays sand and gravel, which continues to bedrock of Diorite Gneiss. The bedrock is probably very uneven, as is indicated by rock outcrops in the area.
2. Due to the sensitive clay layer, the embankment cannot be built at slopes greater than 2:1 without failure.
3. It is suggested that 3:1 slopes, or 2:1 slopes with side berms 10 x 32 feet be used, if no stage construction is considered. If stage construction is to be used, 2:1 slopes will be safe with a factor of safety of 1.2 as applied to an effective stress analysis immediately after the completion of construction. Stage construction would require approximately two hundred days between the first and final lifts.
4. It is suggested that an extra factor of safety is required close to the overpass itself, and that in this area a 3:1 slope would seem advisable.
5. The overpass may be designed with no earth pressures on the abutments, in which case the recommended 3:1 slopes should be used for the embankment slope to the railway. This means that the bridge spans will total more than 250 feet.
6. An alternative design would be to take full earth pressures on the embankments of a single span bridge, and accommodate the earth pressures by batter piles and sheet piles driven to bedrock.



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7. The probable settlement of the embankment is estimated to be of the order of fifteen inches, and it is anticipated that ninety percent of this settlement would be complete within two years.

8. The installation of piezometers would greatly facilitate the control of stage construction, and would also be useful for determining the actual state of the consolidation under field loading conditions. Thus decisions regarding the final grade level can be made with greater assurance that no major settlement would occur at a later date.

9. The overpass should be carried on bearing piles to bedrock, as suggested by the foundation report prepared by the Department of Highways of Ontario, numbered B.A.577.

*P. E. Monk*

P. E. M. Monk, P. Eng.

PERC/MO



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$$\phi = 30^\circ$$

SLIDING BLOCK ANALYSIS

Assumptions

The average depth of existing sand overlaying the clay layer = 10 feet.

The average depth of clay layer = 10 feet.

Granular fill will be used for the construction of the embankment.

Unit weight of embankment = 110 lbs. per sq. foot.

Submerged unit weight of existing sand layer = 60 lbs. per sq. foot.

Submerged unit weight of clay layer = 60 lbs. per sq. foot.

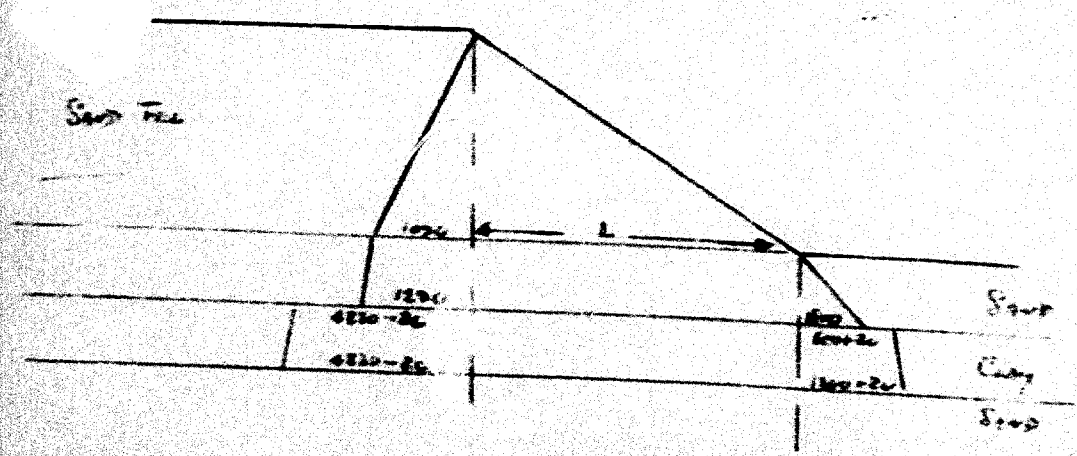
$K_A$  for embankment material = .3

$K_A$  for existing sand = .33

$K_p$  for existing sand = 3.

Height of embankment 32 feet use 33 feet to allow for traffic loads.

Embankment slopes are (a) 2:1 (b) 3:1



Section Through The Embankment

Showing Active + Passive Press. & Distribution



Active pressure to base of  
embankment  $p_a = \gamma h K_a$

$$\therefore P_a = 110 \times 33 \times 3 = 1090 \text{ lbs. per sq. foot}$$

$$\text{Active thrust} = 1090 \times 33 \times \frac{1}{2} = 18,000 \text{ lbs. per run. foot}$$

Active pressure at bottom of existing sand layer  
 $P_{a2} = 1090 + 60 \times 10 \times 1/3 = 1290 \text{ lbs. per sq. foot}$

$$\text{Active thrust in saturated sand} = (1290 + 1090) \times \frac{1}{2} \times 10 = 11,900 \text{ lbs. per run. foot.}$$

$$\therefore \text{Total active thrust to top of clay layer} = 29,900 \text{ lbs. per run. foot}$$

$$\begin{aligned} \text{At top of clay layer } P_{a3} &= 110 \times 33 + 60 \times 10 - 2c = \\ &4230 - 2c \text{ lbs. per sq. foot} \\ &\text{where } c = \text{cohesion.} \end{aligned}$$

$$\begin{aligned} \text{At bottom of clay layer } P_{a4} &= 4230 + 60 \times 10 - 2c = \\ &4830 - 2c \text{ lbs. per sq. foot} \end{aligned}$$

$$\therefore \text{Active pressure in clay layer } (4230 + 4830 - 4c) \times \frac{1}{2} \times 10 = 45300 - 20c \text{ lbs. per run. foot}$$

$$\therefore \text{Total active thrust} = 75200 - 20c \text{ lbs. per run. foot}$$

Passive pressure  $p_p$  at bottom of existing sand layer  
beneath toe of slope  $= \gamma h K_p = 60 \times 10 \times 3 = 1800 \text{ lbs. per sq. foot}$

$$\therefore \text{Passive thrust in sand} = 1800 \times 10 \times \frac{1}{2} = 9000 \text{ lbs. per run. foot}$$

$$\begin{aligned} \text{Passive pressure at upper clay layer beneath toe of slope} \\ P_{p1} &= 60 \times 10 + 2c = 600 + 2c \text{ lbs. per sq. foot} \end{aligned}$$



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At bottom of clay layer beneath toe of slope

$$P_{p2} = 600 + 60 \times 10 + 2c = 1200 + 2c \text{ lbs. per sq. foot}$$

$$\therefore \text{Passive thrust in clay layer} = (600 + 1200 + 4c) \frac{1}{2} \times 10 = 9000 + 20c \text{ lbs. per run. foot}$$

$$\therefore \text{Total passive thrust to bottom of clay layer} = 18,000 + 20c \text{ lbs. per run. foot}$$

Stress S along sliding plane is given by  $S_L = P_a - P_p$

And at marginal stability on any plane  $S = C$

If slopes are 2:1 then for embankment 32 feet high  $L = 64$  feet

$$\text{At top of clay layer } 64c = 29,900 - 9000$$

$$\therefore \text{required cohesion } c = \frac{20,900}{64} = 326 \text{ lbs. per sq. foot}$$

$$\text{At bottom of clay layer } 64c = 75,200 - 20c = (18,000 + 20c)$$

$$\therefore 64c + 40c = 57200$$

$$c = 550 \text{ lbs. per sq. foot.}$$

Comparing this value of cohesion with the values obtained by means of the vane it can be seen that with the exception of Borehole No. 2 T<sub>1</sub> (ignoring the results from Borehole No. 3 which were disturbed due to overclose sampling) the embankment is at the point of marginal stability.

As has been stated in the discussion (after Skempton Geotechnique June 1955) the vane test is a more reliable method of determining the shear strength of sensitive materials and for this reason it is believed that the low values of unconfined strength should be ignored.



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Slope of 3:1 Then for an embankment 32 feet high  $L = 96$  feet  
At bottom of clay layer

$$96c + 40c = 57,200$$

$$c = \frac{57200}{136} = 420 \text{ lbs. per sq. foot}$$

$$\text{Top of clay layer } c = \frac{20900}{136} = 154 \text{ lbs. per sq. foot}$$

These figures of cohesion and shear stress under a 3:1 slope indicate a stable condition even when applied to the lowest values of shear strength obtained by means of the vane test. The lowest value ignoring Borehole No. 3 was 440 lbs. per sq. foot, and this was obtained near the bottom of the clay layer at Borehole No. 2 which was 300 feet north of the proposed crossing. Taking the average of the two lowest vane strengths obtained from Boreholes along the line of the proposed crossing a value of shear strength of 560 lbs. per sq. foot is obtained.

Applying this value of 560 lbs. per sq. foot to the worst shear stress in the clay layer a safety factor of 1.33 is obtained. This factor of safety does not take into account the fast rate of consolidation at the lower clay surface which is the point of maximum shear stress, and is therefore conservative.

#### EFFECTIVE STRESS ANALYSIS

The results of a series of slow drained shear tests are shown on enclosure No. 3. From these tests it can be seen that the angle of shearing resistance is  $28^\circ$ . Further the suggested curve at the lower portion of the graph indicates a shear strength prestress of the order of 500 lbs. per sq. foot which checks with the values obtained by means of the vane.



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The following calculations are based upon the analysis procedure outlined by Skempton and Bishop in "Gain in Stability Due to Pore Pressure Dissipation in a Soft Clay" from the 5th International Conference on Large Dams 1955.

Consider 2:1 slopes, and apply a factor of safety of 1.2 to both the angle of shearing resistance and to the apparent shear strength as shown on enclosure No. 4.

Now the shear stress at:-

Top clay layer = 326 lbs. per sq. foot

Mid clay layer = 467 lbs. per sq. foot

Bottom clay layer = 550 lbs. per sq. foot

Intensity of loading at:-

Top clay layer =  $3630 \div 600 = 4230$  lbs. per sq. foot

Mid clay layer =  $4230 + 300 = 4530$  lbs. per sq. foot

Bottom clay layer =  $4530 + 300 = 4830$  lbs. per sq. foot

Consider the stress conditions at bottom of clay layers:-

Shear stress = 550 lbs. per sq. foot

Load intensity = 4830 lbs. per sq. foot

Draw a stress circle representing these conditions as shown on enclosure No. 4. This circle may be moved to the left until it becomes tangent to the failure envelope, and in this position represents effective stress condition, where



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the distance moved to the left is a measure of the allowable maximum excess pore pressure, before failure will occur. From enclosure no.4 the allowable pore pressure is 3500 lbs. per square foot. This represents an embankment height above existing ground of  $\frac{3500 - 600 - 600}{110}$  feet = 21 feet.

Now, due to drainage at this lower surface, this pore water pressure will dissipate rapidly. Therefore, the critical point so far as the second lift is concerned, is the point at which the pore water pressure has decreased least, i.e. the mid point of the clay stratum. Considering the mid point of the clay strip in an effective stress analysis, the allowable maximum pore water pressure for F.S. = 1.2 is 3100 lbs. per square foot.

This represents  $\frac{3100 - 600 - 300}{110}$  = 22.7 feet of embankment

above ground surface. Therefore, an embankment lift of twenty one feet, as determined for the bottom of the clay, is safe at mid point.

If twenty one feet is built as determined by the allowable stresses at the bottom of the clay layer then, with respect to the centre of the clay layer, it can be considered that 1.7 feet of pressure has already dissipated. In order to build the remaining eleven feet of embankment therefore, a further 9.3 feet of excess pore pressure must be dissipated.

∴ percentage dissipation of pore water to enable final lift to be made, is  $\frac{9.3}{21} \times 100 = 44.3$  percent.

Now, using the curves shown in Taylor's "Fundamentals of Soil Mechanics", page 235, at the centre of a clay strip the time factor T, for 45 percent dissipation, is .34. Considering the consolidation results shown on enclosure no.5, the coefficient of consolidation over the range of pressures involved, may be taken as .04 square feet per day.



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Now the time for required dissipation is given by  $t = \frac{T \times H^2}{C_v}$

where  $t$  = time in days

$T$  = time factor

$H$  = half thickness of strip

$C_v$  = coefficient of consolidation

$$\therefore t = \frac{.34 \times 25}{.04} = 213 \text{ days}$$

Thus the estimated time interval between the completed application of the first lift and the start of the second lift is 213 days. This estimated time for the required dissipation of pore pressure can only be determined, with certainty, from a piezometer installation.

#### STABILITY OF THE UNDERLYING SOFT CLAY BENEATH BRIDGE ABUTMENTS WITH FULL EARTH PRESSURES ON ABUTMENTS

As has been shown on the preceding sections the clay stratum will be unstable under the weight of the 32 feet high embankments and tend to cause a passive failure under the bridge itself. To accommodate the active pressures due to the high embankment it is suggested that sheet piling could be driven beneath the abutments and made integral with them. The following calculations are to determine the length of sheet piling required.

Two cases will be considered.

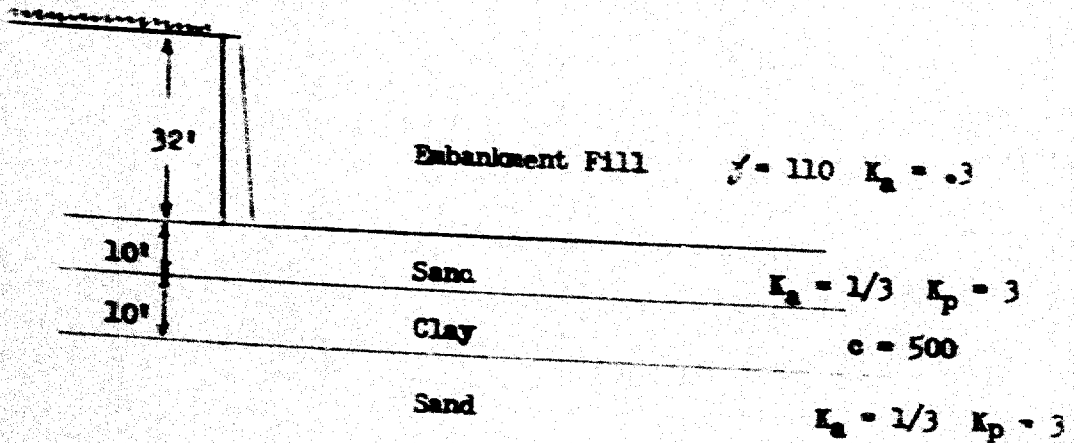
(i) All lateral forces on the abutments themselves are completely accommodated by means of batter piles, and therefore, only active and passive forces beneath the embankment need be considered.

(ii) All lateral forces are acting on a vertical plane through the abutments.



- 8 -

Traffic Load 1 foot fill



Active pressure at existing ground level  $P_{a1} = \gamma h K_a = 1090$

Active thrust on abutment  $P_{a1} = \frac{1}{2} \gamma h^2 K_a = \frac{1}{2} \times 110 \times 33^2 \times .3$   
 $= 18,000 \text{ lbs.}$

Active thrust in existing sand layer  $P_{a2} = 11,900$

Active pressure in top of clay layer  $P_{a2} = 110 \times 33 + 60 \times 10 - 2c$

Assume  $c = 500$   $P_{a3} = 3230 \text{ lbs. per sq. foot}$

Active pressure bottom of clay layer  $P_{a4} = 3230 + 60 \times 10$   
 $= 3830 \text{ lbs. per sq. foot}$

$\therefore$  Active thrust in clay layer  $P_{a3} = (3230 + 3830) \frac{10}{2}$   
 $= 35,300 \text{ lbs.}$

Active pressure at top of underlying sand  $P_{a5} = (3630 + 60 \times 20) \frac{1}{3}$   
 $= 1610 \text{ lbs. per sq. foot}$



- 7 -

Active pressure  $n$  feet below clay layer

$$1610 + (60n \times 1/3) = 1610 + 20n$$

Total active thrust in underlying sand for a depth of  $n$  feet

$$(1610 + 1610 + 20n) \frac{1}{2}n = 1610n + 10n^2 = P_1$$

Overall active thrust to a depth of  $n$  feet below clay layer

$$= 65,200 + 1610n + 10n^2$$

Passive thrust in upper sand layer  $\frac{1}{2} \times 60 \times 100 \times 3 = \underline{9,000 \text{ lbs.}}$

Passive pressure at the top of clay layer  $600 + 2c$

$$\text{assume } c = 500 \quad Pp2 = 1600$$

Passive pressure bottom of clay layer  $P_3 = 1600 + 600 = 2200$

Passive thrust in clay layer  $(1600 + 2200) \times 5 = \underline{19,000}$

Passive pressure top of underlying sand layer

$$P_1 = (600 + 600) 3 = 3600$$

Passive pressure  $n$  feet below clay layer

$$3600 + 60n \times 3 = 3600 + 180n$$

Total passive thrust in underlying sand  $(3600 + 3600 + 180n) \frac{1}{2}n$

$$= 3600n + 90n^2$$

Overall passive pressure  $= 28,000 + 3600n + 90n^2$



- 10 -

For balance between active and passive pressure

$$P_a - P_p = 0$$

Applying a factor of safety of 2 to passive pressures and considering case (i)

$$P_a - P_p = 47,200 + 1610n + 10n^2 - 11,000 - 1,800n - 45n^2 = 0$$

$$\text{i.e.} \quad -33,200 + 190n + 35n^2 = 0$$

$$n = \frac{-190 \pm \sqrt{190^2 + 4 \times 35 \times 33,200}}{70}$$

$$\therefore n = \underline{28 \text{ feet}}$$

$\therefore$  Total depth of sheet piling required below present ground surface if abutment thrusts fully accommodated = 48 feet

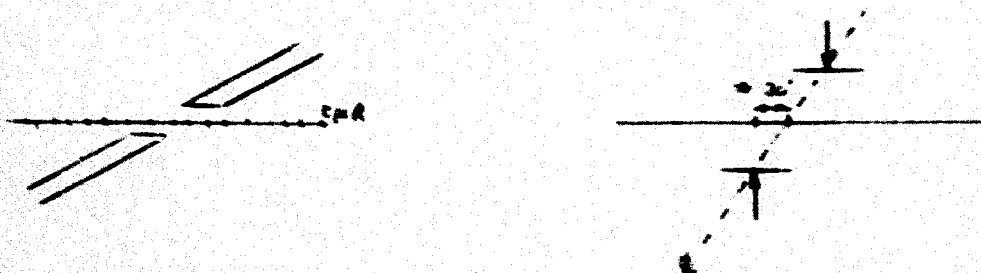
If no factor of safety is applied to passive pressure the total required length below present ground levels is 27.3 feet

Similarly for case (ii)  $n = 36$  feet, and the total depth of sheet piling required if abutment thrusts not accommodated = 56 feet. In this case horizontal compression struts between the bases of the abutments must be provided.



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**STABILITY OF THE BRIDGE STRUCTURE WITH RESPECT TO  
THE ECCENTRIC EARTH PRESSURES DUE TO THE ACUTE  
SKW OF THE APPROACH (IF NO SHEET PILING INSTALLED)**



Eccentricity of each resultant earth thrust approximately 20 feet. Say width of bridge along skew  $y$  feet. From previous calculations assuming the abutment thrusts are fully accommodated, then the active and passive thrusts to the bottom of the clay layer are as follows

$$P_a = 11,900 + 35,300 = 47,200 \text{ lbs. per run. foot}$$

$$P_p = 9000 + 19,000 = 28000 \text{ lbs. per run. foot}$$

$$P_a - P_p = 19,200 \text{ lbs. per run. foot}$$

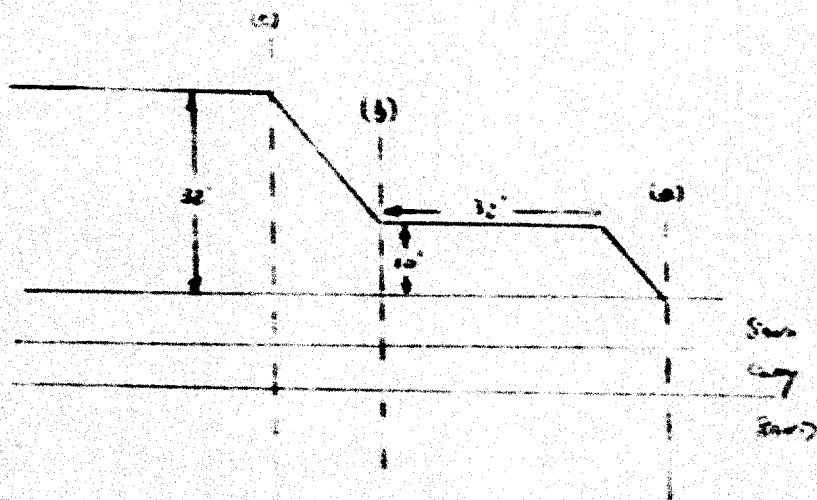
∴ Moment tending to rotate earth mass below abutments

$$= 19,200 \times y \times 20 \times 2 \text{ lbs. feet} \approx 360 y \text{ foot tons}$$

This moment must be accommodated by shear in the clay. The dimensions of the shear cylinder involved are difficult to ascertain. However since the sheet piling required by other stability considerations has been designed to ensure stability of each abutment and the underlying soil individually, provided this sheet piling has been installed the moment has been accommodated.



STABILITY WITH SIDE BERMS 10 x 32 FEET



The stability at section (a) will be as for a 3:1 slope i.e. the factor of safety by a sliding block analysis is 1.3 immediately after construction. Section (b) will be considered for stability to ensure the berm is sufficiently high.

Total active thrust on section (a) as previously computed to the bottom of the clay layer is 65,200 lbs. per run. foot.

Passive thrust on section (b) due to berm = 16,500 lbs. per run. foot

Passive thrust on section (b) due to existing sand = 42,000 lbs. per run. foot



- 13 -

Assuming  $c = 500$

Passive thrust on section (b) due to clay = 30,000 lbs. per  
run. foot

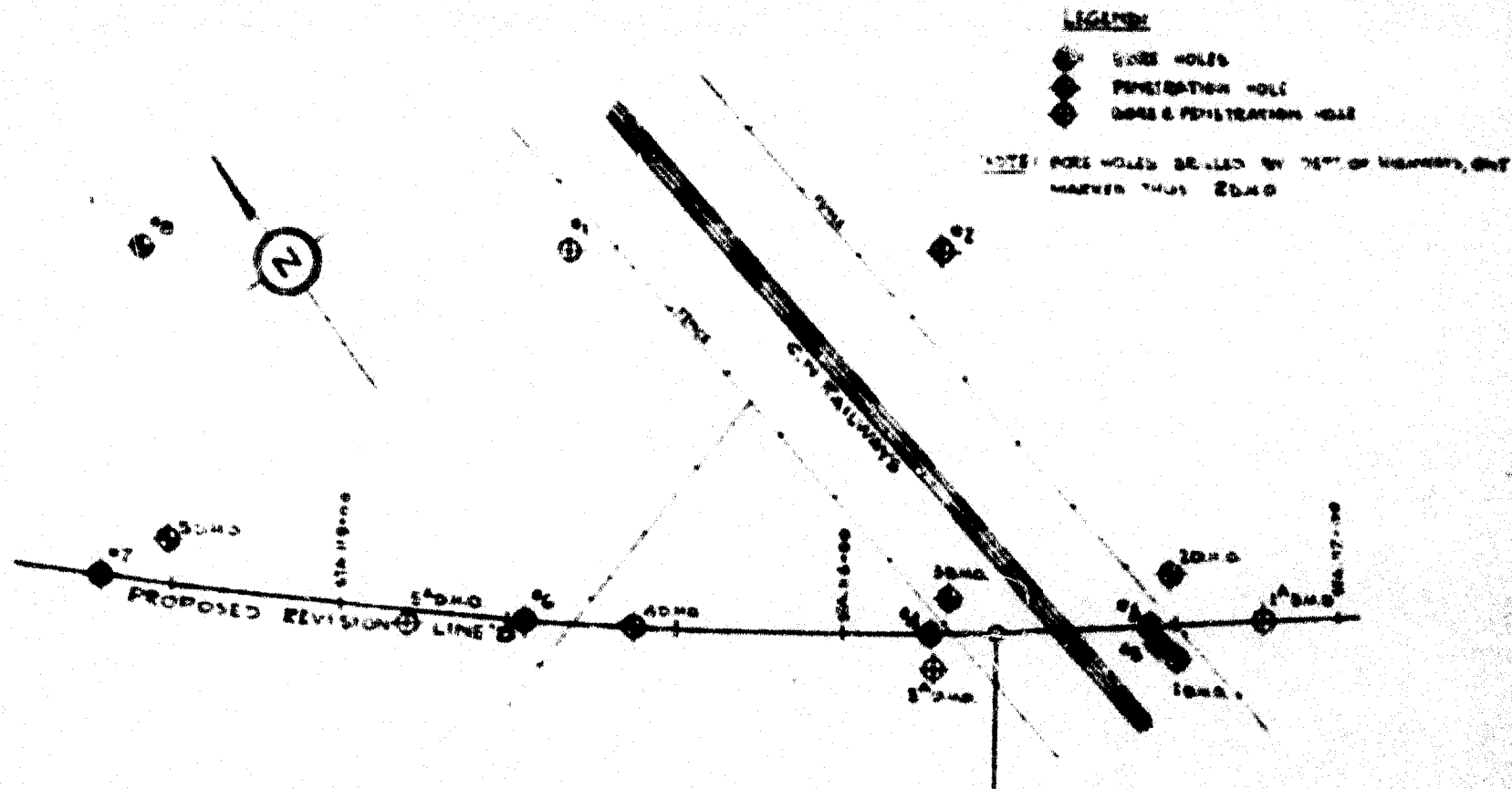
Total passive thrust 88,500

$L = 44$  feet

$$F.S. = \frac{P_p + CL}{P_a} = 1.7$$

∴ Section at toe of berm is the more critical.

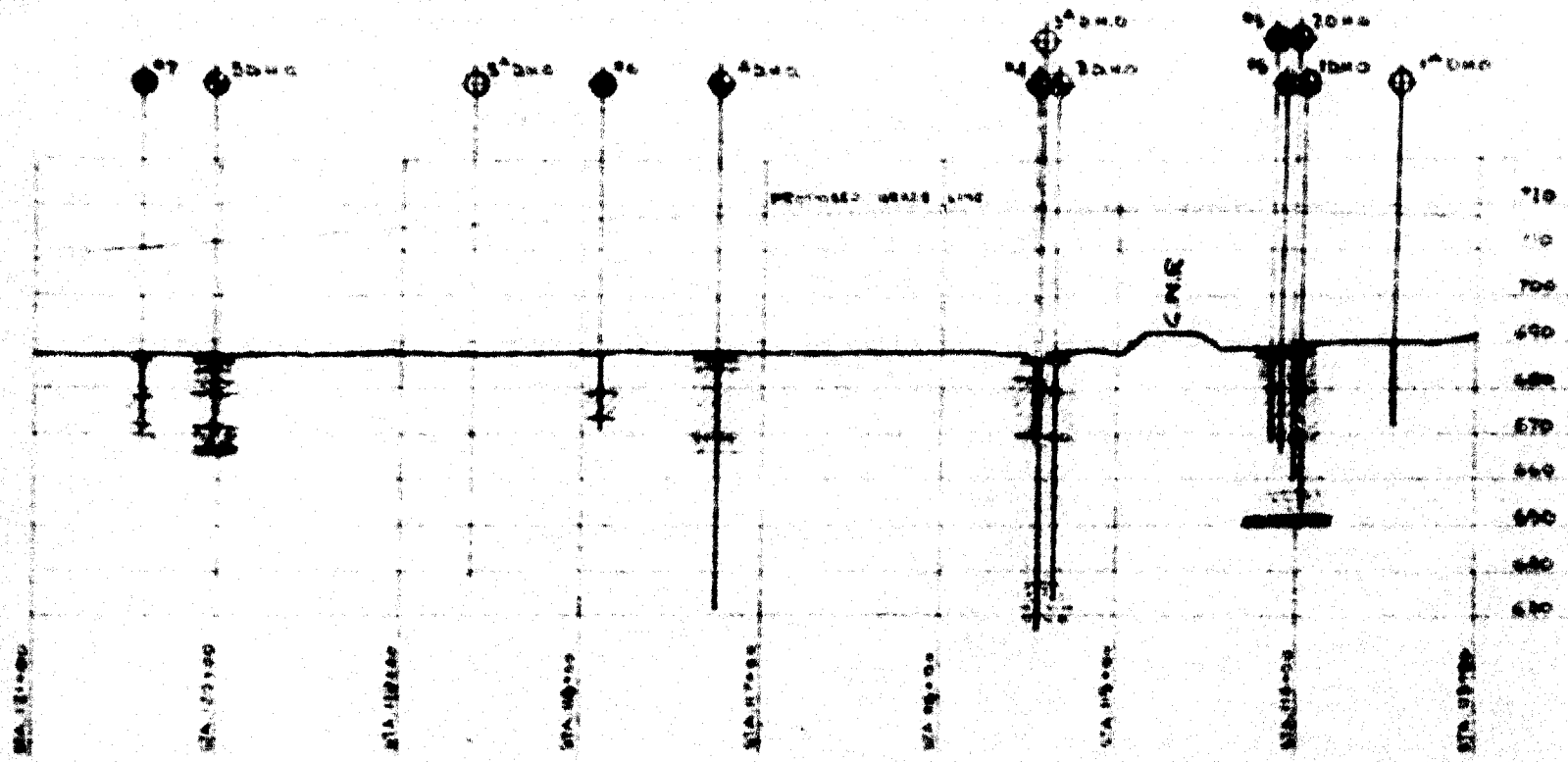




SKETCH - PLAN SHOWING LOCATION OF BORE HOLES  
AT SITE OF PROPOSED OVERPASS - LANDER, ONT.

SCALE: 1"=100'

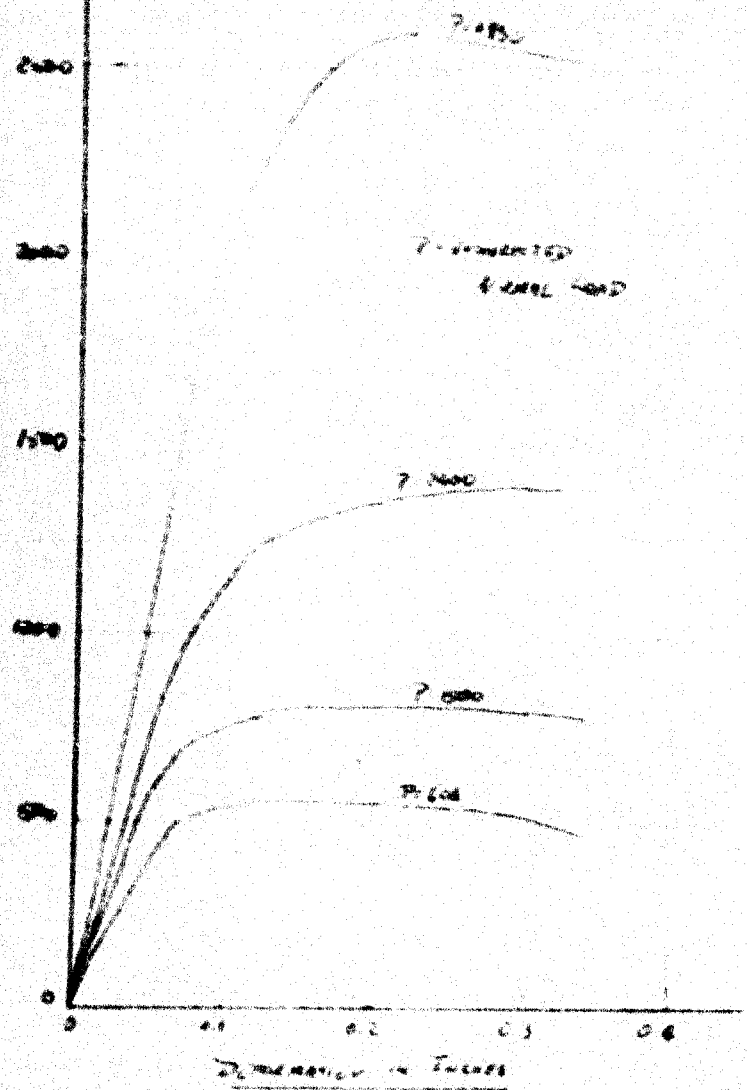




PROFILE SHOWING SOIL STRATIFICATION  
ALONG REVISION LINE 'D'  
FOR PROPOSED C.M.S. OVERPASS AT  
CALLANDER, ONTARIO.

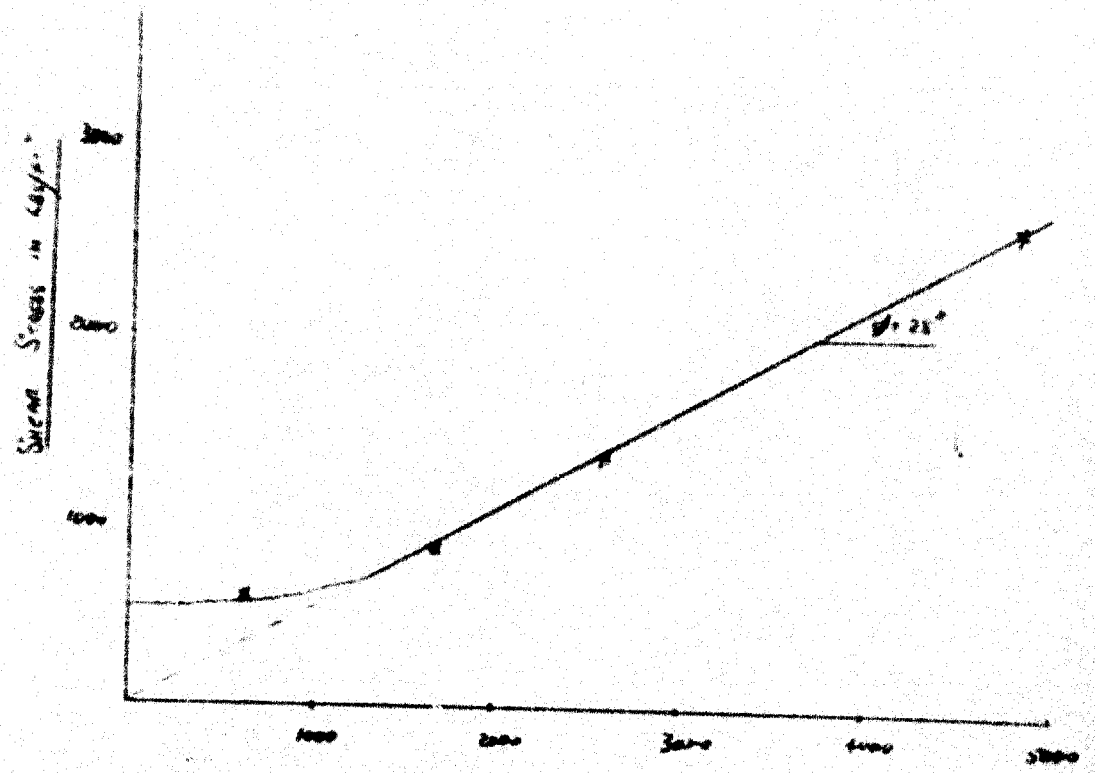
DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT





Shear Stress in  $\text{kg/cm}^2$

Deformation in Inches



Shear Stress in  $\text{kg/cm}^2$

Normal Load in P.F.

Direct Shear Test Results.



TABLE OF LABORATORY AND FIELD RESULTS

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

HOLE	SAMPLE	DEPTH FT.	LONG IN. FT.	V <sub>c</sub>	Q <sub>u</sub>	S	W.C.	L.L.	P.L.	P.L.	L.L.	7	Q <sub>u</sub> c P <sub>c</sub>	DESCRIPTION
1	TW3	10-11.5		875	948	6.0	65.0	35.2	10.2	17.1		105		Sensitive <sup>loose</sup> and brown clay.
				90	932							100		
1	TW3	12-16.9		920	845	7.0	110	30	31.2	26.8		95.5		clay
				132	930							90.0		
2	TW3	10-11.5		572	870	5.2	30.5					107		clay
				110										
2	TW3	12-16.5		440	880	4.0	39.2					114		clay
				110										
3	TW2	8-10.5		208	830	2.7	30.4					99.5		clay
				110										
3	TW3	12-12.5		190	190	1.4	33.5	66.5	23.0	42.0		90.0		clay
				143										

## LEGEND

T.W. = Test Well

S.S. = Split Spore

Q<sub>u</sub> = Unconfined CompressionP<sub>c</sub> = Pore Pressure

S = Sensitivity

W.C. = Natural Water Content

L.L. = Liquid Limit

P.L. = Plastic Limit

P.L. = Plasticity Index

L.L. = Liquid Limit

7 = Natural Unit Wt.

Q<sub>u</sub> = Quasi UnconfinedP<sub>c</sub> = Confined Pressure



Sum Stress in  $\text{Lbs}/\text{ft}^2$

3000  
2000  
1000  
0

EFFECTIVE STRESS DIAGRAM FOR  
THE CALCULATION OF ALLOWABLE  
PURE PRESSURES

$\phi = 28^\circ$

$\frac{1}{12}$

1000

2000

3000  
3500

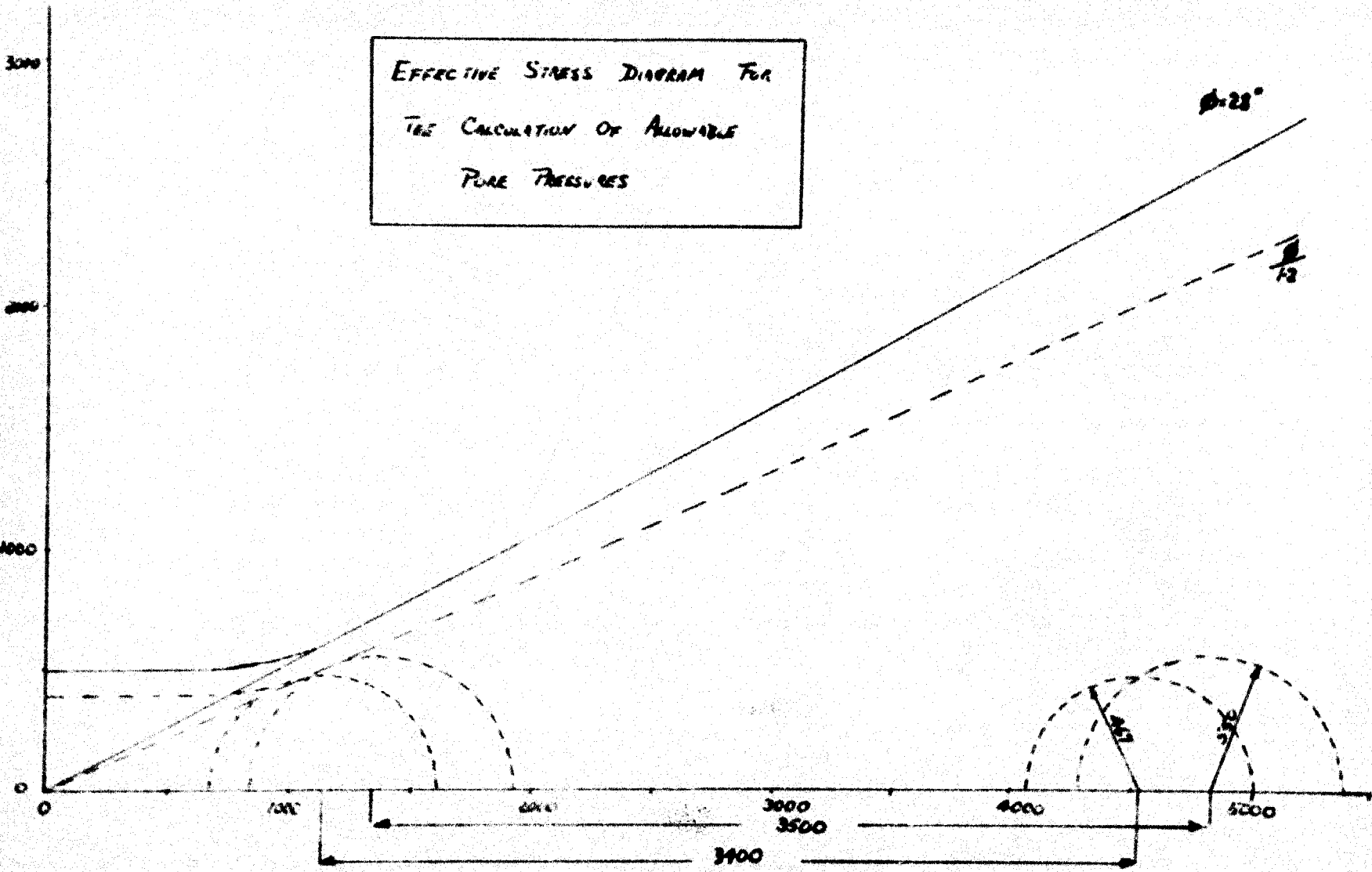
4000

5000

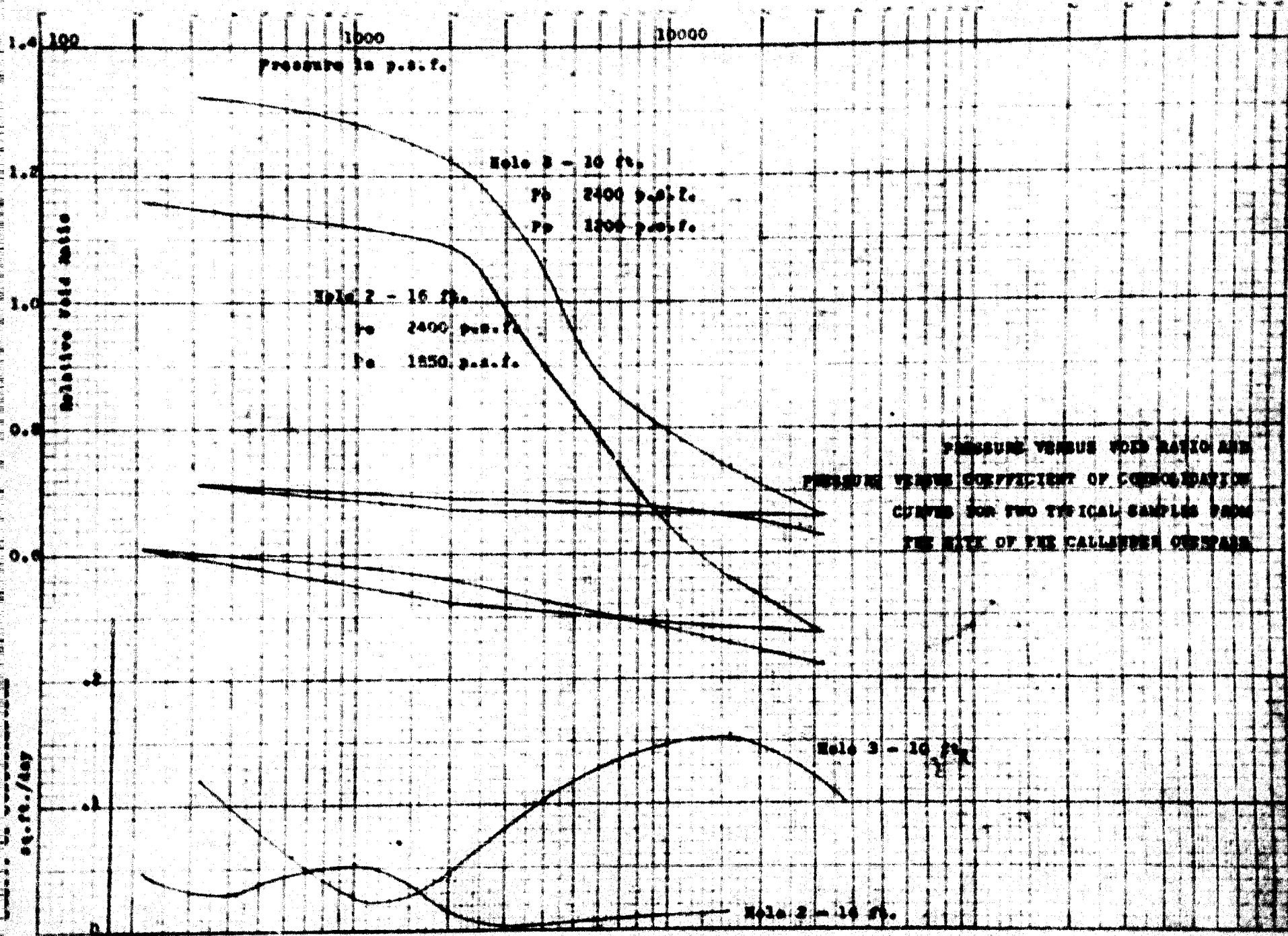
3400

Pore Pressure Stress in  $\text{Lbs}/\text{ft}^2$

Effective  $\phi$









DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

**SECRET**

S = Sensitivity  
M.C. = Natural Water Content  
L.L. = Liquid Limit  
P.L. = Plastic Limit

7.1. - Flushing Tank  
L.T. - Lighting Tank  
7 - Manual Feed P.  
M - Water Main  
P - Pump



Racey, MacCollum & Associates Ltd.

**TABLE OF LABORATORY AND FIELD RESULTS DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT**

NO.	TEST	DATE	W.C.	U.C.	S	L.L.	P.L.	W.C.	U.C.	7	DESCRIPTION
1	TW	10-17.5	462	700	5.2	42.0				104	Sensitive clay and brown clay
			88								
4	TW	10-11.5	> 2200	-	> 5	30.2				-	clay
			440								
4	TW	15-10.5	572	120	1.05	76.4	51.5	22.4	29.1	90	clay
			300								
5	TW	10-11.5	530	300	0.25	60.0				100	clay
			88								
5	TW	15-10.5	702	995	4.5	72.5	61.3	25.2	30.1	90	clay
			176								
6	TW	10-11.5	1100	1100	> 2.0	40.2				105.5	clay
			410								
7	TW	10-11.5	900	1595	7.5	32.4				110	
			132								

**DEFINITIONS**

W.C. = Water Content

U.C. = Unconfined Compression

S = Sensitivity

L.L. = Liquid Limit

P.L. = Plastic Limit

7 = Natural Solid Wt.

clay = Sensitive clay



**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole 1

Project: Foundation Investigation For Proposed Overpass

Location: Callander, Ontario

Hole location: See Enclosure No. 1

Hole Elevation and Datum

Field Work Begun

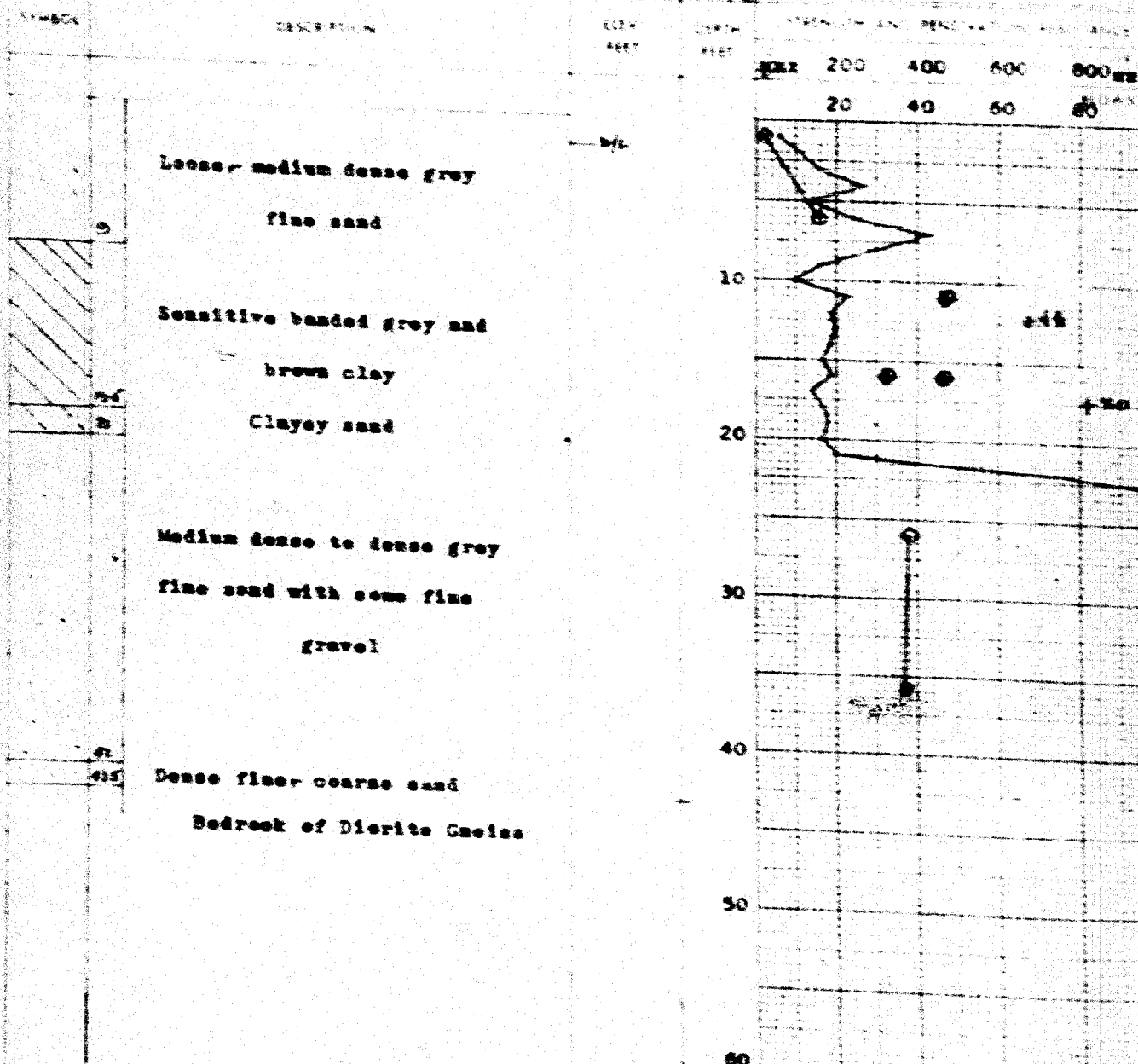
Ended

Field Supervisor A.S.

Driller P.V.

Prep P.M.

Checked





Sampling Method

1. Digging in the

2. Shovel

3. Hand

4. ...

5. ...

NI

...

...

...

...

...

...

...

...

...

SAMPLE

NATURAL  
UNIT WT  
PES

20 40 60 80 100 120

TW1

SS2

TW3

105  
109

TW4

95.5  
98.0

TW5

SS6

SS7

SS8

SS9

Lost

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT



**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: **2**Project: **Foundation Investigation for Proposed Overpass**Location: **Callander, Ontario**Hole Location: **See Enclosure No. 1**

Hole Elevation and Datum:

Field Work Begun

Field Supervisor: **A.H.**Driller: **F.V.**Prep.: **P.M.**

Checked:

Ended

**LEGEND**

Sampling Method:

2" Dia. split tube

2" Shell tube

Penetration Resistance

7' Split tube

7' Dia. tube

**XX** Casing

Strength

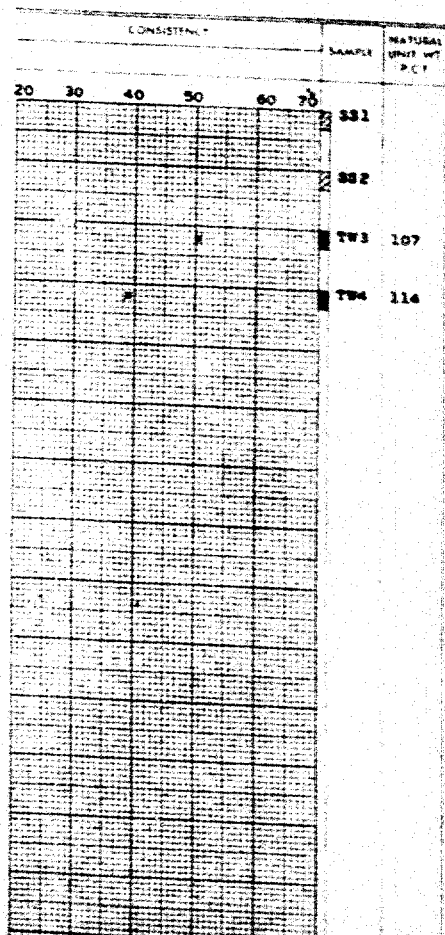
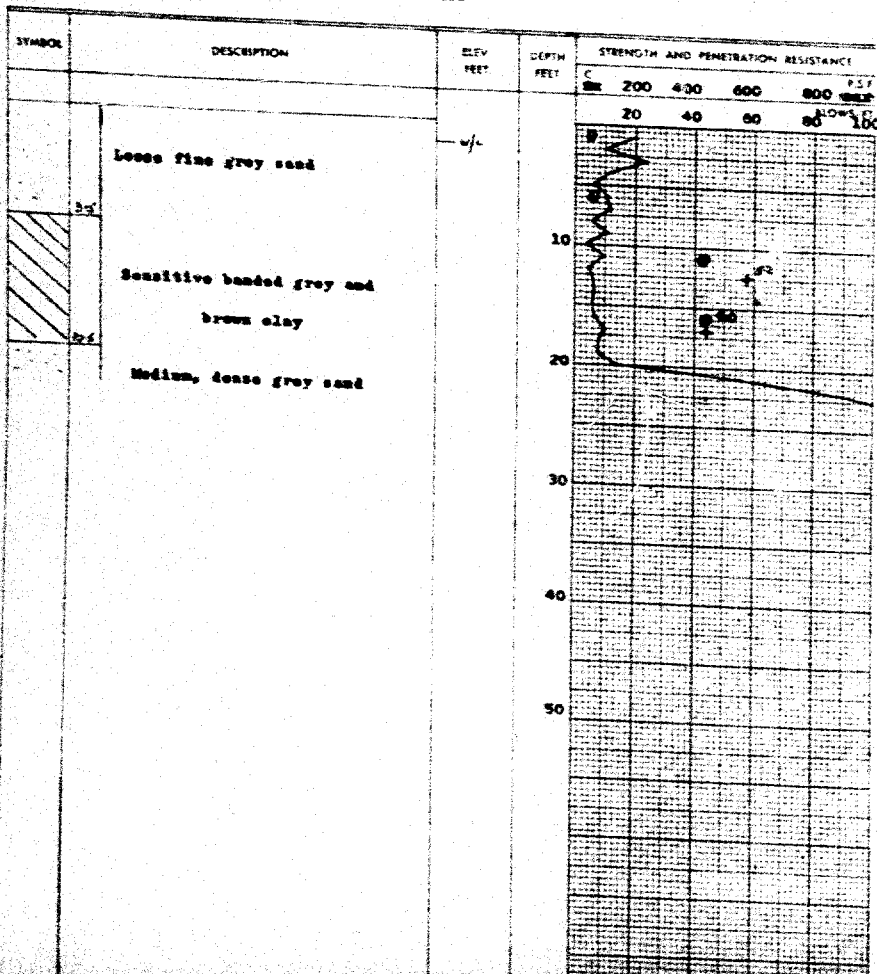
8' Unsat. test components (Q, U)  
None test C and secondary S

Consistency

Moisture measure and

Liquid limit (LL) and plasticity (PI)

Plasticity (PI)





**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3Project: **Foundation Investigation for Proposed Overpass**Location: **Callander, Ontario**Hole Location: **See Enclosure No. 1**

Hole Elevation and Datum:

Field Work Begun

Field Supervisor: **A.H.**Driller: **P.V.**Prep.: **P.W.**

Checked:

Ended

**LEGEND**

Sampling method:  
 1. Thin split tube  
 2. Shelby tube  
 3. Standard Penetration  
 4. Split tube  
 5. Split tube  
 6. Other

**NOTES**

1. Undisturbed specimens for  
 compression and shear tests  
 taken at depths of 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150, 160, 170, 180, 190, 200, 210, 220, 230, 240, 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350, 360, 370, 380, 390, 400, 410, 420, 430, 440, 450, 460, 470, 480, 490, 500, 510, 520, 530, 540, 550, 560, 570, 580, 590, 600, 610, 620, 630, 640, 650, 660, 670, 680, 690, 700, 710, 720, 730, 740, 750, 760, 770, 780, 790, 800, 810, 820, 830, 840, 850, 860, 870, 880, 890, 900, 910, 920, 930, 940, 950, 960, 970, 980, 990, 1000, 1010, 1020, 1030, 1040, 1050, 1060, 1070, 1080, 1090, 1100, 1110, 1120, 1130, 1140, 1150, 1160, 1170, 1180, 1190, 1200, 1210, 1220, 1230, 1240, 1250, 1260, 1270, 1280, 1290, 1300, 1310, 1320, 1330, 1340, 1350, 1360, 1370, 1380, 1390, 1400, 1410, 1420, 1430, 1440, 1450, 1460, 1470, 1480, 1490, 1500, 1510, 1520, 1530, 1540, 1550, 1560, 1570, 1580, 1590, 1600, 1610, 1620, 1630, 1640, 1650, 1660, 1670, 1680, 1690, 1700, 1710, 1720, 1730, 1740, 1750, 1760, 1770, 1780, 1790, 1800, 1810, 1820, 1830, 1840, 1850, 1860, 1870, 1880, 1890, 1900, 1910, 1920, 1930, 1940, 1950, 1960, 1970, 1980, 1990, 2000, 2010, 2020, 2030, 2040, 2050, 2060, 2070, 2080, 2090, 2100, 2110, 2120, 2130, 2140, 2150, 2160, 2170, 2180, 2190, 2200, 2210, 2220, 2230, 2240, 2250, 2260, 2270, 2280, 2290, 2300, 2310, 2320, 2330, 2340, 2350, 2360, 2370, 2380, 2390, 2400, 2410, 2420, 2430, 2440, 2450, 2460, 2470, 2480, 2490, 2500, 2510, 2520, 2530, 2540, 2550, 2560, 2570, 2580, 2590, 2600, 2610, 2620, 2630, 2640, 2650, 2660, 2670, 2680, 2690, 2700, 2710, 2720, 2730, 2740, 2750, 2760, 2770, 2780, 2790, 2800, 2810, 2820, 2830, 2840, 2850, 2860, 2870, 2880, 2890, 2900, 2910, 2920, 2930, 2940, 2950, 2960, 2970, 2980, 2990, 3000, 3010, 3020, 3030, 3040, 3050, 3060, 3070, 3080, 3090, 3100, 3110, 3120, 3130, 3140, 3150, 3160, 3170, 3180, 3190, 3200, 3210, 3220, 3230, 3240, 3250, 3260, 3270, 3280, 3290, 3300, 3310, 3320, 3330, 3340, 3350, 3360, 3370, 3380, 3390, 3400, 3410, 3420, 3430, 3440, 3450, 3460, 3470, 3480, 3490, 3500, 3510, 3520, 3530, 3540, 3550, 3560, 3570, 3580, 3590, 3600, 3610, 3620, 3630, 3640, 3650, 3660, 3670, 3680, 3690, 3700, 3710, 3720, 3730, 3740, 3750, 3760, 3770, 3780, 3790, 3800, 3810, 3820, 3830, 3840, 3850, 3860, 3870, 3880, 3890, 3900, 3910, 3920, 3930, 3940, 3950, 3960, 3970, 3980, 3990, 4000, 4010, 4020, 4030, 4040, 4050, 4060, 4070, 4080, 4090, 4100, 4110, 4120, 4130, 4140, 4150, 4160, 4170, 4180, 4190, 4200, 4210, 4220, 4230, 4240, 4250, 4260, 4270, 4280, 4290, 4300, 4310, 4320, 4330, 4340, 4350, 4360, 4370, 4380, 4390, 4400, 4410, 4420, 4430, 4440, 4450, 4460, 4470, 4480, 4490, 4500, 4510, 4520, 4530, 4540, 4550, 4560, 4570, 4580, 4590, 4600, 4610, 4620, 4630, 4640, 4650, 4660, 4670, 4680, 4690, 4700, 4710, 4720, 4730, 4740, 4750, 4760, 4770, 4780, 4790, 4800, 4810, 4820, 4830, 4840, 4850, 4860, 4870, 4880, 4890, 4900, 4910, 4920, 4930, 4940, 4950, 4960, 4970, 4980, 4990, 5000, 5010, 5020, 5030, 5040, 5050, 5060, 5070, 5080, 5090, 5100, 5110, 5120, 5130, 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6800, 6810, 6820, 6830, 6840, 6850, 6860, 6870, 6880, 6890, 6900, 6910, 6920, 6930, 6940, 6950, 6960, 6970, 6980, 6990, 7000, 7010, 7020, 7030, 7040, 7050, 7060, 7070, 7080, 7090, 7100, 7110, 7120, 7130, 7140, 7150, 7160, 7170, 7180, 7190, 7200, 7210, 7220, 7230, 7240, 7250, 7260, 7270, 7280, 7290, 7300, 7310, 7320, 7330, 7340, 7350, 7360, 7370, 7380, 7390, 7400, 7410, 7420, 7430, 7440, 7450, 7460, 7470, 7480, 7490, 7500, 7510, 7520, 7530, 7540, 7550, 7560, 7570, 7580, 7590, 7600, 7610, 7620, 7630, 7640, 7650, 7660, 7670, 7680, 7690, 7700, 7710, 7720, 7730, 7740, 7750, 7760, 7770, 7780, 7790, 7800, 7810, 7820, 7830, 7840, 7850, 7860, 7870, 7880, 7890, 7900, 7910, 7920, 7930, 7940, 7950, 7960, 7970, 7980, 7990, 8000, 8010, 8020, 8030, 8040, 8050, 8060, 8070, 8080, 8090, 8100, 8110, 8120, 8130, 8140, 8150, 8160, 8170, 8180, 8190, 8200, 8210, 8220, 8230, 8240, 8250, 8260, 8270, 8280, 8290, 8300, 8310, 8320, 8330, 8340, 8350, 8360, 8370, 8380, 8390, 8400, 8410, 8420, 8430, 8440, 8450, 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14400, 14410, 14420, 14430, 14440, 14450, 14460, 14470, 14480, 14490, 14500, 14510, 14520, 14530, 14540, 14550, 14560, 14570, 14580, 14590, 14600, 14610, 14620, 14630, 14640, 14650, 14660, 14670, 14680, 14690, 14700, 14710, 14720, 14730, 14740, 14750, 14760, 14770, 14780, 14790, 14800, 14810, 14820, 14830, 14840, 14850, 14860, 14870, 14880, 14890, 14900, 14910, 14920, 14930, 14940, 14950, 14960, 14970, 14980, 14990, 15000, 15010, 15020, 15030, 15040, 15050, 15060, 15070, 15080, 15090, 15100, 15110, 15120, 15130, 15140, 15150, 15160, 15170, 15180, 15190, 15200, 15210, 15220, 15230, 15240, 15250, 15260, 15270, 15280, 15290, 15300, 15310, 15320, 15330, 15340, 15350, 15360, 15370, 15380, 15390, 15400, 15410, 15420, 15430, 15440, 15450, 15460, 15470, 15480, 15490, 15500, 15510, 15520, 15530, 15540, 15550, 15560, 15570, 15580, 15590, 15600, 15610, 15620, 15630, 15640, 15650, 15660, 15670, 15680, 15690, 15700, 15710, 15720, 15730, 15740, 15750, 15760, 15770, 15780, 15790, 15800, 15810, 15820, 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**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: Foundation Investigation for Proposed Overpass

Location: Callander, Ontario

Male location See Enclosure No. 1

### Hole Elevation and Datum:

### Field Work Begun

Field Supervisor: A.H.

3.8.

Prop: P.M.

Checked:

## Endnote

## LEGEND

**Security Watch**

**1. The first**

3. *Phragmites* *communis* L.

**Abstract**

2000 2001 2002 2003 2004 2005 2006 2007 2008 2009 2010 2011 2012 2013 2014 2015 2016 2017 2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037 2038 2039 2040 2041 2042 2043 2044 2045 2046 2047 2048 2049 2050 2051 2052 2053 2054 2055 2056 2057 2058 2059 2060 2061 2062 2063 2064 2065 2066 2067 2068 2069 2070 2071 2072 2073 2074 2075 2076 2077 2078 2079 2080 2081 2082 2083 2084 2085 2086 2087 2088 2089 2090 2091 2092 2093 2094 2095 2096 2097 2098 2099 2100 2101 2102 2103 2104 2105 2106 2107 2108 2109 2110 2111 2112 2113 2114 2115 2116 2117 2118 2119 2120 2121 2122 2123 2124 2125 2126 2127 2128 2129 2130 2131 2132 2133 2134 2135 2136 2137 2138 2139 2140 2141 2142 2143 2144 2145 2146 2147 2148 2149 2150 2151 2152 2153 2154 2155 2156 2157 2158 2159 2160 2161 2162 2163 2164 2165 2166 2167 2168 2169 2170 2171 2172 2173 2174 2175 2176 2177 2178 2179 2180 2181 2182 2183 2184 2185 2186 2187 2188 2189 2190 2191 2192 2193 2194 2195 2196 2197 2198 2199 2200 2201 2202 2203 2204 2205 2206 2207 2208 2209 2210 2211 2212 2213 2214 2215 2216 2217 2218 2219 2220 2221 2222 2223 2224 2225 2226 2227 2228 2229 2230 2231 2232 2233 2234 2235 2236 2237 2238 2239 2240 2241 2242 2243 2244 2245 2246 2247 2248 2249 2250 2251 2252 2253 2254 2255 2256 2257 2258 2259 2260 2261 2262 2263 2264 2265 2266 2267 2268 2269 2270 2271 2272 2273 2274 2275 2276 2277 2278 2279 2280 2281 2282 2283 2284 2285 2286 2287 2288 2289 2290 2291 2292 2293 2294 2295 2296 2297 2298 2299 2300 2301 2302 2303 2304 2305 2306 2307 2308 2309 2310 2311 2312 2313 2314 2315 2316 2317 2318 2319 2320 2321 2322 2323 2324 2325 2326 2327 2328 2329 2330 2331 2332 2333 2334 2335 2336 2337 2338 2339 2340 2341 2342 2343 2344 2345 2346 2347 2348 2349 2350 2351 2352 2353 2354 2355 2356 2357 2358 2359 2360 2361 2362 2363 2364 2365 2366 2367 2368 2369 2370 2371 2372 2373 2374 2375 2376 2377 2378 2379 2380 2381 2382 2383 2384 2385 2386 2387 2388 2389 2390 2391 2392 2393 2394 2395 2396 2397 2398 2399 2400 2401 2402 2403 2404 2405 2406 2407 2408 2409 2410 2411 2412 2413 2414 2415 2416 2417 2418 2419 2420 2421 2422 2423 2424 2425 2426 2427 2428 2429 2430 2431 2432 2433 2434 2435 2436 2437 2438 2439 2440 2441 2442 2443 2444 2445 2446 2447 2448 2449 2450 2451 2452 2453 2454 2455 2456 2457 2458 2459 2460 2461 2462 2463 2464 2465 2466 2467 2468 2469 2470 2471 2472 2473 2474 2475 2476 2477 2478 2479 2480 2481 2482 2483 2484 2485 2486 2487 2488 2489 2490 2491 2492 2493 2494 2495 2496 2497 2498 2499 2500 2501 2502 2503 2504 2505 2506 2507 2508 2509 2510 2511 2512 2513 2514 2515 2516 2517 2518 2519 2520 2521 2522 2523 2524 2525 2526 2527 2528 2529 2530 2531 2532 2533 2534 2535 2536 2537 2538 2539 2540 2541 2542 2543 2544 2545 2546 2547 2548 2549 2550 2551 2552 2553 2554 2555 2556 2557 2558 2559 2560 2561 2562 2563 2564 2565 2566 2567 2568 2569 2570 2571 2572 2573 2574 2575 2576 2577 2578 2579 2580 2581 2582 2583 2584 2585 2586 2587 2588 2589 2590 2591 2592 2593 2594 2595 2596 2597 2598 2599 2600 2601 2602 2603 2604 2605 2606 2607 2608 2609 2610 2611 2612 2613 2614 2615 2616 2617 2618 2619 2620 2621 2622 2623 2624 2625 2626 2627 2628 2629 2630 2631 2632 2633 2634 2635 2636 2637 2638 2639 2640 2641 2642 2643 2644 2645 2646 2647 2648 2649 2650 2651 2652 2653 2654 2655 2656 2657 2658 2659 2660 2661 2662 2663 2664 2665 2666 2667 2668 2669 2670 2671 2672 2673 2674 2675 2676 2677 2678 2679 2680 2681 2682 2683 2684 2685 2686 2687 2688 2689 2690 2691 2692 2693 2694 2695 2696 2697 2698 2699 2700 2701 2702 2703 2704 2705 2706 2707 2708 2709 2710 2711 2712 2713 2714 2715 2716 2717 2718 2719 2720 2721 2722 2723 2724 2725 2726 2727 2728 2729 2730 2731 2732 2733 2734 2735 2736 2737 2738 2739 2740 2741 2742 2743 2744 2745 2746 2747 2748 2749 2750 2751 2752 2753 2754 2755 2756 2757 2758 2759 2760 2761 2762 2763 2764 2765 2766 2767 2768 2769 2770 2771 2772 2773 2774 2775 2776 2777 2778 2779 2780 2781 2782 2783 2784 2785 2786 2787 2788 2789 2790 2791 2792 2793 2794 2795 2796 2797 2798 2799 2800 2801 2802 2803 2804 2805 2806 2807 2808 2809 2810 2811 2812 2813 2814 2815 2816 2817 2818

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**Source of support:**

1997, 1998, 1999, 2000, 2001, 2002, 2003, 2004, 2005, 2006, 2007, 2008, 2009, 2010, 2011, 2012, 2013, 2014, 2015, 2016, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025, 2026, 2027, 2028, 2029, 2030, 2031, 2032, 2033, 2034, 2035, 2036, 2037, 2038, 2039, 2040, 2041, 2042, 2043, 2044, 2045, 2046, 2047, 2048, 2049, 2050, 2051, 2052, 2053, 2054, 2055, 2056, 2057, 2058, 2059, 2060, 2061, 2062, 2063, 2064, 2065, 2066, 2067, 2068, 2069, 2070, 2071, 2072, 2073, 2074, 2075, 2076, 2077, 2078, 2079, 2080, 2081, 2082, 2083, 2084, 2085, 2086, 2087, 2088, 2089, 2090, 2091, 2092, 2093, 2094, 2095, 2096, 2097, 2098, 2099, 2100, 2101, 2102, 2103, 2104, 2105, 2106, 2107, 2108, 2109, 2110, 2111, 2112, 2113, 2114, 2115, 2116, 2117, 2118, 2119, 2120, 2121, 2122, 2123, 2124, 2125, 2126, 2127, 2128, 2129, 2130, 2131, 2132, 2133, 2134, 2135, 2136, 2137, 2138, 2139, 2140, 2141, 2142, 2143, 2144, 2145, 2146, 2147, 2148, 2149, 2150, 2151, 2152, 2153, 2154, 2155, 2156, 2157, 2158, 2159, 2160, 2161, 2162, 2163, 2164, 2165, 2166, 2167, 2168, 2169, 2170, 2171, 2172, 2173, 2174, 2175, 2176, 2177, 2178, 2179, 2180, 2181, 2182, 2183, 2184, 2185, 2186, 2187, 2188, 2189, 2190, 2191, 2192, 2193, 2194, 2195, 2196, 2197, 2198, 2199, 2200, 2201, 2202, 2203, 2204, 2205, 2206, 2207, 2208, 2209, 2210, 2211, 2212, 2213, 2214, 2215, 2216, 2217, 2218, 2219, 2220, 2221, 2222, 2223, 2224, 2225, 2226, 2227, 2228, 2229, 2230, 2231, 2232, 2233, 2234, 2235, 2236, 2237, 2238, 2239, 2240, 2241, 2242, 2243, 2244, 2245, 2246, 2247, 2248, 2249, 2250, 2251, 2252, 2253, 2254, 2255, 2256, 2257, 2258, 2259, 2260, 2261, 2262, 2263, 2264, 2265, 2266, 2267, 2268, 2269, 2270, 2271, 2272, 2273, 2274, 2275, 2276, 2277, 2278, 2279, 2280, 2281, 2282, 2283, 2284, 2285, 2286, 2287, 2288, 2289, 2290, 2291, 2292, 2293, 2294, 2295, 2296, 2297, 2298, 2299, 2300, 2301, 2302, 2303, 2304, 2305, 2306, 2307, 2308, 2309, 2310, 2311, 2312, 2313, 2314, 2315, 2316, 2317, 2318, 2319, 2320, 2321, 2322, 2323, 2324, 2325, 2326, 2327, 2328, 2329, 2330, 2331, 2332, 2333, 2334, 2335, 2336, 2337, 2338, 2339, 2340, 2341, 2342, 2343, 2344, 2345, 2346, 2347, 2348, 2349, 2350, 2351, 2352, 2353, 2354, 2355, 2356, 2357, 2358, 2359, 2360, 2361, 2362, 2363, 2364, 2365, 2366, 2367, 2368, 2369, 2370, 2371, 2372, 2373, 2374, 2375, 2376, 2377, 2378, 2379, 2380, 2381, 2382, 2383, 2384, 2385, 2386, 2387, 2388, 2389, 2390, 2391, 2392, 2393, 2394, 2395, 2396, 2397, 2398, 2399, 2400, 2401, 2402, 2403, 2404, 2405, 2406, 2407, 2408, 2409, 2410, 2411, 2412, 2413, 2414, 2415, 2416, 2417, 2418, 2419, 2420, 2421, 2422, 2423, 2424, 2425, 2426, 2427, 2428, 2429, 2430, 2431, 2432, 2433, 2434, 2435, 2436, 2437, 2438, 2439, 2440, 2441, 2442, 2443, 2444, 2445, 2446, 2447, 2448, 2449, 2450, 2451, 2452, 2453, 2454, 2455, 2456, 2457, 2458, 2459, 2460, 2461, 2462, 2463, 2464, 2465, 2466, 2467, 2468, 2469, 2470, 2471, 2472, 2473, 2474, 2475, 2476, 2477, 2478, 2479, 2480, 2481, 2482, 2483, 2484, 2485, 2486, 2487, 2488, 2489, 2490, 2491, 2492, 2493, 2494, 2495, 2496, 2497, 2498, 2499, 2500, 2501, 2502, 2503, 2504, 2505, 2506, 2507, 2508, 2509, 2510, 2511, 2512, 2513, 2514, 2515, 2516, 2517, 2518, 2519, 2520, 2521, 2522, 2523, 2524, 2525, 2526, 2527, 2528, 2529, 2530, 2531, 2532, 2533, 2534, 2535, 2536, 2537, 2538, 2539, 2540, 2541, 2542, 2543, 2544, 2545, 2546, 2547, 2548, 2549, 2550, 2551, 2552, 2553, 2554, 2555, 2556, 2557, 2558, 2559, 2560, 2561, 2562, 2563, 2564, 2565, 2566, 2567, 2568, 2569, 2570, 2571, 2572, 2573, 2574, 2575, 2576, 2577, 2578, 2579, 2580, 2581, 2582, 2583, 2584, 2585, 2586, 2587, 2588, 2589, 2590, 2591, 2592, 2593, 2594, 2595, 2596, 2597, 2598, 2599, 2600, 2601, 2602, 2603, 2604, 2605, 2606, 2607, 2608, 2609, 2610, 2611, 2612, 2613, 2614, 2615, 2616, 2617, 2618, 2619, 2620, 2621, 2622, 2623, 2624, 2625, 2626, 2627, 2628, 2629, 2630, 2631, 2632, 2633, 2634, 2635, 2636, 2637, 2638, 2639, 2640, 2641, 2642, 2643, 2644, 2645, 2646, 2647, 2648, 2649, 2650, 2651, 2652, 2653, 2654, 2655, 2656, 2657, 2658, 2659, 2660, 2661, 2662, 2663, 2664, 2665, 2666, 2667, 2668, 2669, 2670, 2671, 2672, 2673, 2674, 2675, 2676, 2677, 2678, 26

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...and the

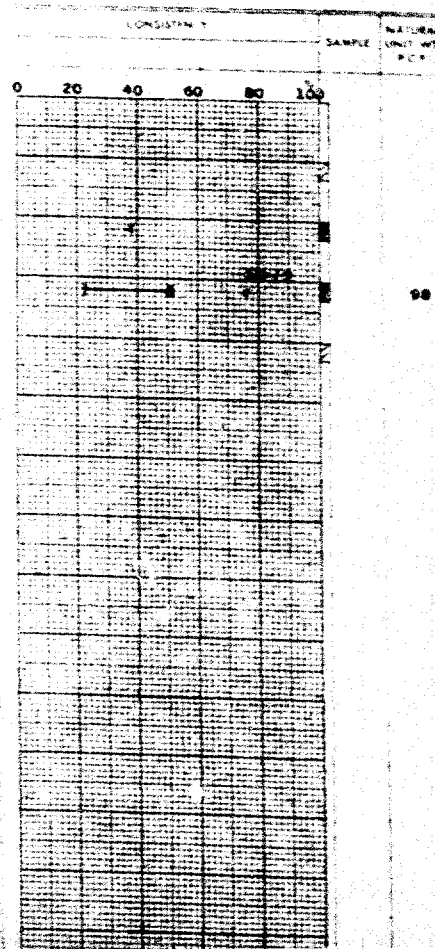
2000 151 2000 151

• **How to Write an Introduction**

$$\left(\frac{1}{2}\right)^n = \frac{1}{2^n} = \frac{1}{2^3} = \frac{1}{8}$$

20.  $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE				
				200	400	600	800	P.S.T. BLANKS
	Medium dense gray fine sand			20	40	60	80	BLANKS
	Thin sandy clay layer							
	Sensitive banded gray and							
	brown clay becoming silty clay							
	Fine gray sand							





**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole **5**Project: **Foundation Investigation for Proposed Overpass**Location: **Callander, Ontario**Hole Location: **See Enclosure No. 1**

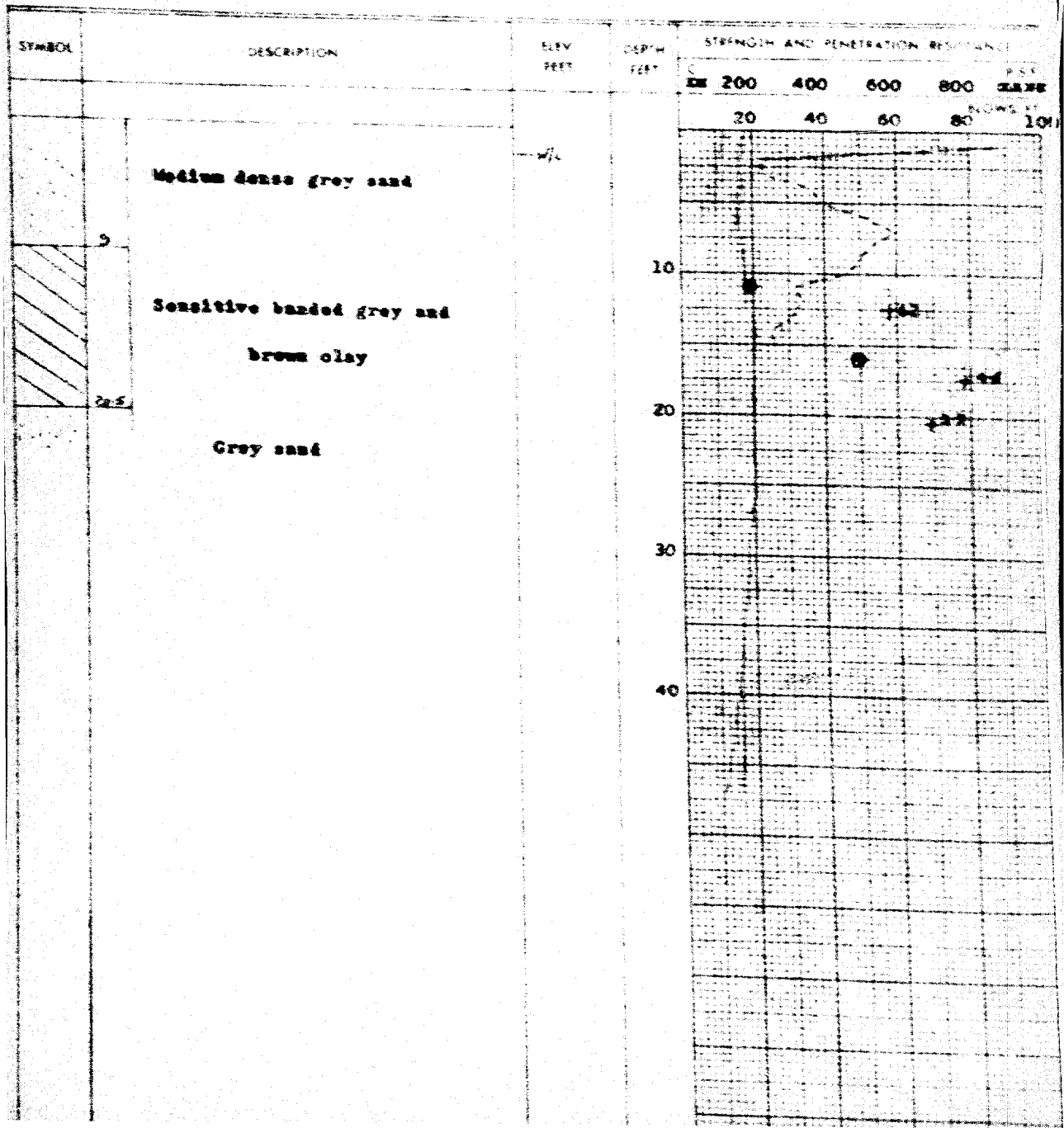
Hole Elevation and Datum:

Field Work Begun:

Field Supervisor: **A.H.**Driller: **F.V.**Prep: **P.M.**

Checked:

Ended









Order No. **1-58274-633**Enclosure No. **18****RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: **6**Project: **Foundation Investigation for Proposed Overpass**Location: **Collingwood, Ontario**Mole Location: **See Enclosure No. 1**

Mole Elevation and Datum:

Field Work Begun

Ended

Field Supervisor: **A.H.**Driller: **P.V.**Prep.: **P.M.**

Checked:

**LEGEND**

Sampling Method

1. Split tube

2. Shelby tube

Penetration Resistance

1. Split tube

2. Cone

**EX** Logging

Strength

1. Unconfined compressive strength test

2. Penetration

3. Natural moisture and

4. Liquid limit

5. Plastic limit

SYMBOL	DESCRIPTION	ELEV FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE				
				C 200	400	600	800	P.S.P. G.A.M.
				20	40	60	80	BLOWS/FT
	Loose fine gray sand							
	This clay layer at 6 feet							
	Sensitive banded gray and brown clay							
	Fine gray sand							

CONSISTENCY					SAMPLE	NATURAL UNIT WT P.C.F.
0	20	40	60	80		
					SS1	
					FW2	105.5
					FW3	







**RACEY MacCALLUM AND ASSOCIATES LTD.**

**Foundation Engineering Division**

## Engineering Data Sheet for Borehole: 7

Project: Foundation Investigation for Proposed Overpass  
Location: [illegible]

Location **Callander, Ontario**

Hole Location: See Enclosure No. 1

**Hole Elevation and Datum:**

**Field Work Begun**

Field Supervisor: A.H.

Driller: E. V.

Prep.: P.M.

Checked:

**Ended**

### LEGEND

### Sampling Method

7-30-42 tube

7- Shelby 1420

[illegible]

2- Spill 1000

2000

2000

320 卷一百一十五

## Learning and Comprehension Outcomes

Value first  $\bar{C}$  and uncertainty  $\Delta S$

**Company**




Not a relative and

**6. Variability Index**

1. *Explain the importance of the following factors in the development of a country's economy:*  
 (a) *Human resources*  
 (b) *Capital resources*  
 (c) *Technology*  
 (d) *Infrastructure*  
 (e) *Government policy*  
 (f) *International trade*  
 (g) *Investment*  
 (h) *Education*  
 (i) *Healthcare*  
 (j) *Environment*  
 (k) *Democracy*  
 (l) *Corruption*  
 (m) *Religion*  
 (n) *Culture*  
 (o) *Language*  
 (p) *History*  
 (q) *Geography*  
 (r) *Climate*  
 (s) *Disaster*  
 (t) *War*  
 (u) *Peace*  
 (v) *Stability*  
 (w) *Security*  
 (x) *Justice*  
 (y) *Equality*  
 (z) *Freedom*  
 (aa) *Democracy*  
 (ab) *Corruption*  
 (ac) *Religion*  
 (ad) *Culture*  
 (ae) *Language*  
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SYMBOL	DESCRIPTION	ELEV FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE				
				200	400	600	800	P.S.F. 1000
  	Loose gray sand							
	Sensitive banded gray and brown clay							
	Gray sand							