

#58-F-258C

W.P.# 617-56

C.N.R. OVERPASS

HUNTSVILLE

By-PASS

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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Project: C108/J109

January 6, 1958.

Dept. of Highways of Ontario,
Room 1424, East Block,
Parliament Bldgs.,
Toronto, Ont.

Attention: Mr. F. Brownridge

Piezometer Installation and Stability Analysis
C.N.R. Overpass - Station 29+00, Huntsville By-Pass

Dear Sirs:

Attached hereto is our report on the piezometer installation, installed early in September 1957, under the north embankment approach to the above-noted C.N.R. overpass. Included, also, is a revised analysis of the stability of this embankment during various stages of its construction.

For your convenience the conclusions of this report will be repeated again and are as follows:

- 1) Additional strength measurements have been made on the soil underlying the north embankment to the C.N.R. Overpass, south west of Huntsville, Ontario. Stability Analyses, based on a careful examination of this supplementary information and on the sections and materials actually in use, indicate that the embankment can be built safely to a height of about 15 feet before control of construction must be exerted. The controlling pore pressures at gauge level of El. 937.5 feet that apply after this height is exceeded are 8 p.s.i. for a height of twenty feet, reducing gradually to 4 p.s.i. for heights above thirty feet.
- 2) The piezometer installation appears to be operating satisfactorily although unit number 2 should be used for future construction control. Since slightly higher pore pressures may develop along the critical failure path at El. 916 feet than are indicated at piezometer level, the gauge pressures of 8 and 4 p.s.i., noted above, should not be exceeded under any circumstances.

We regret any inconvenience to you resulting from the delay in submitting this report. We shall be pleased to discuss any matters that may occur to you after the enclosed contents are reviewed.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

WAT/lt

Yours very truly,
W. A. Trow
William A. Trow (P. Eng.)

DEPARTMENT OF HIGHWAYS OF ONTARIO
ROOM 1422, EAST BLOCK,
PARLIAMENT BLDGS.,
TORONTO, ONTARIO

PIEZOMETER INSTALLATION AND STABILITY ANALYSIS
C.N.R. OVERPASS - STATION 29+00, HUNTSVILLE BY-PASS

TABLE OF CONTENTS

Field Installation Program	Page 1
Appraisal of Soil Strength	3
Review of Embankment Stability	5
Discussion of Piezometer Measurements	7
Conclusions	8

ENCLOSURES

Summary of Laboratory & Field Test results	Table 1
Sketch showing location of piezometers under embankment	Drawing 1
Sketch of tamping hammer & piezometer layout	2
Soil profiles holes 1 to 4	3 - 6
Quick Undrained Triaxial Test Results	7
Consolidation Test Results, Hole 2 - 14 ft.	8
Consolidated Quick Undrained Test Result	9
Test Results on samples of compacted clayey silt fill	10
Slow drained triaxial test results	11
Record of Measurements - piezometer unit 2	12
Sliding Block Analysis	13
Composite Arc Analysis	14
Photographs of piezometer installation and initial embankment compaction	

PIEZOMETER INSTALLATION AND STABILITY ANALYSIS
C.N.R. OVERPASS-STATION 29+00
HUNTSVILLE BY-PASS

This report contains a brief description of the piezometer installation made early in September under the north embankment approach to the C.N.R. Overpass approximately $\frac{1}{2}$ mile south-west of Huntsville, Ont. The results of laboratory tests on samples obtained during this installation program have also been given and a review of the stability of the embankment has been made on the basis of this information.

Field Installation Program

The installation underlying this portion of the overpass approaches consists of 4 piezometers which were placed at the locations shown on drawing number 1, Sept. 9 to Sept. 11, 1957. This report will be concerned only with installation and performance of the piezometers subsequent to Sept. 9th, and no description of the preparatory work involved in assembling and testing the units, making bentonite balls and other laboratory work will be given.

At the time field work was begun, the embankment site had been cleared of timber waste and top soil and had been backfilled to approximate EL. 932.5 feet with fine to medium sharp sand. The water table during the period of installation was approximately 6 inches below this level.

The first piezometer, later designated as unit number 4, was placed at station 29 +00, 25 feet right. The procedure for placing this and the remaining three units essentially conformed to the method outline by A. Casagrande in "Soil Mechanics in the Design and Construction of the Logan Airport", Journal Boston Society of Civil Engineers 1949, with the exception that undisturbed samples of soil were taken down to final piezometer depth before the units were installed. The results of this sampling program are recorded in the soil profiles of drawings 3 to 6 and are discussed later in the report.

For record purposes the installation procedure will be itemized as follows:

(1) A 10 foot length of 2 $\frac{1}{2}$ inch pipe was driven to the sand fill - natural clay contact which was found to lie between 6 $\frac{1}{2}$ and 7 feet below EL. 932.5 ft. Care was taken to ensure that the depth of sand backfill was definitely established. In order to ensure tight contact with the underlying clay, the bottom end of the pipe was unthreaded and had no drive coupling.

(2) The sand fill was washed out to the bottom of the casing and a two inch I.D. Shelby tube sample was obtained. Twelve inches below the lower penetration limit of this sampler a field vane test was made.

(3) The casing was then driven to the next sampling interval, carefully washed out and the aforementioned sampling procedure repeated. Samples were obtained to a depth of 16 feet; the casing was driven to this depth and carefully washed clean.

(4) After this was done, the casing was pulled back two feet and the space below it filled with a measured quantity of wet silica sand grading between the No. 20 and No. 40 mesh screens.

(5) Before placing the piezometer in the borehole, it and the connecting plastic tubing were completely filled with water by connecting the tubing to the wash-pump intake and immersing the piezometer in the water supply tank. When the piezometer was lowered into the casing, an excess head was maintained on the unit by holding a finger over the extreme end of the connecting tubing.

(6) When in place, the piezometer tubing was held down while the casing was withdrawn another two feet. The space between the piezometer and surrounding soil was filled with a measured quantity of wet silica sand.

(7) The casing was pulled up another foot, or to El. 921.5 feet and saturated silica sand was poured in to a depth of approximately 2 feet above its lower surface. This sand was tamped lightly using the special hammer shown in drawing 2.

(8) The casing was filled with water to a height three inches from the top and bentonite balls were dropped in until the water just began to overflow the pipe. These balls were the size of marbles and were rolled from bentonite mixed just above its plastic limit.

(9) Approximately 1 inch of roofers gravel was dropped on the bentonite and the mass was compacted under 20 blows of the tamping hammer. This procedure was repeated until 5 layers of bentonite had been placed.

(10) Another 2 feet of sand was placed on top of this bentonite seal after which time 5 more layers of bentonite were compacted in place. Then the pipe was filled to the top with sand. At this point the top of the 2½ inch pipe was 1 foot below ground surface or at approximate el. 931.5 ft.

(11) Shallow trenches were dug from each pipe along the paths shown in drawing 1 and the plastic tubing from each piezometer was laid in them. In order to encourage the movement of air bubbles out of the tubing the trenches were graded up from the pipe. The housing for the bourdon gauges which terminated the tubing was placed at El. 937.5 ft. on a ramp of compacted sand. Great care was taken to remove all bubbles from the lines, as is shown in the accompanying photographs.

(12) When this work had been completed satisfactorily the trenches were backfilled to approximate El. 934 ft. with sand and each piezometer location was clearly marked. Instructions were given to have the entire piezometer area covered carefully with another two feet of sand before compaction equipment was moved over it.

(13) Because the $\frac{1}{2}$ inch I.D. plastic tubing to each piezometer came in 100 foot lengths and because a permanent seal between each Bourdon gauge connection and the plastic pipe had been made prior to the field program, a connection had to be made in each line half way along the ramp to the gauge housing. This was done by coating a short length of $\frac{1}{2}$ inch O.D. Saran tubing with a plastic glue and then forcing it into the open ends of the piezometer tubing. This glue had been used in the connection to each Bourdon gauge and was found to be leak proof at least up to a maximum pressure of 15 p.s.i.

(14) The final step in the installation consisted of topping-up the piezometer lines with water and then connecting each Bourdon gauge. Since the gauge ends of the piezometer lines were about 5 feet above ground water level, there was sufficient excess pressure to cause a slight flow of water from the piezometers into the surrounding ground. Therefore it was difficult to keep the lines completely filled before the gauges were connected. However after some effort, satisfactory connections were obtained although a slight negative pressure was exerted on each gauge. The threads of the gauges were coated with "plumbers stick" before final connections were made in order to prevent leakage during the recording of positive water pressures.

(15) Prior to freeze-up the piezometer housing and approach trench was thoroughly insulated to prevent freezing of the lines. Despite this precaution the lines in the housing began to freeze early in December. This condition was remedied on Dec. 14th, at which time the gauges were disconnected and tested, the top two feet of line filled with anti-freeze and the units reconnected. Unit number 2, placed at Station 30+00, 25 ft. right, had increased in pressure from 1.9 to 3.0 p.s.i. during this freezing period and water flowed slowly from the line when the gauge was disconnected. When reconnected the pressure returned to about 2 p.s.i. within 5 minutes. Units 1 and 4 were found to contain water up to the gauge level when disconnected, unit 3 was down about 1 foot below this connection. All Bourdon gauges were found to register pressure accurately. Instructions were given to have a Coleman lamp to heat the inside of the housing for a short period each day in order to prevent the recurrence of freezing conditions.

Appraisal of Soil Strength

As stated earlier in the report a limited number of undisturbed samples were taken during the piezometer installation in order to supplement information obtained from a previous investigation, and, in particular, to establish the slow drained strength characteristics of the foundation clay. The results of the tests on these samples are recorded in several of the enclosed drawings and are summarized in table 1. An inspection of this table shows field vane and undrained triaxial test measurements which are generally somewhat higher than the values noted in the previous investigation by Racey MacCallum & Assoc. Since the results

of these tests are used in the analysis of the stability of the embankment for the condition of zero drainage or consolidation during the application of load, the accuracy of the analysis will be affected to a marked degree by the design values of shear strength selected. An examination of the information available from each investigation therefore, may help to resolve this uncertainty.

In the first investigation, reported by Rasey MacCallum & Assoc., field vane shear strengths in the softest zone between El. 920 and 912 feet were found to range from a low of about 180 p.s.f. to values slightly above 800 p.s.f. These results were obtained in four widely spaced borings along centre line north and south of the C.N.R. tracks. The lowest value of 180 p.s.f. was measured at El. 914 ft. in hole 1 located at Station 30+00 feet. At approximate elevation 919 feet in this hole the vane strength was 240 p.s.f.

The lowest field vane strength measured during this piezometer program was at a depth of 16 feet or El. 916.5 feet in hole 4, located at Station 30+00, 25 feet right. The value noted here was 277 p.s.f. and the piezometer unit at this location registered the highest pore pressure during embankment construction. The vane test results below El. 920 ft. in all 4 piezometer holes, which are adjacent to the aforementioned hole 1, ranged from this value up to 700 p.s.f. and the average of 8 tests was approximately 470 p.s.f. The corresponding two undrained triaxial test results, recorded in table 1, had values of 330 and 520 p.s.f. Recent experience has indicated that unconfined compression tests are unreliable and too conservative in varved clays of this nature.

The conclusion to be drawn from this more recent and closer examination of the variation in soil strength around station 30+00, is that the value of 200 p.s.f. cohesion used in the original stability analysis of this embankment is perhaps too conservative. An examination of other information will confirm this opinion.

In each investigation, there was evidence that the natural soil has been pre-consolidated under stresses greater than are acting at the present time. In piezometer holes 1, 2 and 4, this was demonstrated above approximate El. 922 feet by higher shear strengths and by a lower relative moisture content within the plasticity range of the soil. These conditions are indicative of surface drying due to exposure at some period subsequent to deposition. In hole 2 the soil at a depth of 7 to 8 feet was oxidized to a brown colour.

Further evidence that the clay deposit has been consolidated under stresses greater than those exerted by existing overburden loads or that the effects of surface desiccation have been felt at greater depths, is provided by the consolidation test result shown in drawing number 8. This test, made on a sample of the more plastic clay from a depth of 14 feet in piezometer hole 2, indicates a preconsolidation pressure of 1500 p.s.f., which is about 600 p.s.f. higher than existed before embankment construction was begun. The corresponding average

of field vane and undrained triaxial shear tests at this depth is 545 p.s.f. which value is 0.36 times the pre-consolidation pressure of 1500 p.s.f. In the siltier material of hole number 3, the field vane strength was approximately 520 p.s.f.; this value probably errs on the high side because the silt would generate some internal friction during shear. The consolidated quick undrained test recorded in drawing 9 shows that this silty soil increases in shearing resistance to 1150 p.s.f. when almost completely consolidated under 10 p.s.i. pressure. This increase in strength is equivalent to 0.36 times the effective consolidating pressure. Using these relationships as a guide it will be found that even for the overburden conditions of Sept. 10, 1957, the lowest possible shearing strength in the soft material at a depth of 14 feet is about 330 p.s.f.

On the basis of the foregoing reasoning, therefore, the revised stability analysis, which follows later in this report, has been based upon higher shear strength values than were assumed in the original analysis. An average shear strength of 450 p.s.f. was used for the clay down to a depth of 12 feet or to approximate El. 920.5 feet and a value of 300 p.s.f. was taken for the softer, more plastic clay found below this level. The stiffer conditions noted below approximate elevation 912 feet in the original report by Racey MacCallum and Assoc. were assumed to be valid.

In drawing number 11 the results of the slow-drained triaxial shear tests are presented. This information was required in order to establish the maximum permissible or controlling excess pore pressure that can be tolerated as embankment construction proceeds. Two sets of tests were carried out. One, on the less plastic material noted at higher levels in hole 2, indicated a drained shear strength parameter of $C' = 0$ and $\phi' = 24^\circ$. The other test, performed on the softer material found below approximate elevation 920 feet, showed a parameter defined by $C' = 0$ and $\phi' = 24^\circ$. The potential plane of failure lies in this latter material.

Review of Embankment Stability

Since the section and fill composition of the proposed embankment have been finalized, its stability during construction can be estimated with a greater degree of certainty than was possible before this information was available. At its present state of construction the embankment has been provided with 2:1 side slopes, and its entire foundation to a depth of about 2 feet consists of a sharp medium sand backfill. The first approximately 3 feet of the embankment consists of slightly cohesive silt which, because of excessive wet weather during borrow operation, exists in a compacted state a few percent above standard optimum conditions. In the higher sections of fill, this material will be replaced by sharp medium sand.

The shear strength properties for the embankment materials and for the natural varved clay underlying them are recorded in the analysis sheet, designated drawing 13. It will be noted that the cohesive silt, comprising

the 1st 8 feet of embankment, has been assigned an undrained cohesive resistance of 1000 p.s.f. This value is the conservative average of three undrained triaxial tests on specimens compacted to moisture and density conditions approximating those existing in the embankment. The shear strength characteristics assigned to the sand fill have been estimated on the basis of published experience for sand of this type.

The stability of the embankment has been checked in detail using the sliding block method of analysis. Since the weakest zone in the foundation is confined between approximate El. 921 ft. and El. 912 ft., this type of analysis probably reflects potential failure conditions most accurately. The factors of safety for various stages of construction are shown in drawing 13 for possible failure along El. 921 ft. and 5 feet below this level. Although more critical conditions would exist at El. 912 feet, drainage during construction to the underlying sand deposits would be more rapid at this depth and therefore consolidation to a more stable state should take place much sooner than at the levels analysed. In addition, the soil becomes somewhat stiffer and more resistant to shear at this theoretically more critical depth.

Although the sketches in drawing 13 show the computations for potential failure at El. 921 feet, the conditions noted for El. 916 ft. should be the basis for future construction control. Reference to the results for this level shows that the factor of safety against failure becomes less than unity for an embankment just under 20 feet in height.

The method used to determine the maximum permissible pore pressure for fill heights in excess of 15 feet conforms to the procedure outlined in "The Gain in Stability Due to Pore Pressure Dissipation in a Soft Clay Foundation", Skempton & Bishop, International Commission on Large Dams, 1955. It applies to conditions where shear stresses along potential planes of failure are restricted to a horizontal surface, the shear stress under these circumstances being the result of the unbalance between the active pressure exerted by the embankment and the passive resistance generated in the soil at the edge of the fill. The maximum permissible excess pore pressure in the soil is the value indicated when the circle representing the critical stress conditions is moved tangent to the effective strength envelope for the soil. In this case this shear parameter was provided with a factor of safety of 1.3.

It will be noted that the controlling pore pressure for an embankment height of 20 feet is 10.2 p.s.i. and for heights above 30 feet is 6.04 p.s.i. Since the piezometer recording system is founded at El. 937.5 ft., the corresponding readings at this gauge level would be 8 and 4 p.s.i. respectively.

In order to test the validity of the sliding block analysis, the factor of safety against failure for an embankment height of 35 feet has been checked using the composite circular arc method of analysis. The results of this investigation are recorded in drawing 14 and indicate

factors of safety somewhat greater than indicated by the sliding block. Therefore the results of the sliding block procedure should apply.

Discussion of Piezometer Measurements

The record of piezometer measurements to date has been presented in drawing number 12 for unit number 2, located at borehole number 4. This unit has been found to be the most active of the four installed and therefore it should be the basis for any future control. Unit number 4 at station 29+00, 25 feet right, has also registered a small amount of pressure and the excess head in the remaining two units is just below the gauge level of 937.5 feet which is equivalent to a pressure of about 2 p.s.i.

Reference to drawing 12 shows that the piezometer readings have responded quickly to the addition of load and that much of this excess pressure is dissipated at a relatively rapid rate. This initially fast dissipation probably takes place horizontally outward from the embankment along the thin silt and fine sand portions of this varved clay. However it will be noted that the rate of dissipation decreases considerably with time under any given increment of pressure. This may possibly represent a state of back pressure or resistance to additional water generated in the clay bounding the embankments. This resistance can be expected to increase as embankment construction proceeds. The alternative path for relief of excess pore pressure is vertically upward through the varved clay to the sand fill underlying the embankment. The permeability across the varves in this direction is much lower than exists along the varves horizontally. The estimated coefficient of consolidation in this direction based on laboratory test information from drawings 8 and 9, is 0.05 sq.ft. per day. For the period of 48 days up to Dec. 5 when the embankment height remained constant at 7.8 feet, the pore pressure at gauge level dissipated from about 3.0 to 1.9 p.s.i. or about 37%. The vertical path of flow from the bottom of the 2½ inch casing above piezometer level to the bottom of the sand fill represents a distance of about 4 feet. The corresponding coefficient of consolidation for this amount of dissipation across this path during a period of 48 days is approximately 0.1 sq.ft. per day. Assuming then that the rate of dissipation may be controlled by water movement in this vertical direction, it would take about ½ year for the pore pressure at el. 916 ft. to dissipate from the critical value of 6 p.s.i. to a lesser value of 5 p.s.i. It is unlikely however, that this condition of vertical flow will control construction, however, except possibly near the final phases of the work.

The reason for the lower pore pressures registered in units 1, 3 and 4 is somewhat difficult to account for. Possible explanations for this apparent inconsistency can be obtained upon reference to the logs for these borings shown in drawings 3 to 6. For record purposes it should be pointed out that units 1, 2, 3 and 4 were placed in boreholes 3, 4, 2 and 1 respectively. Although samples were not taken in borehole 4, the softest soil conditions at piezometer level were noted here. The soil

in the other borings was somewhat stiffer than at depth in this boring. This was particularly so in borehole 2 which houses unit number 3. If the soil around this unit has been preconsolidated slightly, as appears evident, less pore water will be squeezed out when load is applied. If the amount of excess water is small, rapid dissipation horizontally out through the silt varves could well reduce the pressure locally to an unmeasurable value. It will be noted also that the soil is quite silty around borehole 3, which houses unit 1. More permeable conditions and a more rapid rate of pressure dissipation could be expected here also.

Conclusions

The observations and comments presented in the foregoing sections can be summarized briefly as follows:

- 1) Additional strength measurements have been made on the soil underlying the north embankment to the C.N.R. Overpass, south-west of Huntsville, Ontario. Stability Analyses based on a careful examination of this supplementary information and on the sections and materials actually in use, indicate that the embankment can be built safely to a height of about 15 feet before control of construction must be exerted. The controlling pore pressures at gauge level of El. 937.5 feet that apply after this height is exceeded are 8 p.s.i. for a height of twenty feet, reducing gradually to 4 p.s.i. for heights above thirty feet.
- 2) The piezometer installation appears to be operating satisfactorily although unit number 2 should be used for future construction control. Since slightly higher pore pressures may develop along the critical failure path at El. 916 feet, than are indicated at piezometer level, the gauge pressures of 8 and 4 p.s.i. noted above should not be exceeded under any circumstances.

WAT/lt
January 6, 1958.
C108/J109

W. A. Trow
William A. Trow (P.Eng.)

TABLE NO. 1 -- SUMMARY OF LABORATORY AND FIELD TEST MEASUREMENTS

Elev Feet	Shearing Resistance p.s.f.				Consistency % dry weight									Natural Unit Weight pcf		
	Hole				hole 1			hole 2			hole 3			Hole		
	1	2	3	4	L.L.	P.L.	Wn	L.L.	P.L.	Wn	L.L.	P.L.	Wn	1	2	3
926																
925	416 Qu				55.5	29.5	42							95.5		
924	445 Qu	1770 Qu		1640 V				28	19.8	26			28.7	105	123	
923	610 V	2788 V	533 V	S=3.3												
	S=2.7	S=2	S=2.7													
922				410												
				S=5												
921		1204 Qu						24	18.9	30.3	24.2	19.8	28.4		129.5	130
										28.6					127	
920	308 V	564 V	523		26.5	21.5	28.8									
919	S=2.1	S=2.5	S=3.2	443												
918		520 Qu		S=1.4				40.3	23.6	47					112.5	
										50.6				110.1	111.0	
917	331 Qu	700 V	513		38	22.4	52.2							111.2		
916		S=3.8	S=2.5	277										110.3		
				S=2.3												
915	430 V															
	S=1.1															
914																

LEGEND: V = Vane strength
 S = Sensitivity
 Qu = Undrained Triaxial
 L.L. = Liquid Limit
 P.L. = Plastic Limit
 Wn = Natural Moisture Content

View looking north-east. Piezometer lines covered
and piezometer locations marked.

Bourdon gauges in temporary housing on sand ramp.
Clayey silt fill in background up to approx. El. 941 ft.

Earthmoving equipment placing clayey silt.
Borrow pit in the background.

View of compaction operations looking south-west
toward railway. Fill at approx. El. 941 ft.

View of porous piezometer and $\frac{1}{2}$ " I.D. plastic tubing; bentonite balls in basket

View of plastic tubing during the operation of compacting the bentonite seal.

View looking north. Piezometer tubing in trenches leading to sand ramp and gauge housing

View looking west, showing the operation of removing air bubbles in the plastic tubing. Timber waste excavated from top seven feet of foundation-background

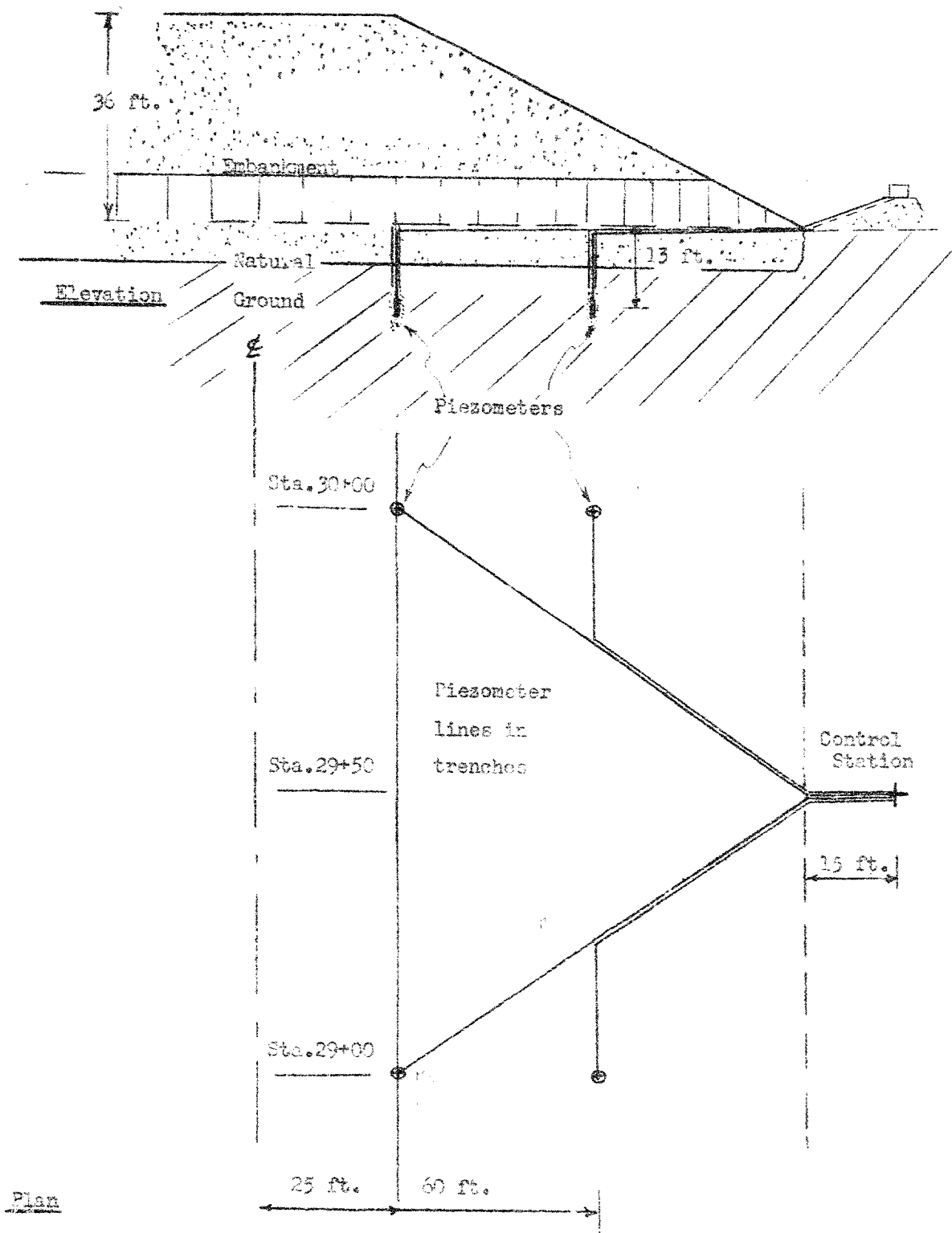
CONTRACT 57-146.

PIEZOMETERS - HUNTSVILLE BY-PASS

Sta.30/00 60' rt. #1.	Sta.30/00 25' rt. #2.	Sta.29/00. 60' rt. #3.	Sta.29/00. 25' rt. #4.	Height above 933.0
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October 1957.

3. 8 a.m.	-	-	-	-
5 p.m.	-	-	-	-
4. 8 a.m.	-	-	-	0.5
5 p.m.	-	1.0	-	2.5
5. 8 a.m.	-	1.0	-	2.5
12.01 p.m.	-	1.5	- 0.5	3.5
7. 8 a.m.	-	1.0	-	3.5
6 p.m.	-	1.25	- 0.75	5.0
8. 7 a.m.	-	1.0	- 0.55	5.0
6 p.m.	-	2.0	- 1.5	6.0
9. 7 a.m.	-	1.8	- 1.25	6.0
6 p.m.	-	1.75	- 1.0	6.5
10. 7 a.m.	-	1.5	- 0.8	6.5
6 p.m.	-	1.4	- 0.65	6.5
11. 7 a.m.	-	1.25	- 0.55	6.5



Sketch showing location of piezometers, trenches and control station
Huntsville By-Pass

PROJECT NO.

C 108/5109

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **Piezometer Study**
Huntsville By Pass
 LOCATION **Sta.29+00 25 ft. Right**
 HOLE LOCATION
 HOLE ELEVATION AND DATUM **932.6**

BORE HOLE NO. **1**
 FIELD SUPERVISOR **WT**
 DRILLER
 PREP.

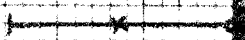



DRAWING NO.

3

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				200	400	600	P.S.F. 800 BLOWS/FT.
	Top of Sand backfill	932.6	0				
	Med. sand backfill	931.5					
		926.0					
	Medium stiff to soft grey varved clayey silt (Clay and silt layers approx. 1/16 in.thick) becomes more plastic around 15 ft.						
	NOTES: All samples pushed easily by hand.						

CONSISTENCY				SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.					
25	35	45	55		
				1	105
					lost
				3	Distur
				4	bed 110.5

PROJ. NO.

C108/J107

DRAWING NO.

4

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

Piezometer Study

PROJECT: Huntsville By-Pass

LOCATION: Sta. 29+00 60 ft. right

BORE ELEVATION

BORE ELEVATION AND DATUM 932.8

BOREHOLE NO. 2

FIELD SUPERVISOR

DRILLER

PREP.

LEGEND

2" DIA. SPLIT TUBE

3" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1.2 UNCONFINED COMPRESSION (Qu)

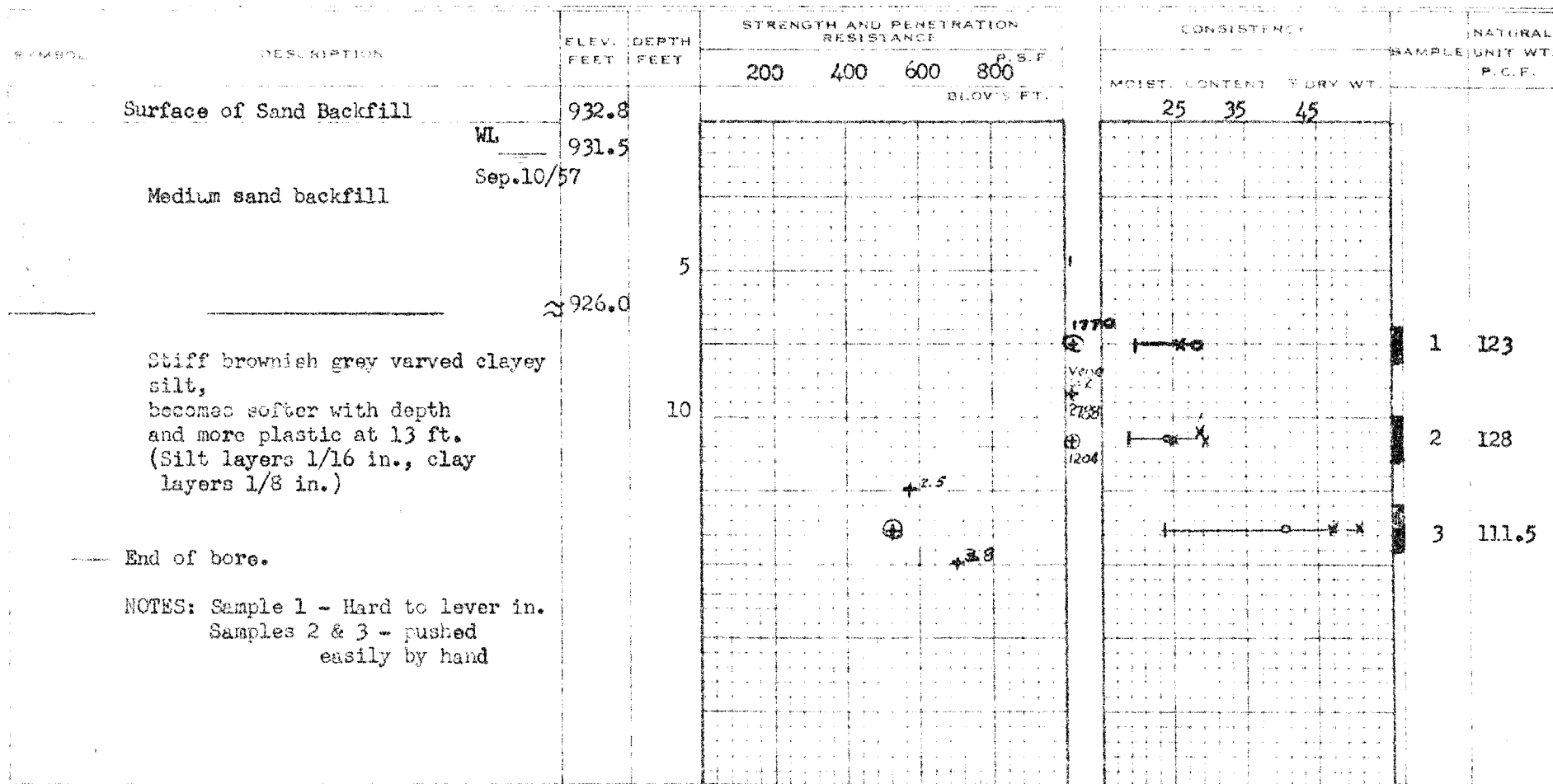
VANE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT



PROJECT NO.

C 108/5109

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

Piezometer Study

PROJECT Huntville By-Pass

LOCATION Sta. 30+00: 60 ft. right

HOLE LOCATION

HOLE ELEVATION AND DATUM 932.4

BOREHOLE NO. 3

FIELD SUPERVISOR

DRILLER

PREP.

DRAWING NO.

5

LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1/2 UNCONFINED COMPRESSION (Qu)

VANE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE				S.F.
				200	400	600	800	
		932.4	0					
	W.L. —	931.5						
	Medium sand backfill							
		≈ 926.0	5					
	Soft grey slightly clayey varved silt. (Alternate thin layers fine silt and coarse silt to fine sand)		10					
	1/4 in. layer fine sand at El. 921 ft.							
	End of bore.		15					

NOTES: 1) All samples pushed in
easily by hand.
2) Samples tended to slump.
Could not do triaxial tests.

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.				
25	35	45		
			1	
			2	130
			3	

PROJECT NO.

C 108/3109

DRAWING NO.

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

296 DEET

Piezometer Study

Huntsville By-Pass

Sta. 30+00 25 ft. right

ATION

VATION AND DATUM 932.1

BOREHOLE NO. 4

FIELD SUPERVISOR

DRILLER.....

PREP.

LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2¹¹ DIA. CONE

CASING

2¹ SHELBY

1/2 UNCONFINED COMPRESSION (QU)

VANE TEST (C) AND SENSITIVITY (S)


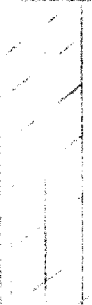
NATURAL MOISTURE AND

LIQUIDITY INDEX

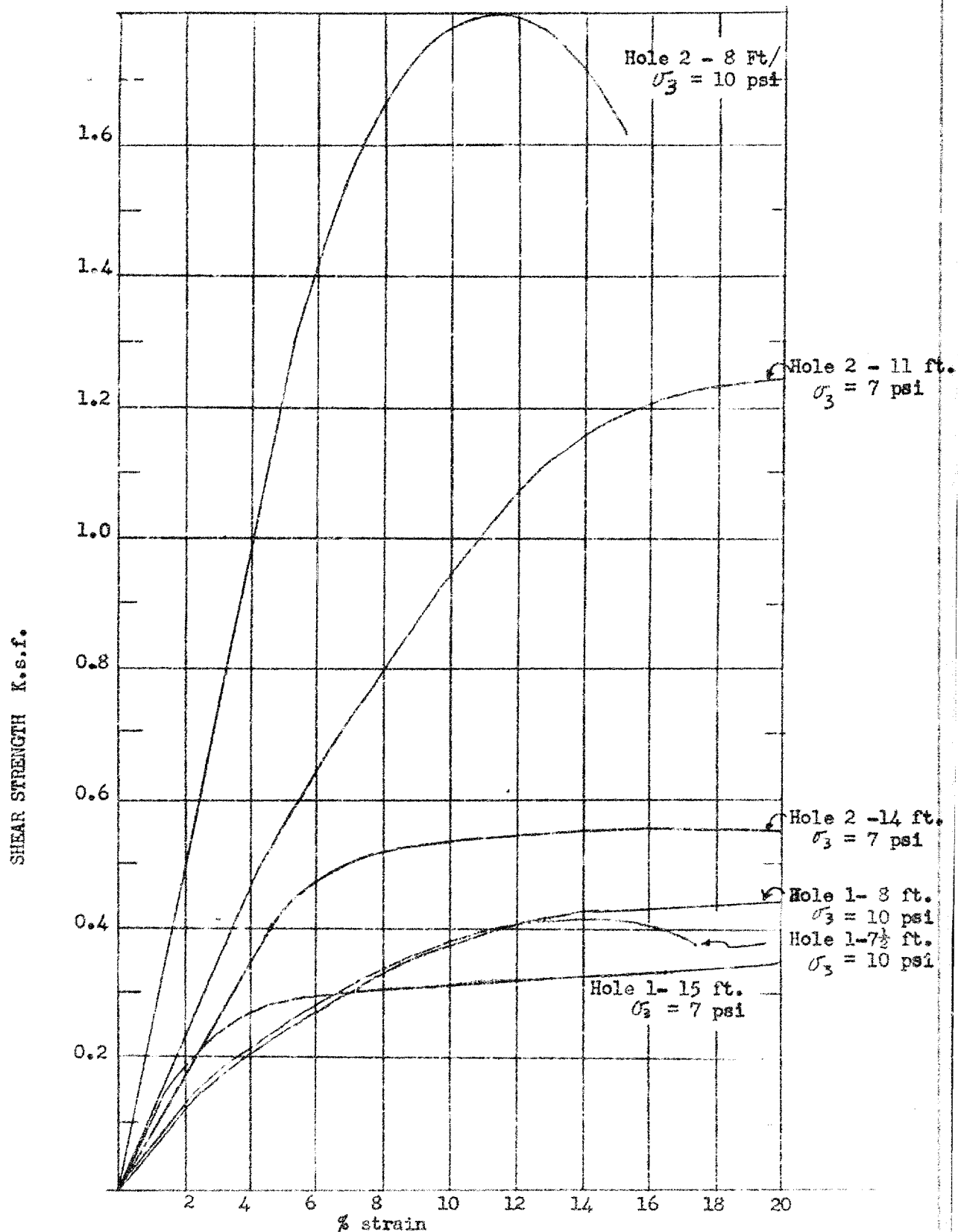
LIQUID LIMIT

PLASTIC LIMIT

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SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				200	400	600	800 P.S.F. BLOWS/FT.
	Surface of sand backfill	932.1	0				
	W.L. 	931.5					
	Medium sand backfill		5				
		925.0					
	Soft grey clayey silt		10				
	End of bore		15				
NOTES: No samples taken							

[illegible]

RESULTS OF QUICK UNDRAINED TRIAXIAL
TESTS ON SAMPLES FROM HOLES 1 AND 2

Pressure in K.s.f.

.02 .05 .1 .2 .5 1 2 5 10 20

1.2

1.1

1.0

0.9

0.8

0.7

Coeff. of Consolidation
sq.ft./day

1.0

0.5

0.0

Rebound Curve

Preconsolidation
pressure = 1500 psf
Overburden
pressure = 910 psf

Compression Index
 $C_c = 0.412$

Coeff. of
Consolidation C_v

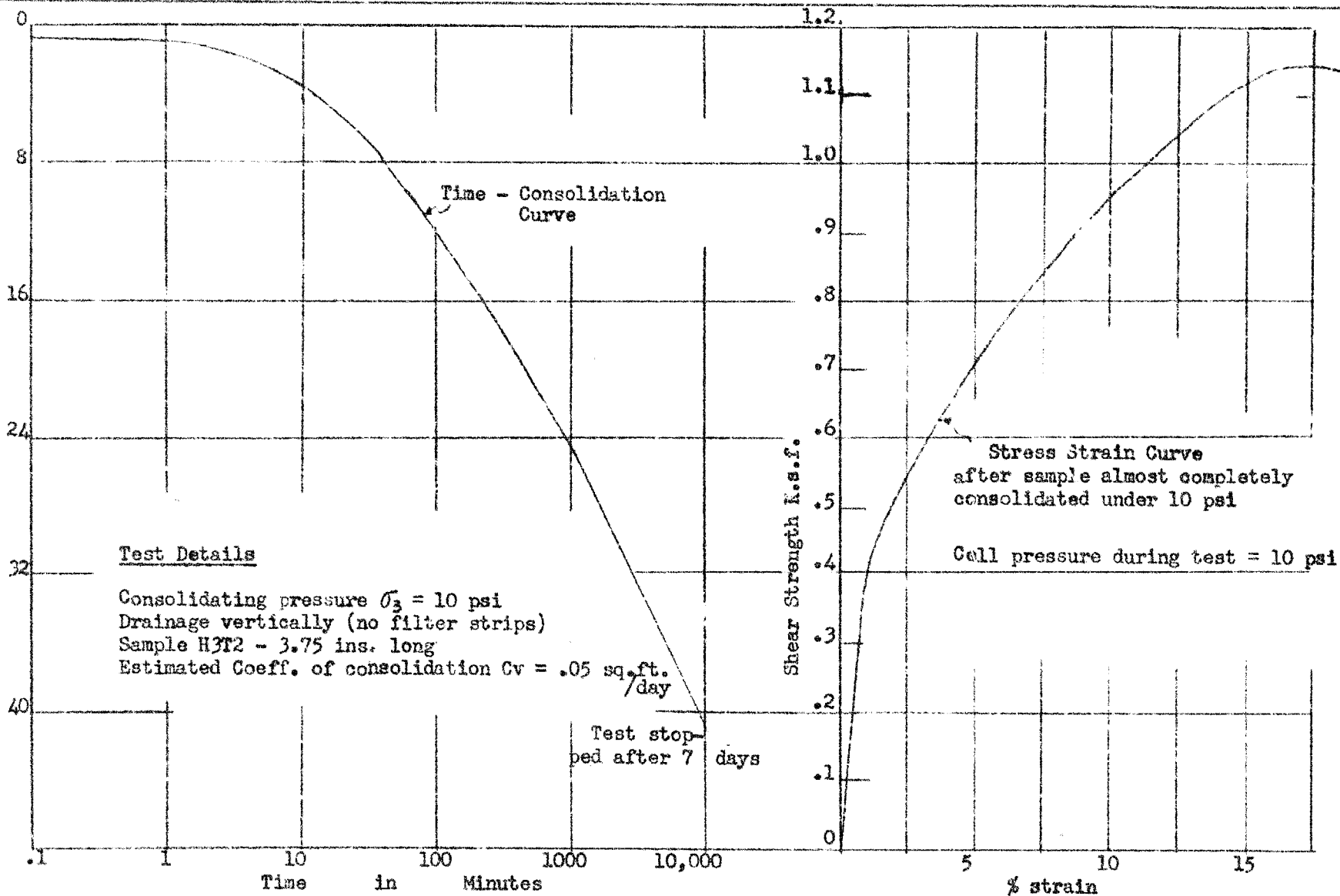
Pressure Void Ratio Curve
for sample from 14 ft.

Hole 2.

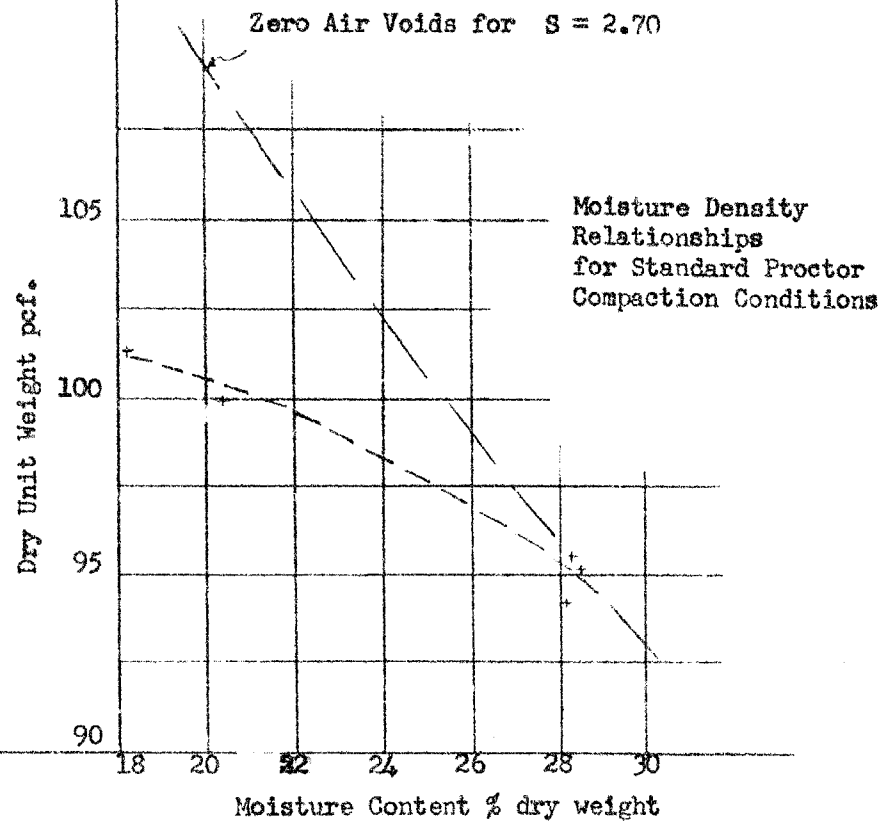
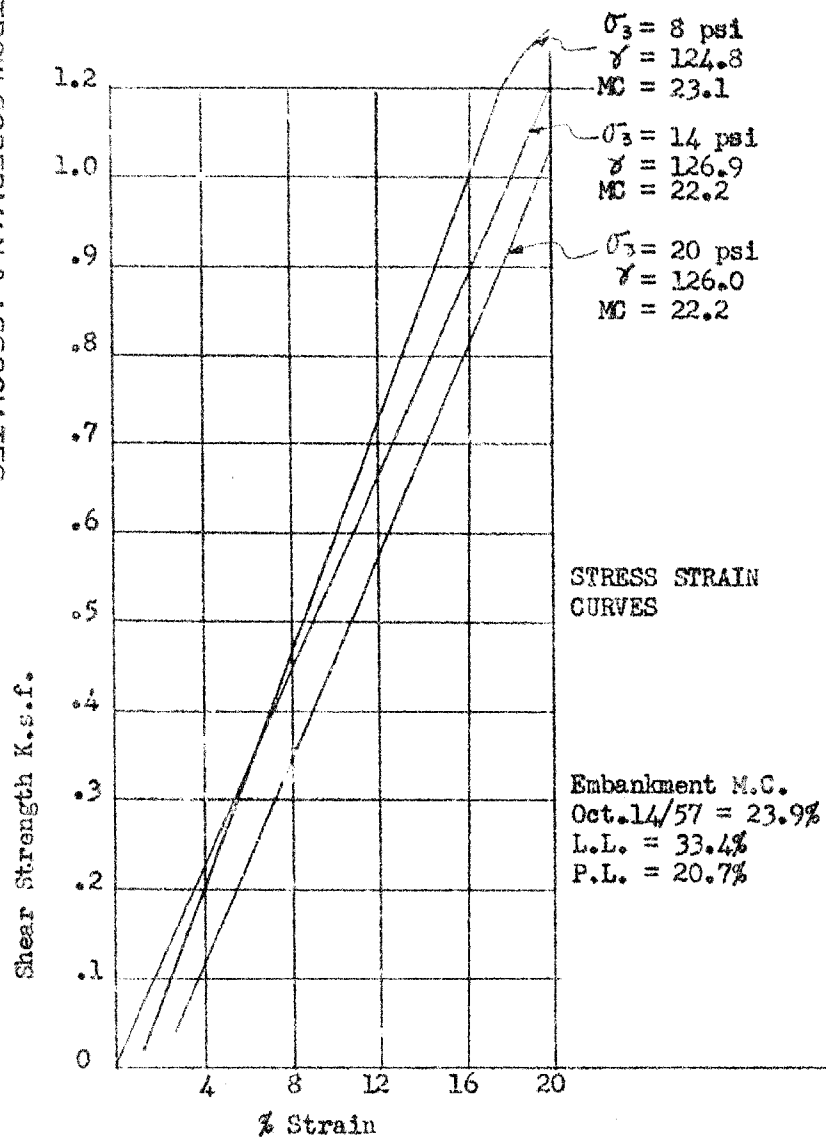
L.L. = 40.4%

P.L. = 23.1%

Wn 48%



CONSOLIDATED QUICK UNDRAINED TEST PERFORMED ON A SAMPLE OF VARVED SLIGHTLY
COHESIVE SILT FROM HOLE 3



RESULTS OF QUICK UNDRAINED TRIAXIAL TESTS ON CLAYEY SILT FILL
RUN AT APPROXIMATE EMBANKMENT PLACEMENT CONDITIONS

Deviator Stress p.s.i.

H2 11 ft. $\sigma_3 = 25$ psi
 L.L. $\approx 26\%$
 P.L. $\approx 18.9\%$
 Wn $\approx 30.3\%$

H2 11 ft.
 $\sigma_3 = 15$ psi

H1 15 ft.
 $\sigma_3 = 25$ psi
 L.L. $\approx 38.0\%$
 P.L. $\approx 22.4\%$
 W.N. $\approx 52\%$

H1 16 ft.
 $\sigma_3 = 15$ psi

H2 14 ft.
 $\sigma_3 = 15$ psi
 L.L. $\approx 40.3\%$
 P.L. $\approx 23.6\%$
 Wn $\approx 50.6\%$

LEGEND:

σ_3 = consol.
 pressure
 L.L. - Liquid Lt. 20
 P.L. - Plastic "
 Wn - Natural M.C.

Stress Strain
Curves

% Strain

Units of water drained

Shearing Resis. psi.

Consolidation V_s Time
Prior to slow shear

Time in Minutes

Mohr Diagram

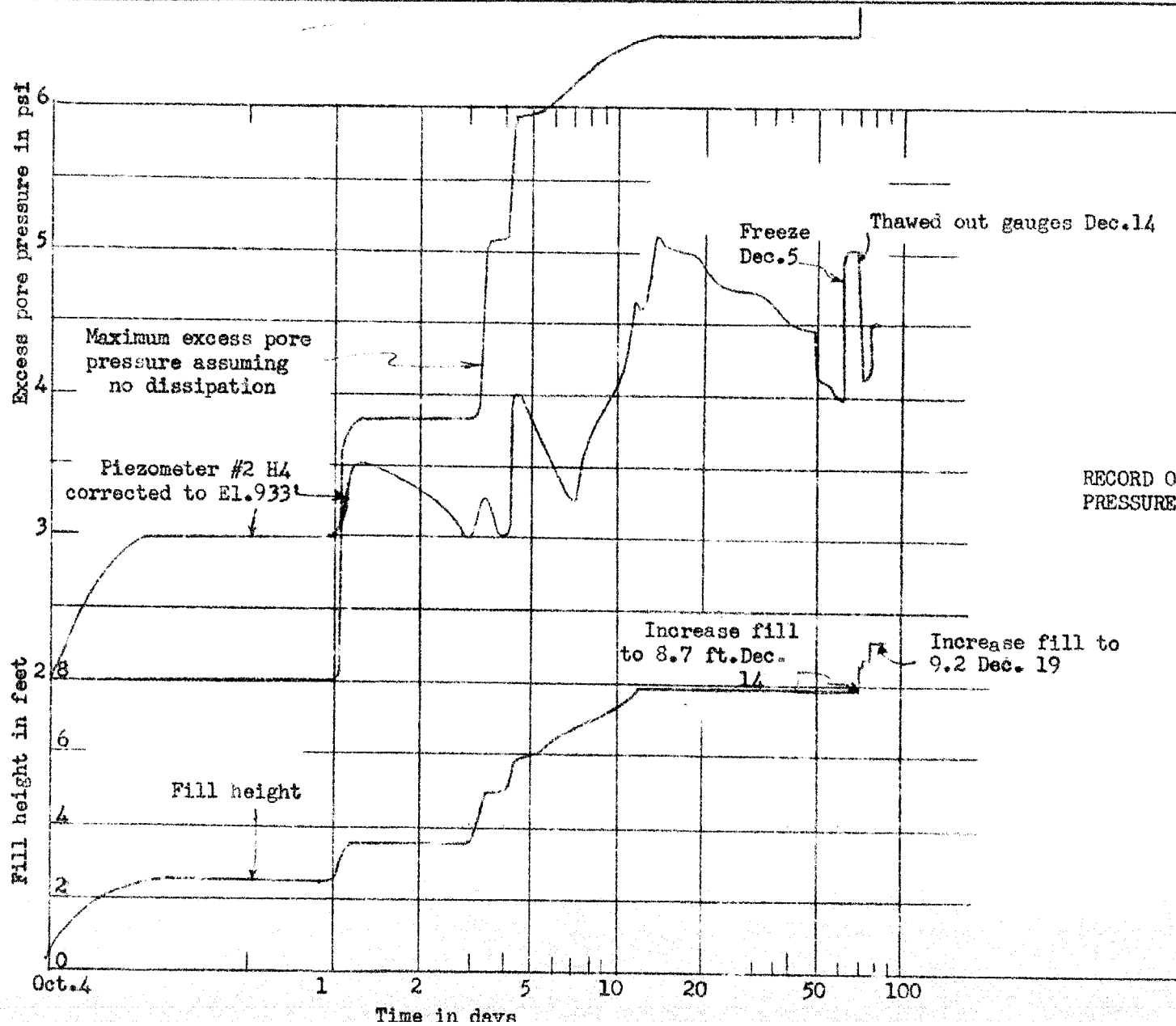
slightly varved
clayey silt
H2 11 ft.Varved silty
clay
H1 15 ft. & H2 14 ft.

Principal Stresses psi

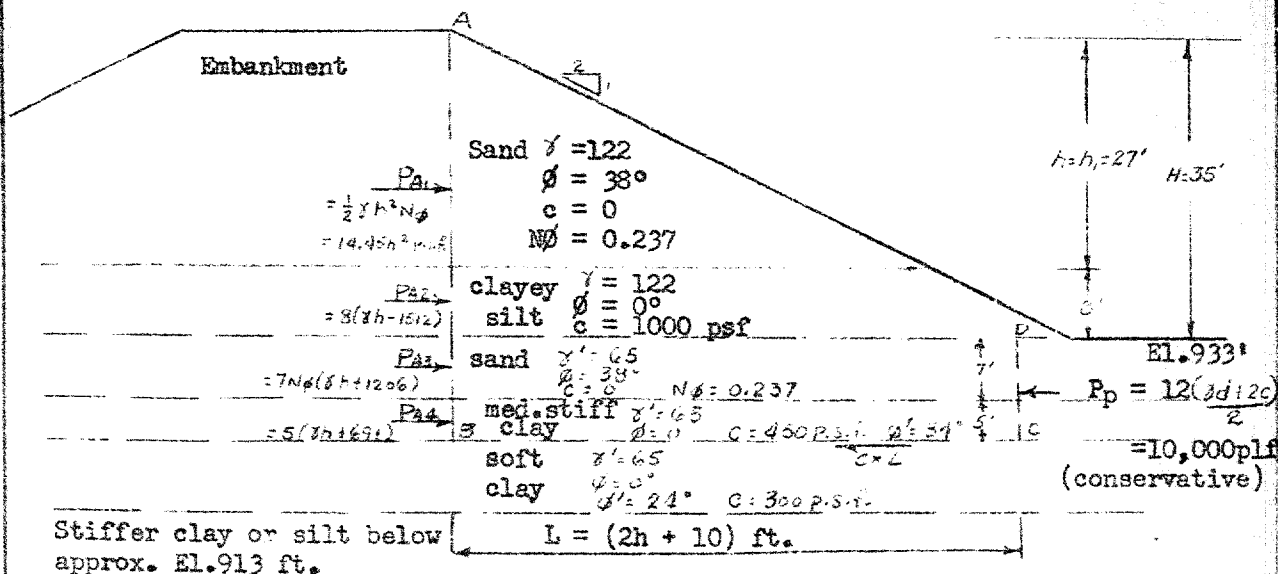
RESULTS OF SLOW DRAINED SHEAR TESTS (Rate of Strain = 0.000289 ins./min.)

PROJECT NO. C108/5109

DRAWING NO. 11



RECORD OF EXCESS PORE
PRESSURES FOR PIEZOMETER #2
Sta. 30+00
25 ft. right



TOTAL STRESS ANALYSIS

$$F.S. = \frac{cL + P_p}{P_A}$$

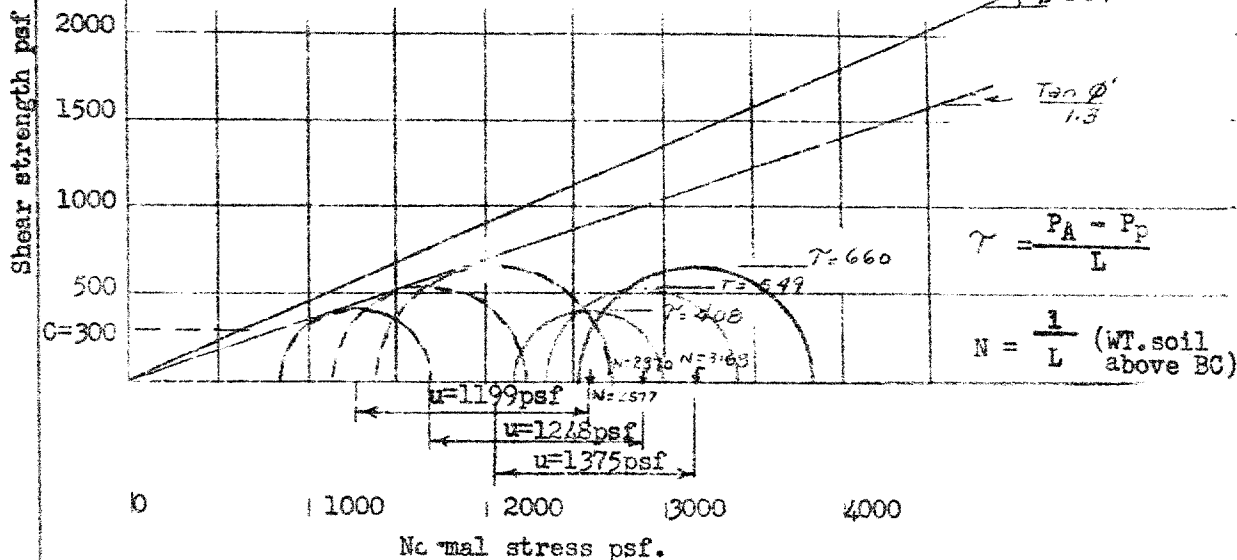
For $H = 35'$
 $H = 30'$
 $H = 25'$
 $H = 20'$
 $H = 15'$

BC at 921'
 F.S. = 0.560
 F.S. = 0.661
 F.S. = 0.831
 F.S. = 1.195

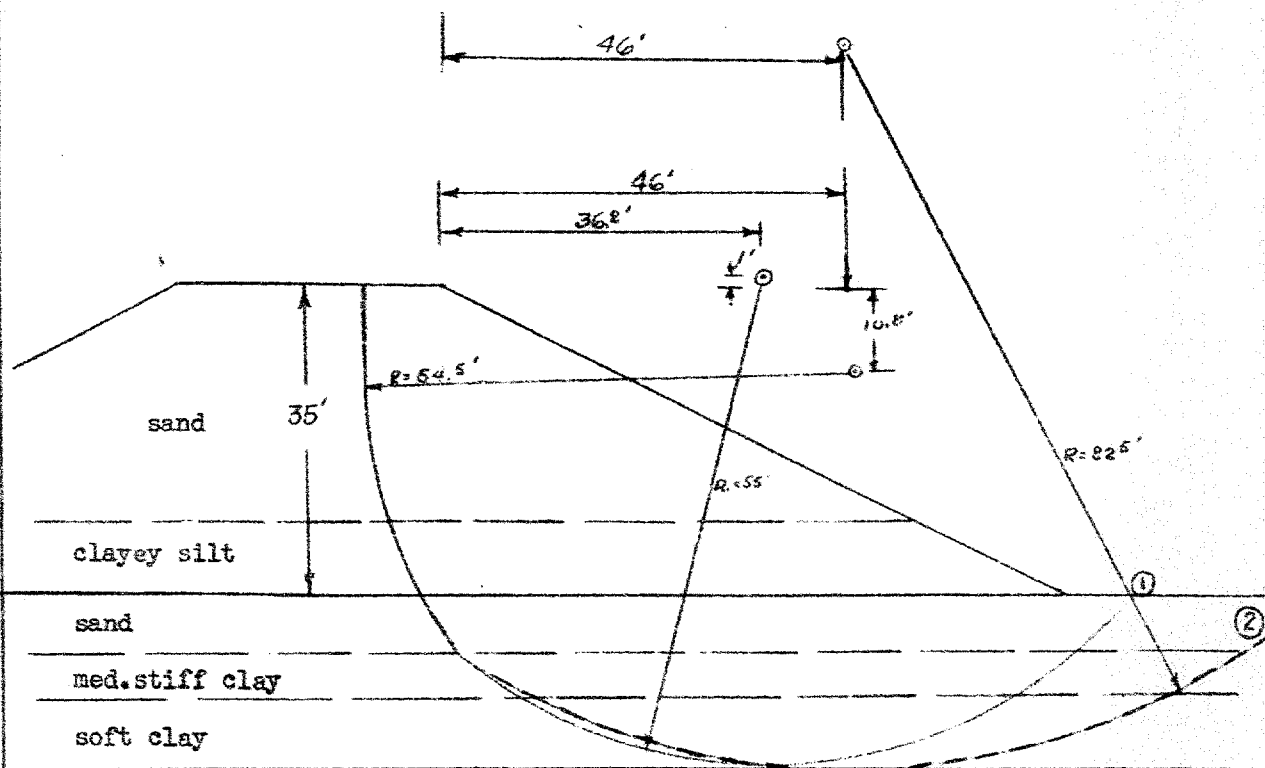
BC at 916 ft.
 = 0.491
 = 0.569
 = 0.69
 = 0.938
 = 1.45

Permissible excess pore pressure u

	El. 921 ft.	El. 916 ft.
$H = 35$ ft.	$u = 8.33$ psi	$= 6.04$
$H = 30$ ft.	$u = 8.67$ psi	$= 6.04$
$H = 25$ ft.	$u = 9.55$ psi	$= 7.65$
$H = 20$ ft.	$u = \infty$	$= 10.2$
$H = 15$ ft.	$u = \infty$	$= \infty$



SKETCHES SHOWING SLIDING BLOCK ANALYSIS AND METHOD OF COMPUTING PERMISSIBLE EXCESS PORE PRESSURE (ASSUME SLIDING ALONG EL. 921 FEET).



Total Stress Analysis

Composite Arc	1	F.S. = 0.644
" "	2	F.S. = 0.765

SKETCH SHOWING DIMENSIONS OF CRITICAL CIRCLES IN COMPOSITE
ARC ANALYSIS

DEPARTMENT OF HIGHWAYS ONTARIO

WORK ORDER

TO **Trow, Soderman and Associates**
884 Wilson Avenue, Downsview, Ontario.

DISTRICT 75 **Materials Res.**

TYPE OF WORK **Embankment Stability Analysis**

CONTRACT No.

HWY. LOCATION **Materials Research Section**

PROJECT No. **8-31458**

DATE **Sept. 25/58**

The Department acknowledges receipt of your letter Sept. 4/58
 and accepts your estimate of \$164.00 for the following work on
 Contract 57-146.

Field Testing of Monometers and
 Additional Stability Analyses,
 Huntsville By-Pass

Copy to: Internal Audit

Copy to: Materials Research Engineer

JFMCG/prc

CAPITAL

OFFICIAL WORK ORDER No.

8-32620

NOT VALID UNLESS NUMBERED HERE

WORK APPROVALS	TENDER	MATERIAL	ENGINEERING	SUNDRY EXP.	TOTAL
APPROVALS THIS WORK ORDER					
PRIOR APPROVALS					
TOTAL TO DATE					

MONEY ALLOTMENTS	TENDER	MATERIAL	ENGINEERING	SUNDRY EXP.	TOTAL
ALLOTMENTS THIS WORK ORDER					
TOTAL PRIOR WORK ORDERS THIS YEAR					
TOTAL PRESENT FISCAL YEAR					
TOTAL EXPENDITURES PRIOR YEARS					
TOTAL COMMITMENT TO DATE					

FUNDS ALLOTTED

FINANCIAL COMPTROLLER

AUTHORIZED

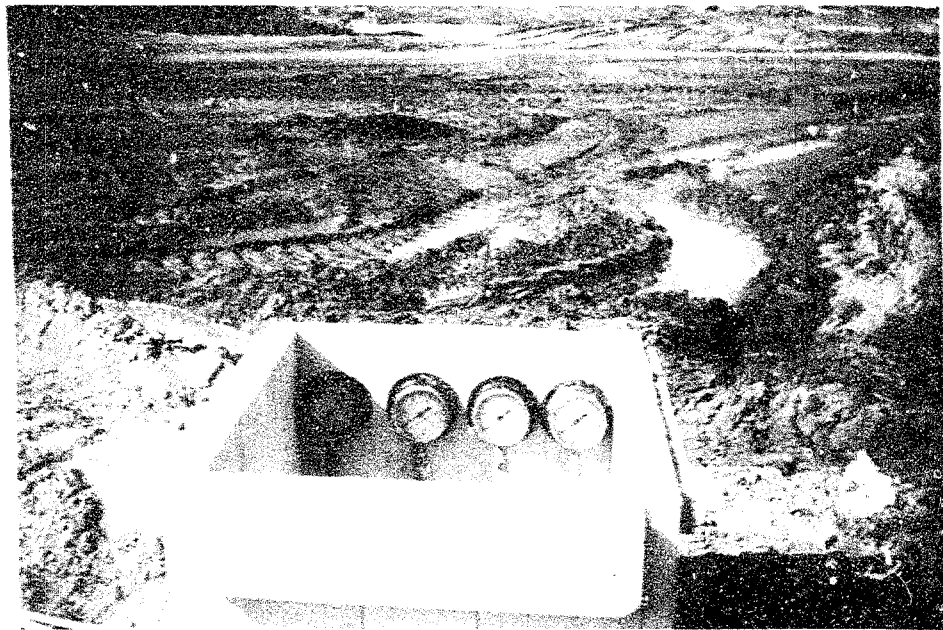
D. Beaudro
 CONTRACT CONTROL ENGINEER

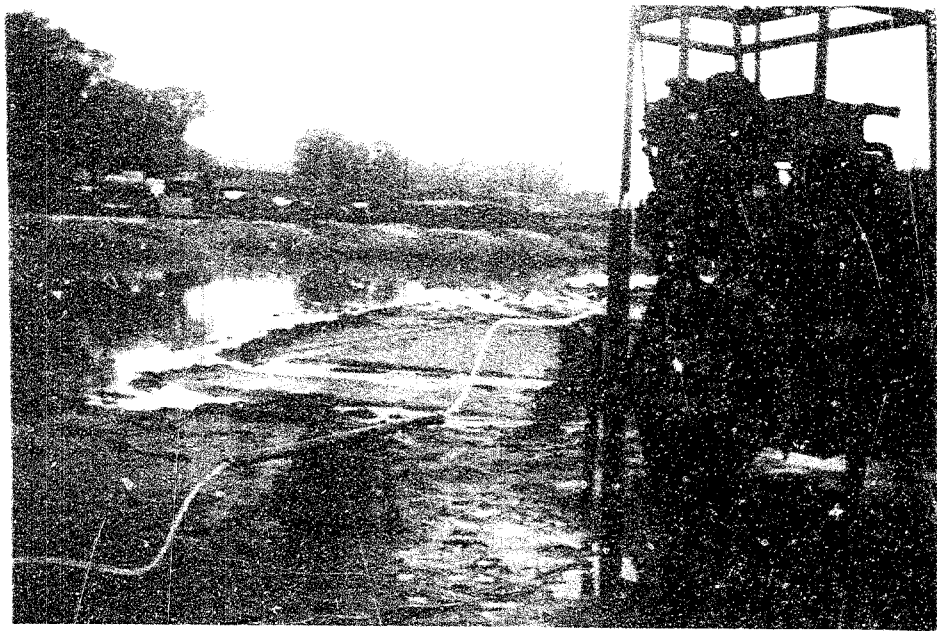




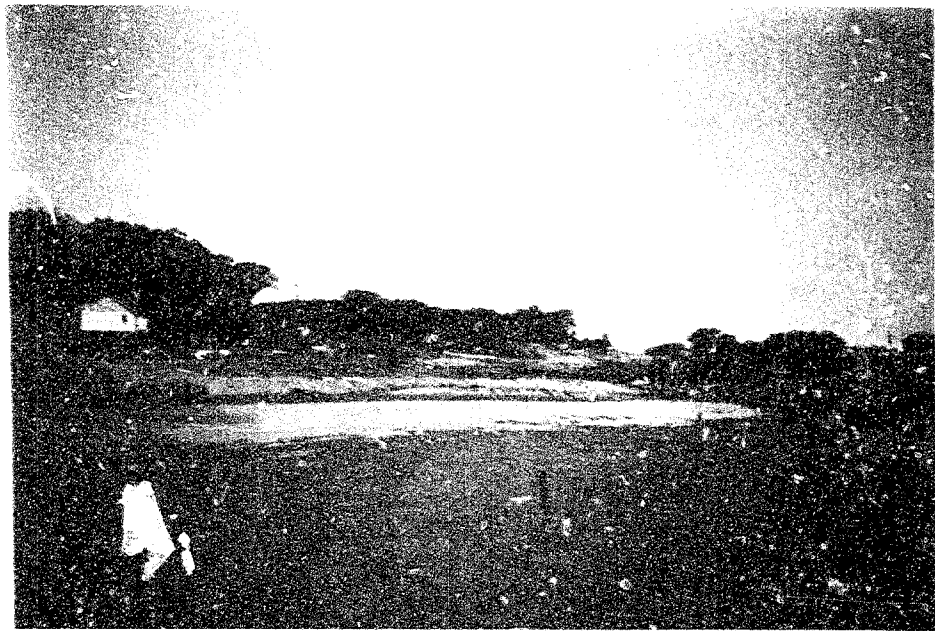


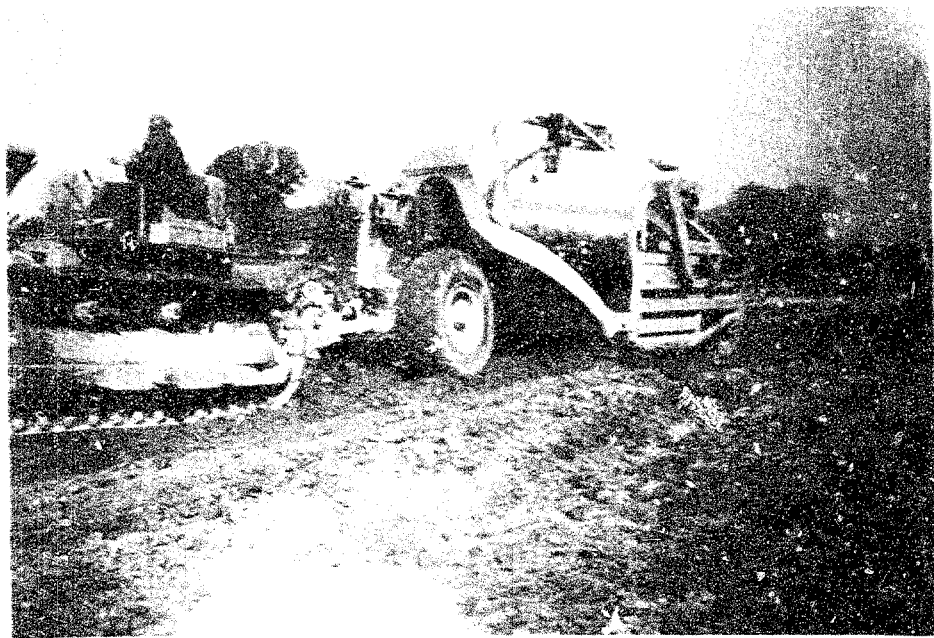


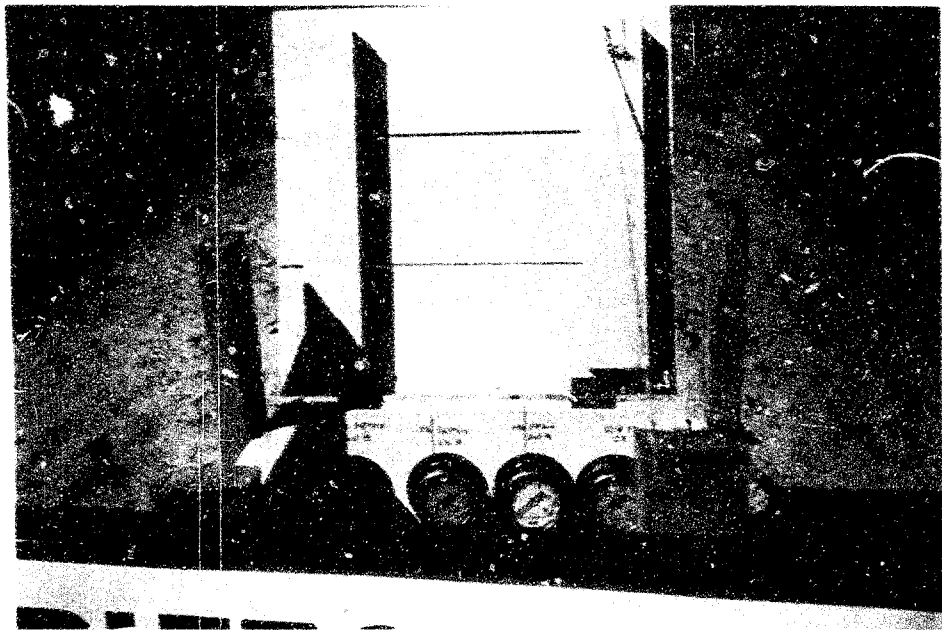


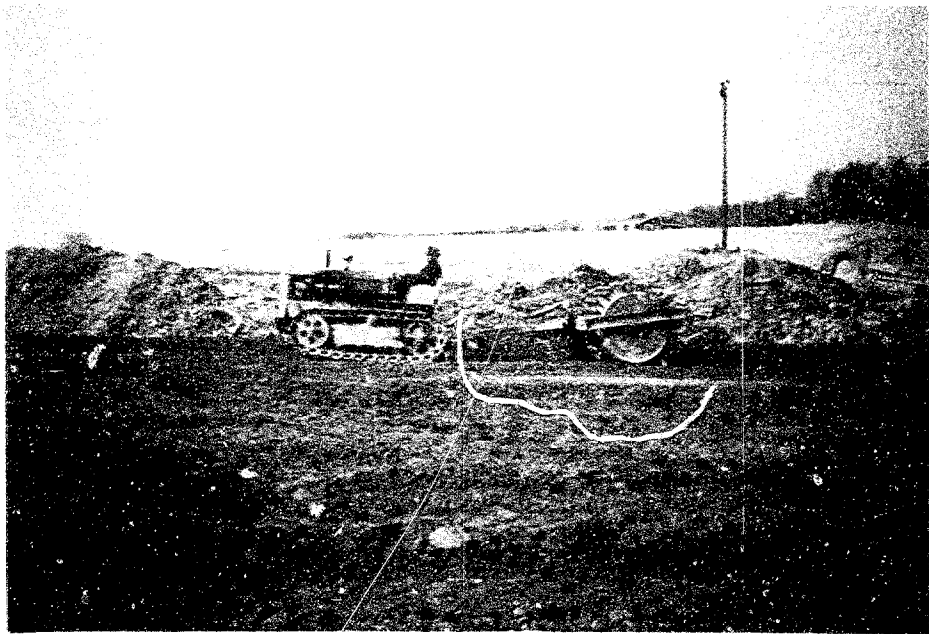






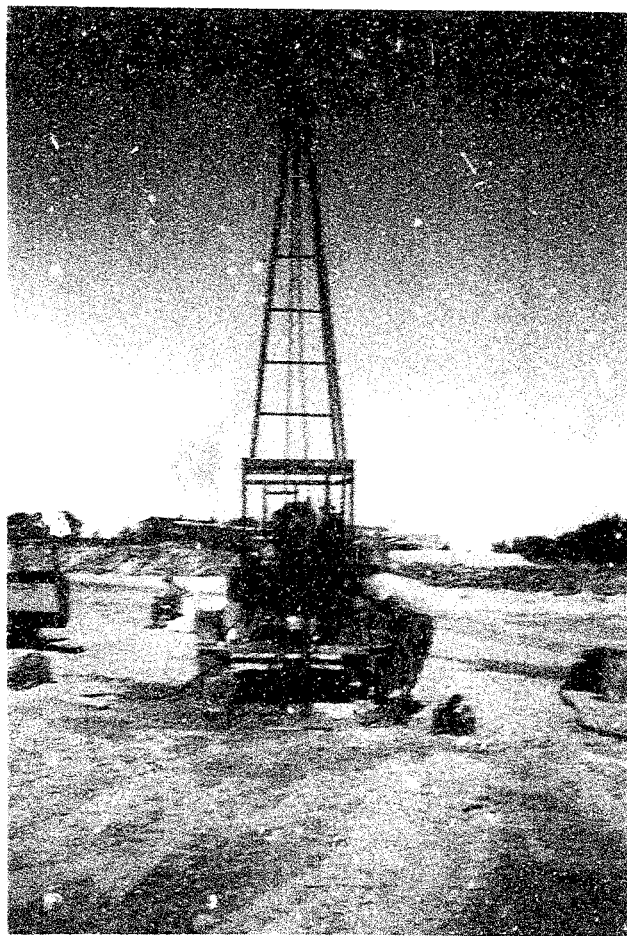






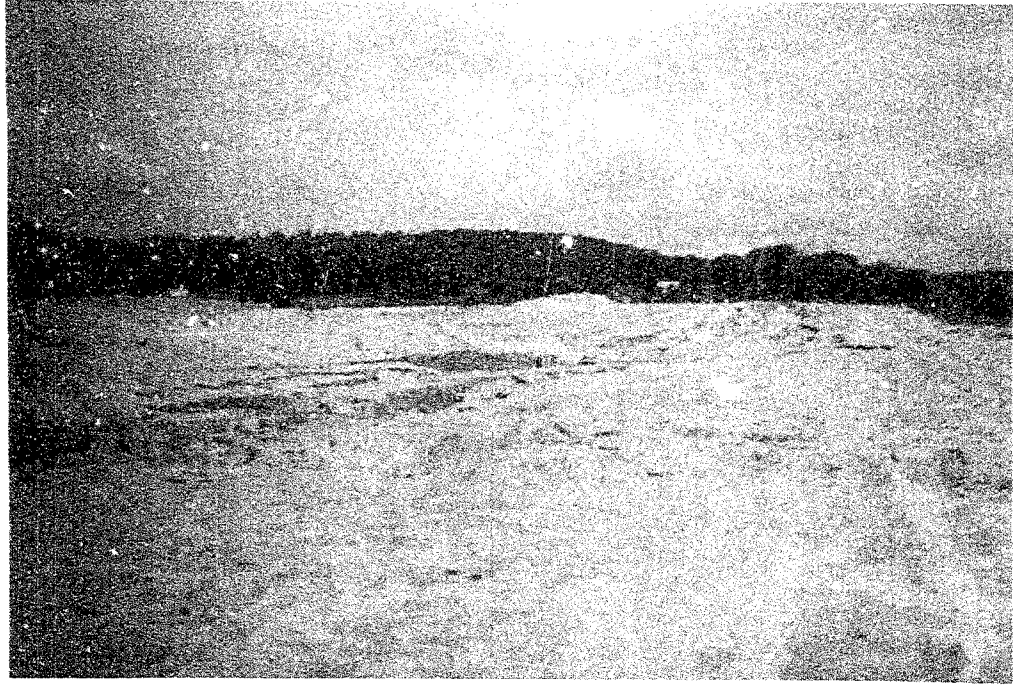


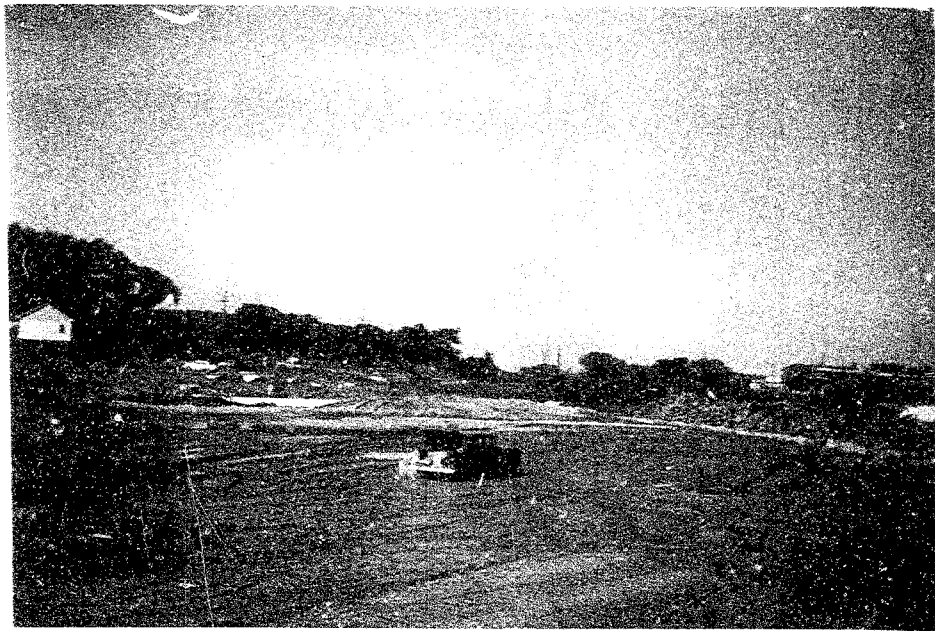




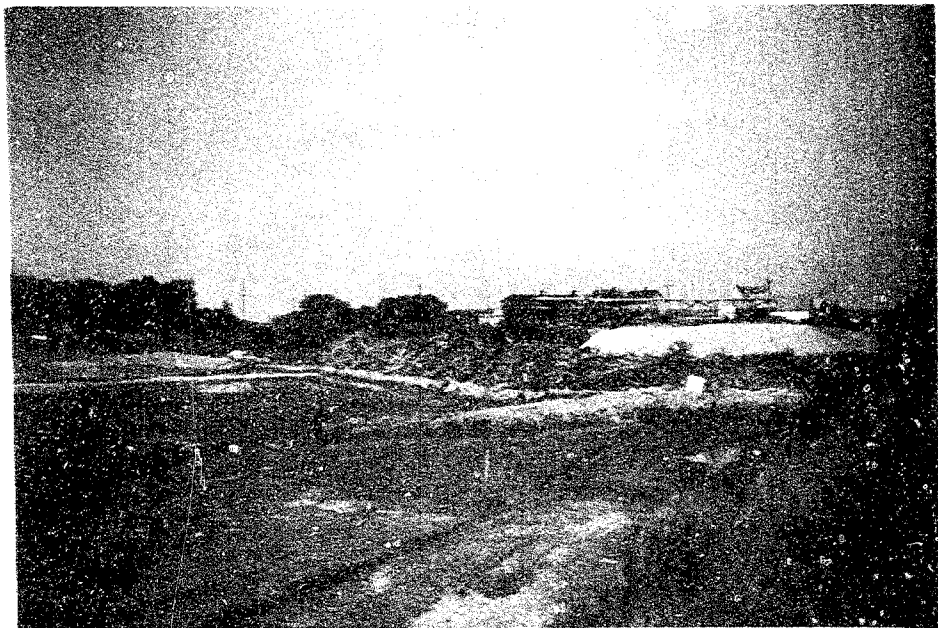


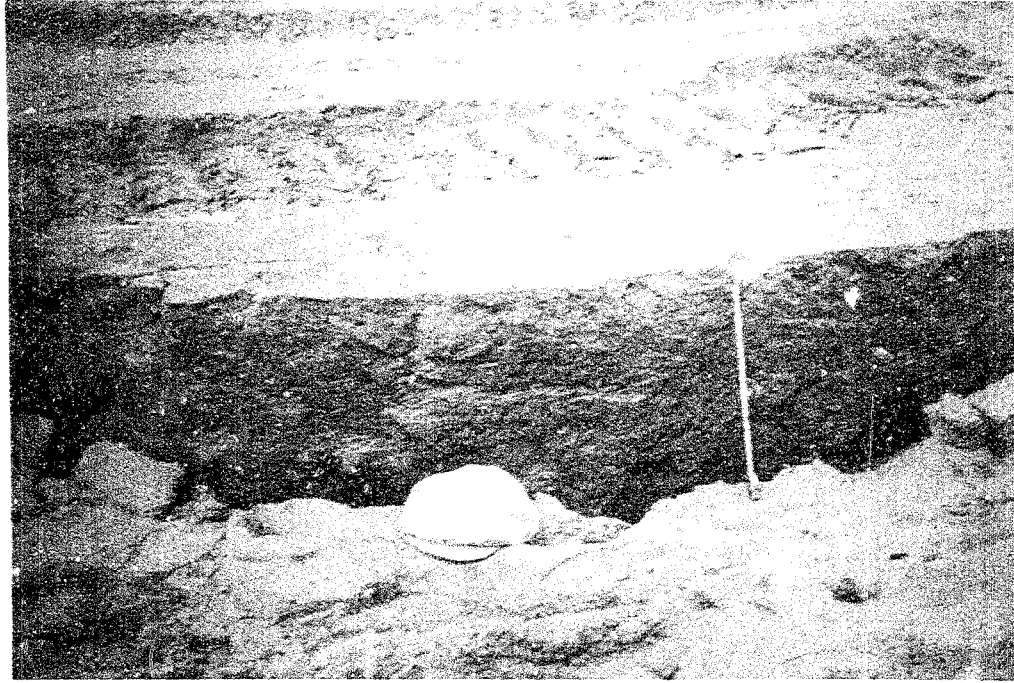
SEP 58

















SEP 58



SEP 58



PIEZOMETER INSTALLATION

C.N.R. OVERPASS - STA. 29+00 HUNTSVILLE BY-PASS

NOTE:

THE GROUND WATER ELEV.

931.5

GAUGE HOUSE ELEV.

937.5

FROM THE ABOVE INFORMATION IT APPEARS THAT THE GAUGES WERE AT LEAST SIX FEET ABOVE GROUND WATER ELEVATION THUS BEING SUBJECT TO A NEGATIVE ~~WATER~~ PRESSURE OF 2.6 PSI.

IN THE PAPER "EXPERIENCE WITH CANADIAN VARIED CLAYS" BY MILLIGAN, SODERMAN & RUTKA EXCESS PORE WATER PRESSURES ARE GIVEN. IT IS BELIEVED THAT THESE ARE VALUES CORRECTED FOR THE INITIAL NEGATIVE PRESSURE HOWEVER, A NEGATIVE PRESSURE OF ONLY 5 PSI WAS TAKEN AND THEREFORE OUR RESULTS DIFFER FROM THE ONES IN THE PAPER FOR APPROX 2 PSI. OUR RESULTS ARE ABSOLUTE ^{GAUGE} READINGS - NOT EXCESS PORE PRESSURES

APRIL 28, 1964

AGS/TERMAC

HUNTSVILLE EMBANKMENT

SHEET 1 OF 7

TIME IN DAYS		FILL HEIGHT IN FEET	P ₁	P ₃	P ₂	P ₄
NOTE: FOR CORRECTED TIME IN DAYS	0	0	0	0	0	0
	1	6.8	0		1.8	1.1
	2	7.0	0		2.1	1.2
	4	7.2	0.3		2.3	0.7
	5	7.5	0	0	2.65	1.3
	7	7.8	0.85	0	3.20	1.70
	8	7.8	0.75	0	3.10	1.60
	9	7.8	0.75	0	3.0	1.5
	9	7.8	0.70	0	3.0	1.4
	11	7.8	0.5	0	3.0	1.25
	12	7.8	0.5	0	3.0	1.25
	13	7.8	0.45	0	2.95	1.20
	14	7.8	0.40	0	2.9	1.2
	15		0.1	0	2.85	1.1
ADD 3 DAYS TO FIG. QUOTED IN THIS COLUMN.	16		0	0	2.8	1.0
	18		0	0	2.75	0.9
	19		0	0	2.75	0.9
	20		0	0	2.75	0.9
	21		0	0	2.75	0.9
	22		0	0	2.75	0.9
	23		0	0	2.75	0.85
	25		0	0	2.70	0.5
	33		0	0	2.55	0.5
	34		0	0	2.55	0.5
	35		0	0	2.5	0.5
	36		0	0	2.45	0.5
	37		0	0	2.45	0.5
	39		0	0	2.45	0.5
NOTE: FOR CORRECTED TIME IN DAYS	41		0	0	2.45	0.5
	42		0	0	2.45	0.5
	43		0	0	2.35	0.5
	44		0	0	2.2	0.5
	46		0	0	2.1	0.5
	47		0	0	2.1	0.5
	48		0	0	2.1	0.5
	50		0	0	2.0	0.5
	51		0	0	2.0	0.5
	53	7.8	0	0	2.0	0.5

HUNTSVILLE EMBANKMENT

SHT 2 OF 7

TIME IN DAYS	FILL HEIGHT IN FEET	P ₁	P ₃	P ₂	P ₄
54	7.8	0	0	1.95	0.4
55		0	0	1.9	0.3
56		0	0	2.25	0.2
57		0	0	3.0	0.55
58		0	0	2.95	0.45
61		0	0	3.0	0.5
62		0	0	3.0	0.3
63		0	0	2.9	0
64	7.8	0	0	2.7	0
65	8.7	0	0	2.1	0.6
69	8.7	0	0	2.1	0.5
70	9.2	0	0	2.5	0.5
71		0	0	2.5	0.5
72		0	0	2.5	0.5
75		0	0	2.5	0.5
81		0	0	2.5	0.5
82	9.2	0	0	2.5	0.5
85	10.1	0	0	2.5	0.5
86		0	0	2.5	0.5
88		0	0	2.5	0.5
89		0	0	2.5	0.5
90		0	0	2.5	0.5
91		0	0	2.5	0.5
92		0	0	2.5	0.5
93	10.1	0	0	2.5	0.5
95	10.9	0	0	2.5	0.5
96	11.7	0	0	2.5	0.5
97		0	0	3.4	1.0
98		0	0	3.4	1.0
100		0	0	3.4	1.0
102		0	0	3.4	1.0
103		0	0	3.4	1.0
104		0	0	3.35	1.0
106		0	0	3.35	1.0
107		0	0	3.35	1.0
108		0	0	3.35	1.0
110		0	0	3.35	1.0
111	11.7	0	0	3.35	1.0

HUNTSVILLE EMBANKMENT

SHT 3 OF 7

TIME IN DAYS	FILL HEIGHT IN FEET	P ₁	P ₃	P ₂	P ₄
112	11.7	0	0	3.35	1.0
113	12	0	0	3.35	1.0
114	12	0	0	3.3	1.0
117	12	0	0	3.25	0.8
118	12	0	0	3.25	0.8
119	12	0	0	3.2	0.75
123	12	0	0	3.1	0.6
124	12	0	0	3.1	0.6
126	12	0	0	3.1	0.5
130	12	0	0	3.1	0.5
132	12	0	0	2.75	0.25
137	12	0	0	2.5	0
141	12	0	0	2.25	0
145	12	0	0	2.2	0
148	12	0	0	2.1	0
152	12	0	0	2.05	0
155	12	0	0	2.05	0
159	12	0	0	2.05	0
162	12	0	0	2.0	0
203	14.0	1.0	0	3.3	1.0
205	12	0.75	0	3.1	0.75
206	12	0.9	0	3.4	0.65
207	12	0.9	0	3.3	0.6
208	12	0.85	0	3.3	0.6
209	12	0.8	0	3.25	0.55
212	12	0.7	0	3.2	0.55
213	12	0.7	0	3.2	0.55
214	12	0.6	0	3.2	0.5
215	12	0.5	0	3.2	0.55
216	12	0.55	0	3.2	0.55
217	12	0.55	0	3.2	0.55
219	12	0.55	0	3.15	0.55
220	12	0.55	0	3.15	0.55
221	12	0.55	0	3.1	0.55
222	12	0.55	0	3.1	0.55
223	12	0.55	0	3.05	0.55
224	12	0.55	0	3.0	0.55
226	14.0	0.55	0	3.0	0.55

MAR. 21-58

MAY 31-58

JUNE 2

TIME	DEPTH	P ₁	P ₂	P ₃	P ₄	
227	35 14.0	0.55	0	2.9	0.53	
228	36	0.53	0	2.8	0.53	
229	37	0.5	0	2.6	0.52	
230	38	0.5	1	2.6	0.51	JUNE 27
235	43	0.5	0	2.5	0.51	JULY 2
236	44 14.0	0.5	0	2.5	0.51	
237	45 12.5	0	0	1.0	0	
240	48 11.0	0	0	0	0	
241	49	0	0	0	0	
247	54	0	0	0	0	
251	57	0	0	0	0	
254	62	0	0	0	0	NO RECORD ON FILM
255	63 11.0	0	0	0	0	
256	64 12.0	0	0	1.0	0.5	
257	65 12.5	0	0	1.25	0.6	
258	66 14.06	0.5	0	3.25	2.1	JULY 25
259	67 14.05	0.5	0	3.25	2.05	
261	69 16	0.5	0	3.75	2.2	
262	70 17	1.8	0	4.2	2.5	
262	71 17.5	1.0	0	4.55	3.25	
263	72	1.02	0	4.55	3.1	
264	73	1.02	0	4.55	3.1	JULY 31
265	74	1.02	0	4.55	3.0	
266	75	1.02	0	4.55	3.0	
268	76	1.02	0	4.55	3.0	
269	77	1.02	0	4.55	2.95	
270	78	1.02	0	4.55	2.9	
271	79	1.02	0	4.5	2.75	
272	80	1.02	0	4.5	2.65	
273	81	1.02	0	4.5	2.65	
275	83	1.02	0	4.48	2.5	
276	84	1.02	0	4.4	2.5	
277	85	1.02	0	4.35	2.5	
279	87 17.5	1.01	0	4.25	2.48	
282	90 18.0	1.0	0	4.25	2.5	
283	91 18.0	1.01	0	4.48	2.7	
284	92 18.6	1.01	0	4.75	3.35	

MUNTSVILLE EMBANKMENT

SHT. 5 OF 7

TIME	FILL		P1	P3	P2	P4	
IN DAYS	Height Ft.	Elev.					
285	20.0	953.0	3	1.48	0	5.5	3.5
286			4	1.9	0	6.0	3.51
287			5	1.9	0	6.0	3.51
288			7	1.9	0	6.0	3.51
290			7	1.8	0	6.0	3.5
291			2	1.5	0	5.75	3.48
292			20	1.5	0	5.7	3.48
293			1	1.5	0	5.7	3.48
294			2	1.5	0	5.5	3.48
297			5	1.48	0	5.5	3.35
298			6	1.48	0	5.49	3.35
299			7	1.48	0	5.49	3.35
300			2	1.48	0	5.49	3.25
301			2	1.48	0	5.49	3.25
303			1	1.48	0	5.25	3.1
304			2	1.48	0	5.15	3.0
305			2	1.48	0	5.05	3.0
306			10	1.48	0	5.0	3.0
307			5	1.48	0	5.0	3.0
308			6	1.48	0	5.0	3.0
310	20.0	953.0	5	1.0	0	4.5	2.0
311	21.0	954.0	9	1.0	0	5.0	3.0
312			20	1.0	0	5.25	3.0
313			1	1.5	0	5.9	3.0
313				1.52	0	6.0	3.9
314	22.0	955.0	2	1.32	0	6.0	4.5
315			1	1.52	0	6.0	4.5
317			5	1.52	0	6.0	4.5
318			2	1.5	0	5.99	4.1
319			1	1.48	0	5.75	4.1
320			3	1.48	0	5.75	4.2
321			1	1.48	0	5.75	4.1
322			20	1.48	0	5.75	4.1
324	22.0		2	1.48	0	5.6	4.05
324	22.5	955.5	2	1.48	0	5.8	4.1
325	22.5		3	1.48	0	6.0	4.15
325	23.0	956.0	3	1.5	0	6.25	4.15
326		956.0	24	1.5	0	6.45	4.15

OCT. 1

HUNTSVILLE EMBANKMENT

SHT. 6 OF 7

TIME	FILL		P.1	P.3	P.2	P.4
IN DAYS	HUNT R.	Elev.				
327		9560	1.5	0	6.4	4.1
328			1.5	0	6.35	4.1
329			1.5	0	6.25	4.1
331			1.5	0	6.0	4.1
332			1.5	0	6.0	4.05
333			1.5	0	6.0	4.05
334			1.5	0	6.0	4.0
335			1.5	0	5.95	4.05
336			1.5	0	5.55	4.0
340			1.5	0	5.5	4.0
341			1.5	0	5.5	4.05
342			1.5	0	5.5	4.05
343			1.5	0	5.4	4.0
346			1.5	0	5.3	4.0
347			1.5	0	5.1	4.0
348	23.0	9560	6	1.65	5.0	4.05
349	24.0	9570	7	1.6	5.4	4.05
350			1	1.45	5.5	4.05
352	24.0	9570	10	1.48	5.7	4.0
353	25.0	9580		1.48	5.65	4.05
354				1.48	5.55	4.05
355				1.48	5.55	4.05
356				1.48	5.5	4.05
358				1.48	5.5	4.4
360				1.48	5.5	4.4
361				1.48	5.48	4.4
362				1.48	5.3	4.3
363				1.48	5.3	4.3
366			370	1.48	5.1	4.3
367			5	1.46	5.1	4.3
368				1.46	5.1	4.3
369				1.46	5.05	4.3
370			378	1.46	5.0	4.3
371	25.0	9580	9	1.44	4.95	4.3
372	25.5	9585	10	1.44	5.0	4.4
373	26.5	9595	1	1.46	5.5	4.3
374	27.0	9600	2	1.48	5.5	4.5
375		9600	3	1.49	5.5	4.52

Nov. 3

LUNTSVILLE EMBANKMENT

SHT 7 OF 7

TIME	FILE	P.1	P.3	P.2	P.4	
IN DAYS	11000 F. ELEV					
376		960.0 374 148	0	5.5	4.5	
377		148	0	5.5	4.5	
378		148	0	5.5	4.5	
379		148	0	5.5	4.5	
380		148	0	5.5	4.5	
381	27.0	960.0 148	0	5.5	4.5	1958
382		960.0 320 148	0	5.10	4.5	Nov. 28

CONTINUED ON NEXT PAGE

DATE	FILL HEIGHT OR ELEV.	P ₁	P ₂	P ₃	P ₄	TIME IN DAYS. (CORRECTED)
DEC. 1-50	27	1.48	5.0	0	4.4	393
2		1.48	5.0		4.2	94
4		1.47	4.95		4.05	96
10		1.25	4.55		5.0	402
11		1.10	4.25		5.0	03
12			4.50		4.25	04
15			4.30		4.5	07
18		1.0	4.30		4.6	11
22		1.2	4.60		4.1	14
26			4.50		3.5	18
30		1.2	4.40		3.6	22
JAN 5-50			4.3		3.2	28
6			4.3		3.2	29
8			4.4		3.3	31
12			4.1		3.8	35
16			4.1		3.8	39
18			3.5		3.2	42
23		1.2	3.6		3.6	46
27		1.1	3.5		3.0	450
FEB 2		1.1	3.4		2.5	56
5		0.7	3.2		2.2	59
10			3.1		2.0	64
13			2.9		1.8	67
17		0.7	2.8			71
20		0.6	2.8		1.8	74
24		0.7	2.6		1.7	78
27		0.7	2.6		1.7	81
MAR 3		0.6	2.4		1.5	85
6		0.5	2.3		1.5	88
9-17			2.1		1.3	91-93
24			2.0		1.5	506
26		0.5			1.4	93
31		0.4			1.5	13
APR 7		0.5			1.3	20
14					1.25	27
17		0.5	2.0	0	1.5	30

DATE	FALL HEIGHT OF FLL	P ₁	P ₂	P ₃	P ₄	TIME IN DAYS. (CORRECTED)
APR 21 ⁵⁰	27	0.5	2.0	0	1.4	5 34
28		0.5	2.0		1.7	41
MAY 1			1.9		1.6	44
8					1.4	51
15		0.5				58
18		0.4	1.9		1.4	62
21			1.8		1.5	64
22			1.8			65
25		0.4	2.1			68
26		0.5	2.35			69
27			2.6			70
28			2.8			71
29			2.9			72
30			3.0		1.5	73
JUNE 1			3.0		1.3	75
2			3.1		1.5	77
4		0.5			1.3	78
5		0.6	3.1		1.2	79
8			3.0		1.3	82
9						83
11						85
12		0.6			1.3	86
15		0.5	3.0		0.8	89
16			2.8		0.8	90
18			2.7		1.2	93
23			2.5		1.0	97
26			2.3		1.0	600
30			2.2		0.8	84
JULY 3 ⁵⁰	27	0.5	2.1	0	0.6	607
APR 12 ⁵¹		0.5	0.5	1.5	2.5	1935

Datum elevations of gages at CNR overhead and Huntsville By-pass

Apr. 23/6

0.09 944.84 944.75 BM

7.27 942.29 982 935.02 TP

4.60 941.45 5.44 936.85 TP

1.88 942.52 0.81 940.64 TP

4.87

937.7

937.65

Datum elev gages
#1 & #3

5.84

936.7

936.68

Datum elev gages
#2 & #4

4.58

937.94

New BM - top of
SE strap for
underground
cable (approx 50'
north of gage box)

1.21 941.72 2.01 940.51 TP

5.87 943.16 4.43 937.29 TP

10.02 945.45 7.73 935.43 TP

0.67

944.78

(944.75)

BM

Readings (Apr. 23/64)

P1 -0.8

0.5

P2 +0.5

✓

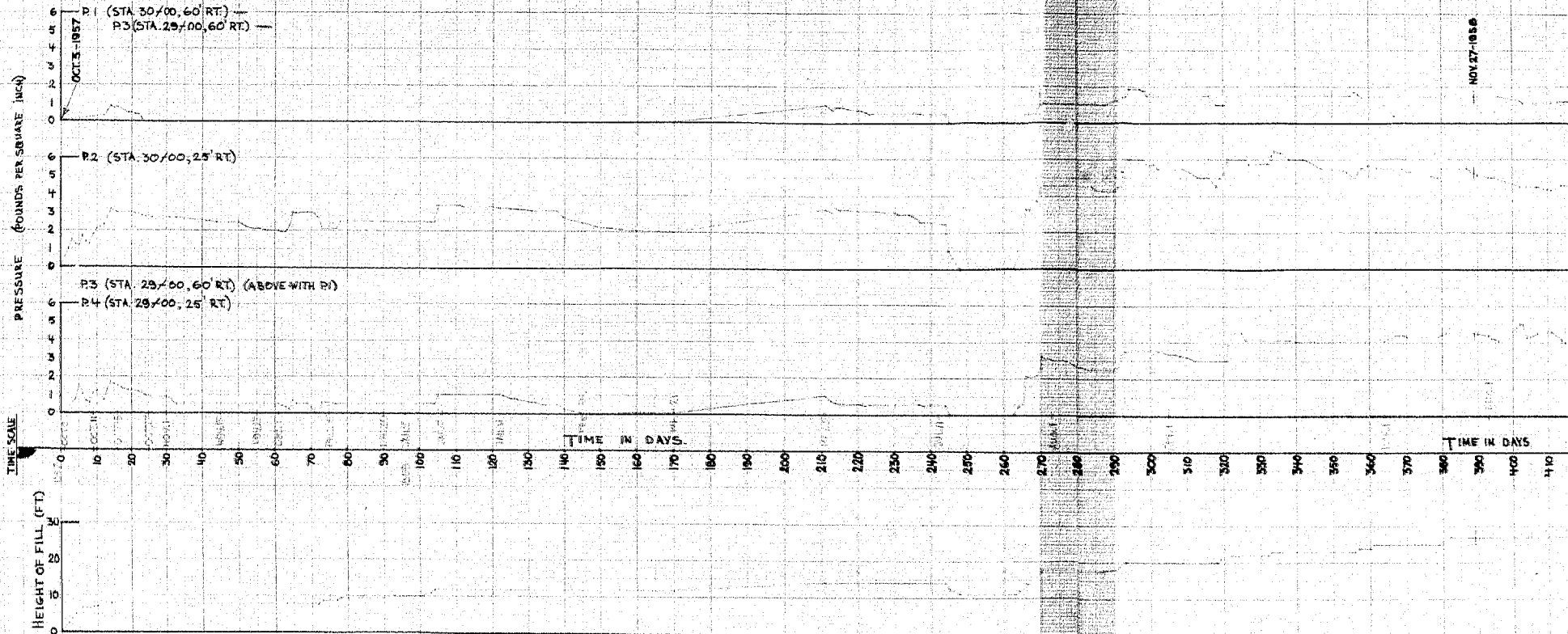
P3 -1.5

1.5

P4 +2.0

2.5

HUNTSVILLE BY-PASS



ME IN DAYS

390
400
410
420
430
440
450
460
470
480
490

500
510
520
530
540
550
560
570
580
590

TIME IN DAYS

600
610
620

NOV 27-1956

JULY 5 1959

1935

1945

1955

APR 17-1964

P1

P2

P4