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HIGHWAY RESEARCH REPORT

EMBANKMENT TESTING WITH THE MENARD PRESSUREMETER

A sub-project of:

MOVEMENT WITHIN LARGE FILLS

INTERIM REPORT

68-22

STATE OF CALIFORNIA

TRANSPORTATION AGENCY

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

MATERIALS AND RESEARCH DEPARTMENT

RESEARCH REPORT

NO. M & R 632509-2

Prepared in Cooperation with the U.S. Department of Transportation, Bureau of Public Roads May, 1968

STATE OF CALIFORNIA—TRANSPORTATION AGENCY RONALD REAGAN, *Governor*

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

May, 1968
Interim Report
No. M & R 632509-2

Mr. J. A. Legarra
State Highway Engineer

Dear Sir:

Submitted herewith is a research report titled:

EMBANKMENT TESTING WITH THE
MENARD PRESSUREMETER

a sub-project of

MOVEMENT WITHIN LARGE FILLS

TRAVIS W. SMITH
Principal Investigator

ROBERT E. SMITH
Co-Investigator

Assisted By

Dale W. Sathre

Very truly yours,

JOHN L. BEATON
Materials and Research Engineer

ACKNOWLEDGMENTS

Appreciation is expressed to Districts 01 and 07 for the cooperation and assistance extended in this project. An acknowledgment is made of the contributions of all those within the department who make it possible to conduct and complete such a program.

This is one of several reports to be prepared on various phases of the project, "Movement Within Large Fills." This work was conducted under the HPR Work Program (D-3-3) in co-operation with the U.S. Department of Transportation, Federal Highway Administration, U.S. Bureau of Public Roads.

The opinions, findings, and conclusions expressed in this publication are those of the authors, and not necessarily those of the Bureau of Public Roads.

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ABSTRACT: The testing and analysis of data from the investigation of three high embankments with the Menard Pressuremeter is reported. The pressuremeter is a rubber-walled hydraulic cell device for the in-place testing of soils. A method of estimating shearing strength based on test data is proposed. Estimates of embankment strengths in terms of parameters analogous to angle of internal friction (Ø) and cohesion (C); and values of a stress-strain modulus are given for the embankments tested. It is concluded that the particular device is somewhat limited in practicality, in its present form, for embankment investigations. However, it demonstrated the feasibility of in-place testing, and provides the basis for further development.

KEY WORDS: Soils, testing, analysis, in situ methods, testing equipment, soil subsurface testing, strength, strength theory, stress strain relations, embankments, fills, field tests, stability analysis.

CONTENTS

INTRODUCTION

The July, 1966, work plan for the "Movement Within Large Fills" project cited evidence of fill subsidence, sideslope distortion, and damaged structures; resulting from extreme stresses within high fills. A proposal was made to enlarge this research to include a general investigation of the soil stresses, stress-strain relationships, and the structural integrity of several high embankments to be constructed. This included instrumenting these fills to measure horizontal and vertical movements, and static pressures at critical locations.

Under this program it was also proposed to evaluate the Menard Pressuremeter, a patented device developed by Louis Menard of Paris, France. This is basically a hydraulic cell which applies an incremental load to the walls of a borehole and allows measurement of the resultant deformation. A record is kept of the increments of pressure versus the volume of fluid required in the expansion of the cell. The process is usually repeated at several locations down the borehole. From this information it is possible to estimate the shearing strength of the soil, and to calculate a load-deformation modulus.

This report is concerned with the investigation of the following three embankments with the pressuremeter:

Liebre Gulch, a 210-ft. high shale fill on Interstate Route 5, about 50 miles north of Los Angeles. Here it was desired to obtain information on the strength characteristics and performance of the soils at the distressed arch culverts.

Squaw Creek, a 383-ft. high mixed shale fill on U.S. 101 just north of Cummings, about 105 miles south of Eureka. The object of testing this fill was to obtain the stress-strain relationship of the embankment soil, and to verify the in-place strength of the selected and specially compacted "Zone B" material.

Chadd Creek, located about 28 miles south of Eureka just below the town of Pepperwood, is a clayey shale embankment approximately 100 ft. high. This embankment was investigated with the pressure meter to gain added experience in the testing of fills with the device. It was also desired to develop a stress-strain modulus for use by the Bridge Department in their computer analysis of soil stress and its effect on culverts.

The first portion of the report is given to a brief statement of the factors which led to the evaluation of the pressuremeter in embankment testing. This is followed by a description of the equipment. A presentation is made of the approach taken in the interpretation of testing. This includes an analysis of strength data based on the Mohr's strength circle concept, which provides information on the relationship of the shearing strength of the embankment with depth. It is shown that this functional relationship may be expressed in parameters similar to the conventional angle of internal friction (Ø) and cohesion (C). Also, the derivation of a deformation modulus from the pressuremeter data, analogous to the modulus of elasticity, is included. The moduli obtained in testing embankments, when modified by the appropriate constants, should be satisfactory for use in those approaches to soils analysis based on an elastic similitude, such as finite element incremental analysis.

The pressuremeter testing of the individual embankments is discussed separately, together with a presentation of other information as seemed appropriate.

CONCLUSIONS

Liebre Gulch

The objective of gaining information on the embankment soil over the distressed arch culverts has been accomplished. The Liebre Gulch embankment has adequate strength and is stable in its "as constructed" condition. However, it is known that there was ground moisture in the lower portion of this fill near the concrete arches. It is believed that the shale in this vicinity which had granular characteristics when placed, deteriorated under the influence of this moisture and pressures, and became cohesive in nature. This created high lateral loads on the reinforced concrete culverts for which they were not designed, thus causing their distress. Any future activity which may affect the drainage of ground water, or cause the impounding of water, should be preceded by a thorough investigation of the possible consequences on the stress distribution within the embankment.

Squaw Creek

The strength of the Zone B material is equal to, or greater than, that assumed in the design of the embankment.

The stress-strain moduli were determined for this fill, and will be correlated with the data from the horizontal movement and soil pressure instrumentation in a later report.

Chadd Creek

The stress-strain moduli information desired for this embankment was developed satisfactorily with the pressuremeter testing.

General Remarks

1. It is possible with a device such as the pressuremeter to estimate the in-place strength of an embankment soil. The general relationship of strength and depth may be determined. If desired, this relationship may be expressed in terms similar to the conventional parameters, angle of internal friction (Ø)

and cohesion (C). These values are relevant to the particular test conditions, and assumptions made, and may not be directly comparable to the results of laboratory tests on the same soil.

2. Curves, plotted from test data, show a proportional range from which a load deformation modulus can be determined. This modulus requires a mathematical correction for use in various applications.

3. There may be considerable variance in the moduli between soils of equivalent ultimate strength.

4. There are mechanical problems with the present pressuremeter equipment which somewhat limit its practicality for embankment testing. The principal difficulty lies in the dependence on rubber cells to apply the loads. These are fragile and prone to rupture, limiting the amount of strain possible.[1] To a lesser degree, the lack of a check valve in the probe limits the device for testing at depths much greater than 100 ft. The best use of the present equipment is probably in deposits of soft and medium strength, naturally occurring materials; and in shallow foundation investigations.

5. The present device presupposes the existence of a clear, clean borehole. These are expensive at best, and at times impossible to maintain throughout the testing routine. This factor becomes an increasing problem at greater depths.

RECOMMENDATIONS

1. The pressuremeter should be utilized over an extended period to permit the investigation of a greater variety of foundation conditions.

2. It is believed that the immediate need for improved methods of in-place testing merits further study of the general problems involved. It is recommended that research be undertaken in the development of improved methods of in-place testing of soil strength, and other properties.

3. The active program of investigating new soils exploration methods as they become available should be continued. An example of a device which may be suitable for study is that developed by Handy and Fox at Iowa State University for the direct shear testing of boreholes.(1).

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[1] We are informed by Geocel that improved membrane material is available which may alleviate problems arising from this source.

DISCUSSION

HISTORY

The pressuremeter equipment was demonstrated at the Materials and Research Laboratory in April, 1966, by Louis Menard. Subsequently a service contract was negotiated with Geocel,[1] the firm handling the pressuremeter in the United States, for the lease of the Menard device during the period from May 16, 1966, to November 1, 1967.

The opportunity was taken to use the pressuremeter to test three high embankments at different locations within the state. This is reported in later portions of the discussion. The investigation of these embankments was done in connection with a current, federally financed, research project entitled, "Movement Within Large Fills." This project is under the administration of the Materials and Research Department. Its object is to obtain information on the internal movement, soil stresses, and strength parameters of highway embankments now being routinely constructed to unprecedented heights.

The Menard Pressuremeter was also used to measure the in-place modulus of a shale formation for another Bureau of Public Roads financed research project entitled, "Rebound of Materials in Highway Cuts." This pressuremeter data will be included in the final report for that project.

EQUIPMENT, TESTING AND ANALYSIS

Equipment

The pressuremeter equipment is shown in the following Photo Nos. 1, 2, and Figure 1; also Photo Nos. A-1, A-6, in the Appendix. The total apparatus consists of: a source of pressure, a source of water, the volumeter, coaxial tubing, and the testing probe for insertion in the borehole.

The volumeter consists of: a water reservoir and a sight tube to indicate its water level, a high pressure gage indicating the bottle pressure, a regulator to control and reduce the pressure, gages to show the pressure in the water and air cells, and the necessary valving. Originally water was introduced in the volumeter reservoir by pouring into a small funnel. A garden sprayer was later modified to more easily add controlled amounts of fluid.

The probe is a metal cylinder upon which the inner water cell (measuring) and outer air cells are constructed. This may be most easily visualized by referring to the Photo Nos. A-1, A-4. The

[1]Present address: Geocel, 16027 West 5th Avenue, Golden, Colorado 80401.

measuring cell reflects the radial strain of the central portion of a larger stress field, enabling the assumption of plane radial strain in the analysis of data. This is accomplished with the dual cell construction of the probe which provides a larger total stress field than is measured with the inner measuring cell. The pressure in the air cell is reduced slightly below that of the water cell. This insures firm and consistent contact of the measuring cell with the central portion of the air cell when the probe is expanded in a borehole.

The tubing between the probe and volumeter is coaxial, see Photo No. A-6. Air is conducted in the outer tubing to the air cell, and water through the small inner tube to the measuring cell. As the outer tube is carrying almost as high a pressure as the inner, it is not necessary to compensate for expansion of the inner tubing (leading to the measuring cell) when correcting the test data.

Field Operations

Connectors, Photo Nos. A-4, 5, were fabricated to go between standard drill rod of various sizes and the probe. This enabled handling the probe with the windlass and hydraulic lift on the drill rig. The probe used to gather the bulk of the data presented in this report is about 2 ft. long and a nominal 2-3/4 in. in diameter, or "NX" size.

It is desirable to have the borehole itself only slightly larger than the probe. Sufficient clearance must be provided however to enable insertion and removal of the instrument. The probe used is slightly greater than 2-3/4 in. in greatest diameter. A nominal 3-1/8 in. hole drilled with a rotary rock bit was found to be satisfactory. The best results were obtained by keeping the borehole filled with heavy drilling fluid. When testing concurrently with the drilling operation, the presence of the drilling mud does not seem to alter the test data. Without its use the boreholes tend to ravel which obstructs the insertion of the probe, or causes tearing of the membrane. If raveling occurs while the probe is in the hole it may engender complete loss of the unit when withdrawal is attempted.

In testing these fills it was generally found that only one test was obtained with each probe fabricated. The total expansion required to get a satisfactory data curve is near the maximum the probe will stand. It is considered essential to get as complete a stress-strain curve as possible for each test to enable consistent interpretation. Consequently, pressure increments were taken till the soil was positively in shear, or the probe ruptured. The fabrication of a new cell requires about one-half hour. It is convenient to have a portable bench equipped with a pipe vise for the fabrication of these units. Two probes of the size used in testing these embankments were on hand, allowing the preparation of an extra unit as time permitted.

Figure 1

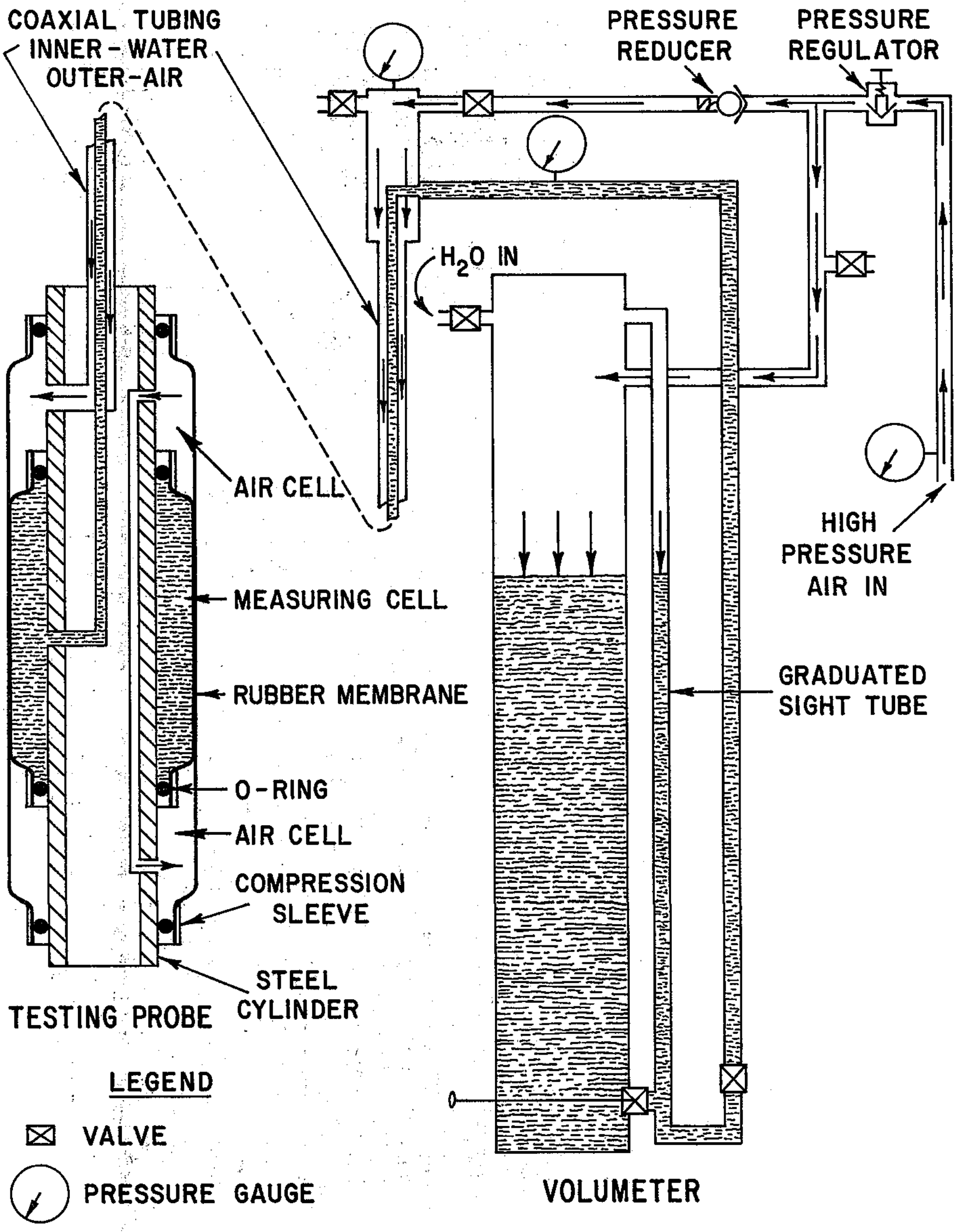
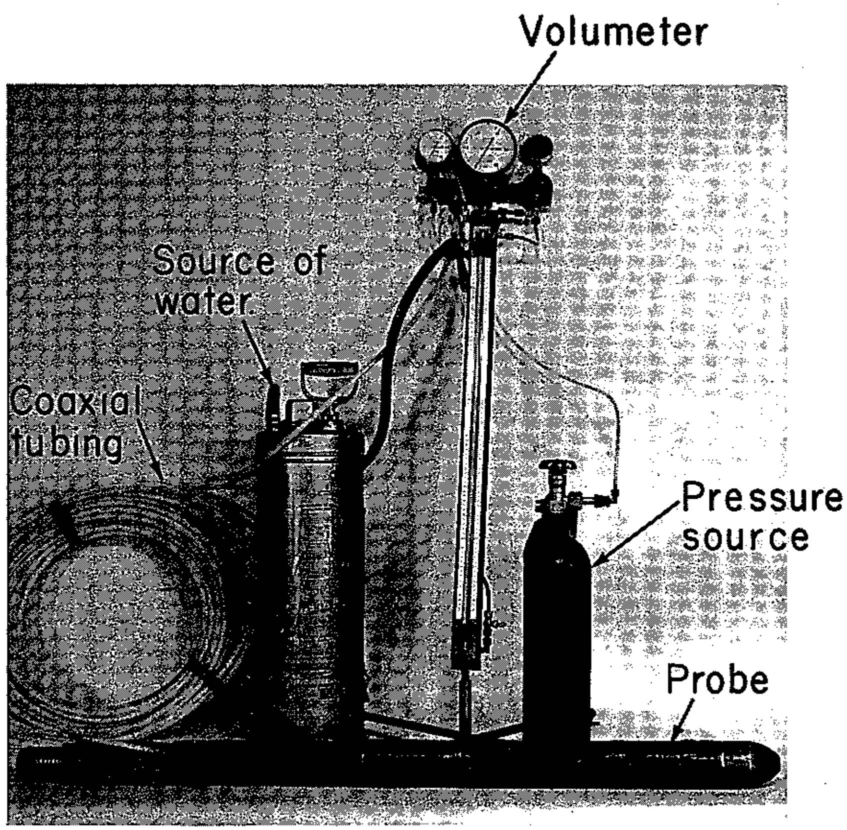

COAXIAL TUBING
INNER - WATER
OUTER-AIR
PRESSURE
REDUCER
PRESSURE
REGULATOR
H_2O IN
AIR CELL
MEASURING CELL
RUBBER MEMBRANE
O-RING
AIR CELL
COMPRESSION
SLEEVE
STEEL
CYLINDER
TESTING PROBE
HIGH
PRESSURE
AIR IN
GRADUATED
SIGHT TUBE
VOLUMETER
LEGEND
VALVE
PRESSURE GAUGE

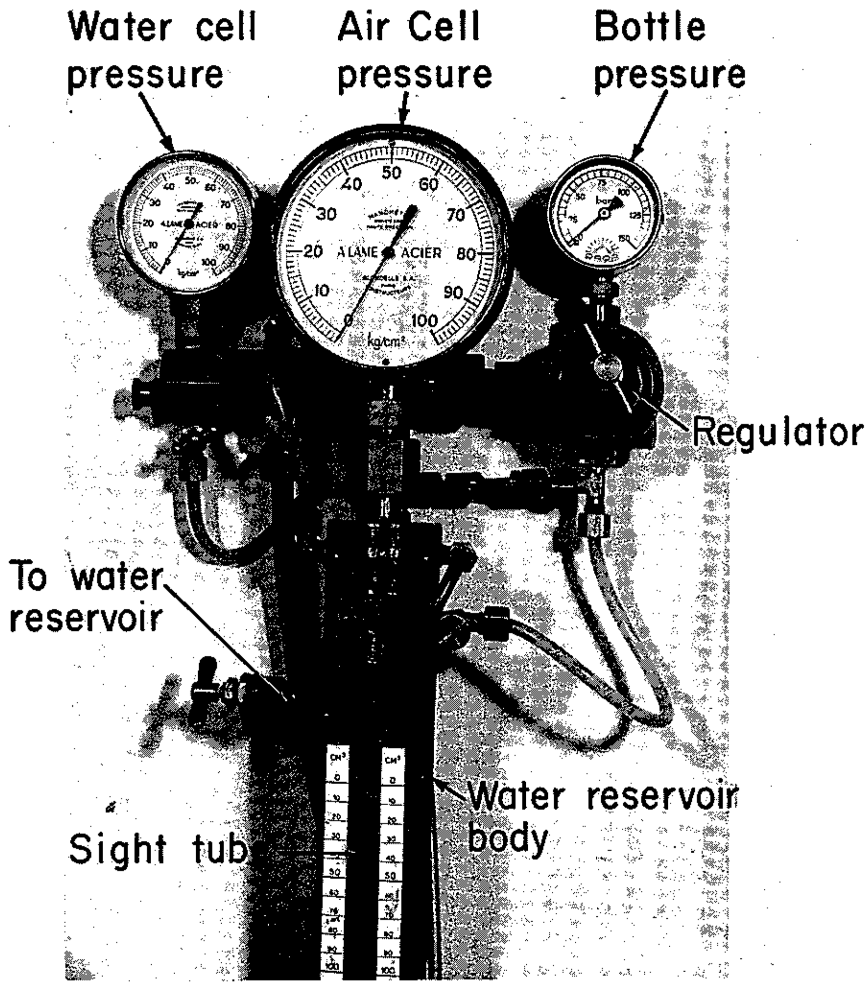
Figure 2

PHOTO NO. 1



Assembled pressuremeter equipment.

PHOTO NO. 2



Volumeter

Generally, it was possible to have the pressuremeter equipment ready to go when the hole was drilled to the next desired depth. The individual test would then take up to an hour including going in and coming out of the hole. This varies, of course, with the depth and conditions encountered. It was found that an efficient operation was effected by testing these boreholes at intervals of around 15 ft.

Test Data

The procedure for taking a test at a depth is straightforward. The fluid level in the sight tube is recorded both at 30 seconds and at 60 seconds after the incremental increase in pressure. Provision is made on the field data sheet for the technician to plot the data while the test is being performed. The volumetric displacement presented in the graphs of this report is the 60 second displacement. The differential between the 30 and 60 second reading is regarded as "creep." This value is used as an aid in determining when the soil is in failure. Each pressure increment takes up to two minutes to complete.

The lower curve represented by the solid line is the calibration or inertia curve for the bare probe expanded in air. The parallel curve (dashed line) is the corrected data curve for inertia and head of water, if required. (See Figure 2).

The portion A to B on the curve is the probe expanding to meet the walls of the borehole. The recompression of the borehole is represented by the portion of the curve B to C. From C to D is the proportional portion of the curve, or "pseudo-elastic" as termed by Menard.(2) The soil strain within this range of stress is accompanied by little or no shearing between soil particles. Between D and E the soil goes into shear.[1] It is well to note that in many cases the curve is also proportional, but with flatter slope, in this shear portion. Thus, though the soil is in shear, it is not in a condition of failure. The maximum strength of the soil is mobilized at the estimated point of failure, point E.

From E to F the immediate zone around the cell is in failure, although additional increments of pressure are required as the failure progresses. In summary, the pressuremeter data curve traces the progress of the soil in the immediate zone around the probe as it goes from recompression through failure under the increments of pressure applied to the interior of the borehole.[2]

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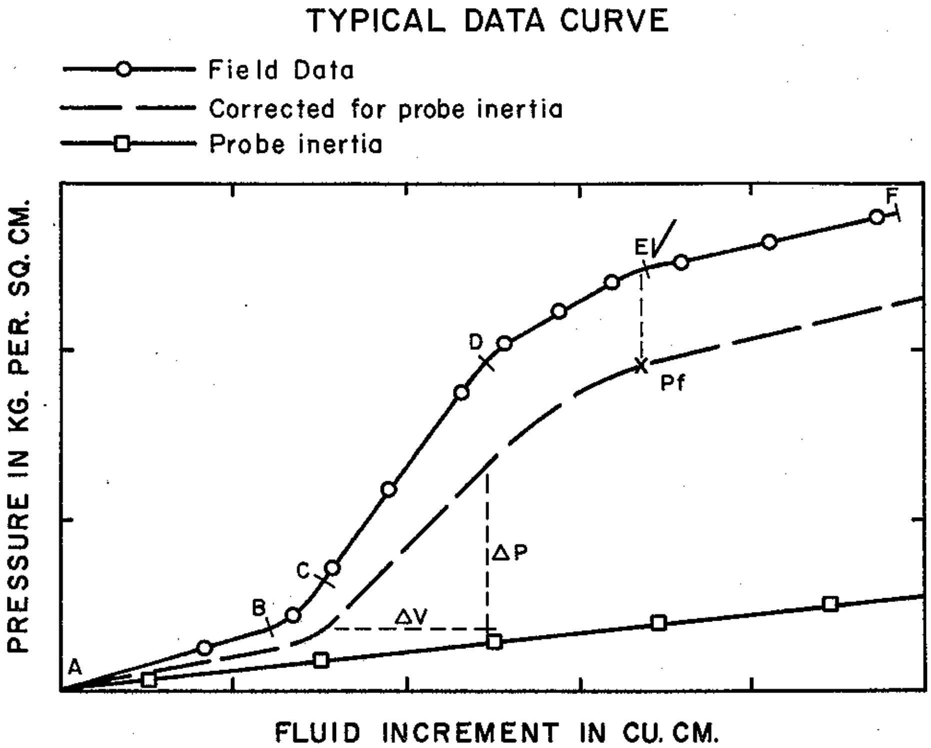
[1]Terzaghi observed an elastic phase and a shearing phase in regard to the decrease of pressure behind a model retaining wall when the wall was moved away from the soil. Engineering News Record, Sept. 30, 1920. pp 637.

[2]This general interpretation and the following suggested method of analysis of the pressuremeter strength data is that of the writer, and differs from that proposed by Menard.

The plots of pressuremeter data from the embankment testing program may be found in Appendix C, Figures 2-6, 8-14, 16-20.

A curve plotted from typical data is shown. The solid line is plotted through the field data points.

Figure 2



The similarity in configuration of this curve and that of a conventional stress-strain diagram is evident. The ordinate in in kg per sq cm which represents the pressure within the probe, and against the wall of the borehole. The abscissa is in cu cm of fluid added to the probe. Although given in cubic units, this is very nearly a linear function of the expanding radius of the hole for small increments of fluid.[1]

Soil Strength

The point of failure, P_f, is arbitrarily taken as the best estimate of point E, the transition from the shearing phase into failure, see Figure 2. This is consistent with the concept of cohesion and angle of internal friction expressing the maximum shearing strength the soil can develop before failure, being permitted sufficient strain.

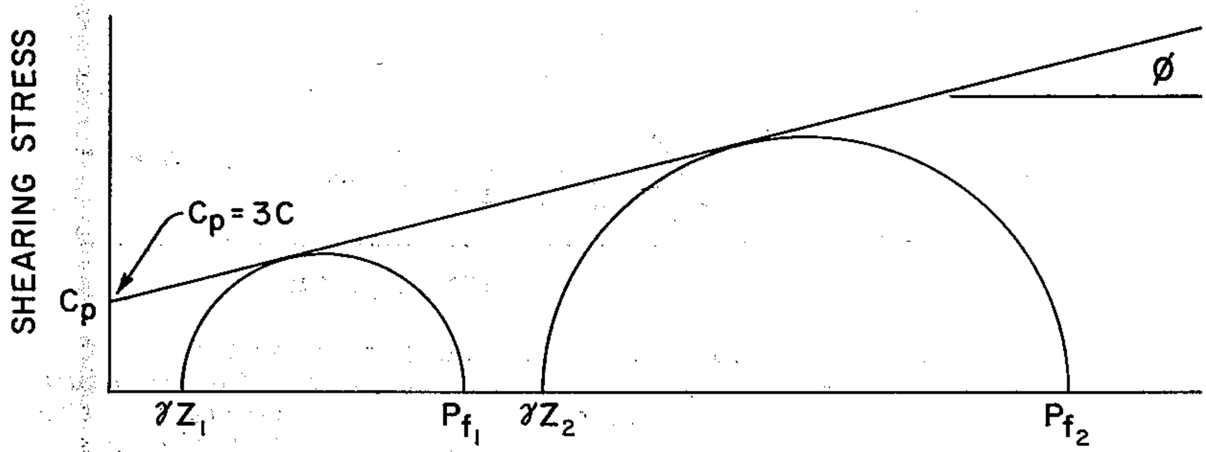
[1]Reference may be made to equations (2-5) and (2-6) in Appendix B.

It is acknowledged at this point that the manner in which passive resistance is mobilized in a borehole with the pressuremeter test undoubtedly differs from the simple assumptions made in the following analysis. Therefore, the shearing strengths (and parameters $\emptyset$ and C) estimated on the basis of such a test would vary from those determined for the soil by conventional laboratory testing procedures. It is, however, believed that the approach offers a basis for an initial evaluation of the data. Further experience may suggest suitable modifications of the suggested procedure for analysis of pressuremeter test information.

For the purposes of the following analysis, it is assumed that the overburden pressure, γZ, represents the minor stress, $\sigma 3$; and that the failure pressure, P_f, is equal to the major stress, $\sigma 1$. The overburden pressure is assumed to be the depth of the test, Z, times an average in-place wet unit weight, γ.

These values may be plotted for individual tests at different depths, and the Mohr's strength circles drawn, (assuming similar materials) as in Figure No. 3.

Figure 3



The parametric expression for this relationship of P_f and γZ is:

$$P_f = \gamma Z \tan^2 \left[45 + \frac{\emptyset}{2}\right] + 2C_p \tan \left[45 + \frac{\emptyset}{2}\right]$$

where C_p is defined as the pressuremeter cohesion, and $\emptyset$ is the apparent angle of internal friction of the soil for the pressuremeter test. The development of this expression for the pressuremeter method of test is given in Appendix B, equations (1-1) through (1-3). It is explained that the modification of the shearing stress intercept ($C_p = 3C$) from that of the conventional triaxial test data is due to the difference in the mechanics of failure of the two tests.

Since the embankment material is not always similar in character throughout the fill, the following procedure is helpful: A value P_z is determined, which is somewhat analogous to the deviator stress in a triaxial test, by subtracting the calculated overburden stress from the failure pressure $P_z = (P_f - \gamma Z)$. P_z is plotted versus depth of test, which helps to determine the materials with similar and dissimilar strength characteristics. For example, if there are two materials, each with unique strength properties; the plot of P_z versus depth will show two trend lines. Such a plot is presented with each individual discussion of the embankments tested, Figures 5, 6, 7. The trend lines shown are visual estimates, and not linear regression lines.

The Mohr's strength circles for the similar materials in the embankment can now be drawn as a family of curves on one graph. The general functional relationship of the soil's shearing strength with depth may now be estimated. If desired, this may be expressed parametrically in terms of an angle of internal friction (Ø) and cohesion (C_p = 3C) by fitting the best tangents to the groups of circles. These constructions for the respective embankments are also shown on Figures 5, 6, 7. As previously discussed, these parameters are relevant to the particular test conditions, and may not be directly comparable to laboratory test values for the same soil.

Correlation with Laboratory Testing

In an attempt to present as much data as possible, the results of selected pertinent laboratory testing are included. However, since these data represent tests performed on remolded specimens, direct correlation cannot be expected. Such data may be helpful in judging the reasonableness of the field evaluation, and are presented in the discussions of the individual embankments.

Pressuremeter Modulus

A modulus, E_p, may be computed for the proportional zone C to D of the pressuremeter test graph. The derivation of this modulus may be found in Appendix B. It is developed from the equation for radial strain of a point on the interior of a hole in an elastic material of infinite extent. The basic equation is given by Timoshenko who credits it to Lamé. (3, p. 208). The modulus E_p is basically that suggested by Geocel (2, p. 6), and Rocha (4); except that it does not include a correction factor based on Poisson's ratio to get a calculated modulus of elasticity "E." This may be done by multiplying E_p by the factor (1 + U), where U is Poisson's ratio. It may be seen that the assumptions regarding radius of borehole, volume of probe, etc., become a constant. E_p is therefore a direct function of the basic test data times a constant.

This basic modulus, when corrected, is believed suitable for use in soils analysis computer programs based on an elastic analogy, such as finite element incremental analysis. A correction factor based on Poisson's ratio is necessary to use the modulus in specific applications. Formulas for a few of these may be found in Appendix B. Particular attention is called to equations (3-3), (3-4), (3-5). These indicate the theoretical relationships between stresses and strain in a confined soil layer (no lateral strain permitted) when subjected to a uniform lead. This condition is approximated by incremental soil layers in the central portions of embankments and foundations. It is noted in (3-5) that the lateral stresses are a function of the apparent Poisson's ratio u, and independent of the modulus of elasticity E.

In practice, the most direct procedure is to graph E_p versus depth. Then, for the particular depth or zone under consideration, a value of the modulus may be estimated. This type of plot is presented for each of the three embankments, Figures 5, 6, 7. The depths are with respect to the elevation at the time of testing. As there is a general tendency for E_p to increase with depth; the trend line is visually estimated and drawn through each plot. By this means, the effect of additional embankment height on E_p can be judged.

Caution must be taken that these moduli are used with stresses which lie within the proportional range of the pressuremeter data curve. In general, the overburden stresses lie within this range.

Poisson's Ratio

The effective or apparent Poisson's ratio for these materials must be estimated on the basis of the relative strength, stress level, and degree of saturation. Other work indicates that its apparent value is within the range 0.4 to 0.5 for embankments with significant percentages of clayey materials. This range is higher than commonly assumed. A discussion of the topic is beyond the scope of this paper; but in brief, the ratio of minor to major stress in the at-rest soil condition is higher than ordinarily assumed (5, 6, 7). Necessarily, this is associated with a high apparent Poisson's ratio.

An estimated value of the apparent Poisson's ratio for the individual embankments is presented in their respective discussions.

LIEBRE GULCH EMBANKMENT

Introduction

The first embankment tested was at Liebre Gulch on the New Interstate Route 5 under current construction, about 40 miles north of Los Angeles. At this site a specially designed reinforced concrete arch culvert with a span of 16 ft. beneath a 210-ft. high fill had failed under earth pressures imposed upon it.[1] This fill is built largely of a shale which is a hard competent material on excavation, but which breaks down rapidly on exposure to the effects of weather and ground moisture. It is a type of embankment material in which it is very difficult to take in-place samples for conventional laboratory evaluation.

Pressuremeter Strength Data

In July and August, 1966, the pressuremeter was used to test boreholes placed in the vicinity of the distressed culvert to determine the shearing strength versus depth of the embankment. It was also desired to obtain an in-place soil modulus for possible use in the analysis of the cause of failure.

Considerable difficulty was experienced in attempting to test this fill with the pressuremeter. It was intended to test at intervals through the fill and into the original ground. However, the borehole could not be kept open below a depth that would be about 190 ft. below the completed height of embankment. Circulation of drilling fluid could not be maintained, apparently because of voids in the lower embankment. Attempts to use force to insert and remove the probe only resulted in damage to the membranes. It may have been feasible to place casing inside the hole and proceed, but this did not appear justified under the circumstances. Therefore, tests were not run in the foundation soils.

Additional problems were caused by the poor quality of rubber tubing supplied for membranes at this time. Much of it appeared to be age cracked, which is believed to have contributed to the high rate of probe bursts experienced. Subsequently the department received a shipment of a heavier membrane tubing which performed more satisfactorily.

It was also found that the expansion of the probe under its own head of water caused the probe to stick at greater depths. Although the water supply valve was closed, the vacuum in the tube will only support a limited amount of fluid before cavitation occurs. This was alleviated somewhat by using a water supply tubing of smaller internal diameter as provided by Geocel. However, this advantage was offset by the excessive times required to fill and empty the probe, and a general sluggishness when taking a reading.

The original test data for Liebre Gulch is presented in Appendix C. A sketch and cross-section are provided showing the location of the test borings, Figure C-1. The individual graphs of pressure versus increments of fluid are arranged by boring

[1] "Culverts Fail Under Deep Fill," Western Construction, Dec., 1966, p. 36.

number and depth, Figures C-2, 6. It should be noted that all depths are given relative to the height of fill at the time of testing. The cross-section indicates this approximate height, and its relationship to the completed embankment.

A summary of the pressuremeter data analysis is given in Figure 5. Graph 5-B is a plot of P_z versus depth. A positive trend line is difficult to establish. This undoubtedly may be attributed in part to the fact that this was our first major attempt to use the device. The difficulty encountered in drilling a satisfactory hole, and the mechanical problems, all contributed to the scatter of the data. The Mohr's circle construction, Graph 5-C, seems to indicate an angle of internal friction that varies from 11° to 18° and a cohesion of from 0.3 to 1.5 tons per sq. ft.

It is concluded that the higher values represent the strength of the embankment, as constructed, in the zones tested. The low values were possibly caused by the excessive amounts of moisture introduced into the boreholes by the drilling operations. The results of the field investigation with the Menard Pressuremeter seem to agree with the laboratory testing in that the strength of this embankment material is variable and dependent, at least in part, on exposure to moisture.

Laboratory Strength Data

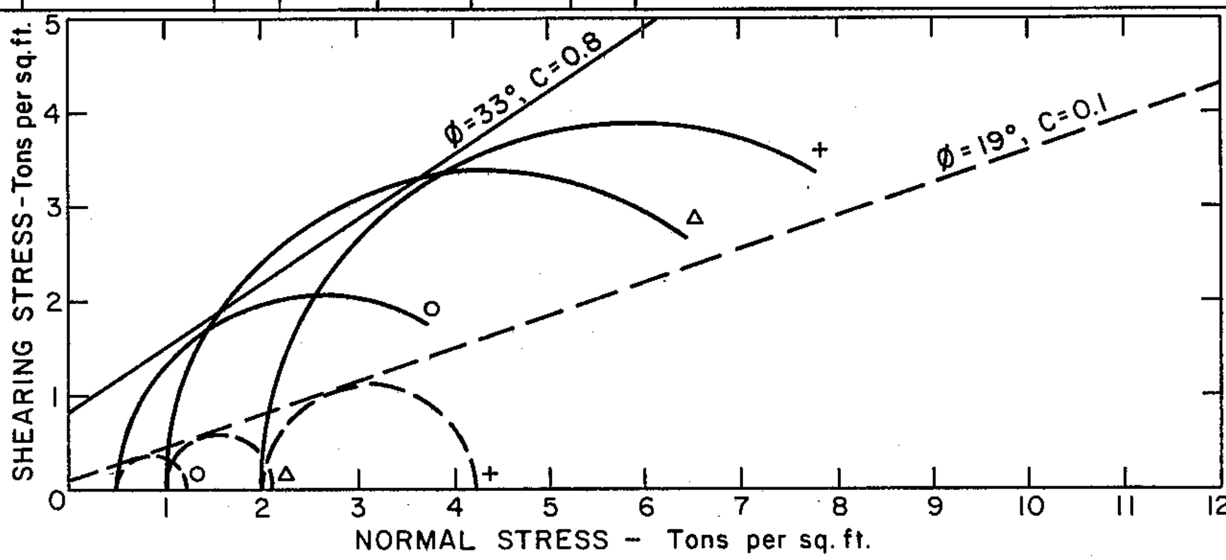
Two sets of triaxial tests were performed on remolded embankment material from the neighboring West Fork Liebre Gulch embankment. This embankment material is considered to be the same as that of Liebre Gulch as tested with the pressuremeter. The specimens were 2.8 x 5.6 in. cylinders fabricated to 90 percent relative compaction and at "impact" optimum moisture content, about 12.5 percent. Quick undrained triaxial tests were then run on a set of specimens at chamber pressures of 0.5, 1.0, and 2.0 tons per sq. ft., respectively.

Another set of specimens were prepared to the same moisture and density as the first set, and then tested under similar conditions, except that each was saturated under reduced chamber pressure. After this they were allowed to consolidate under the test chamber pressure until equilibrium was established as determined by longitudinal strain. Drained triaxial tests were then run on the specimens of the second set. The results are shown in the following Figure 4. There is a considerable difference in the indicated values of ϕ and C for the Mohr's construction of the first and second sets of data. These tests were run at the rate of 0.05 in. per minute. Pore pressures were not recorded, but water appeared to move freely from the sample. There was no evidence of dilatancy.

While it is possible that significant pore pressures may have developed due to the high strain rate, it is believed that the major cause of strength loss was due to slaking of the shale fragments during saturation and consolidation.

Figure 4

Sym.	Sample Ident.	Type Test	$\sigma 3$ T.S.F.	$\sigma 1-\sigma 3$ T.S.F.	Wet Unit Wt.	% Moist.	Remarks (West Liebre Embankment Material)
—o—	#1	UU	0.5	4.20	133 lb/ft^3	11.5	Remolded to 90% RC at opt H_2O, 12.6%.
—Δ—	2	✓	1.0	6.74	✓	11.3	
—+—	3	✓	2.0	7.65	✓	11.5	
-o--	#1	CD	0.5	0.69	✓	15.6	Remolded as above. Specimen #1 sheared
-Δ--	2	✓	1.0	1.14	✓	20.3	immediately after saturation. Specimens 2 & 3
-+--	3	✓	2.0	2.19	✓	19.5	saturated, then allowed to consolidate 2 days
							before shearing.



Pressuremeter Modulus

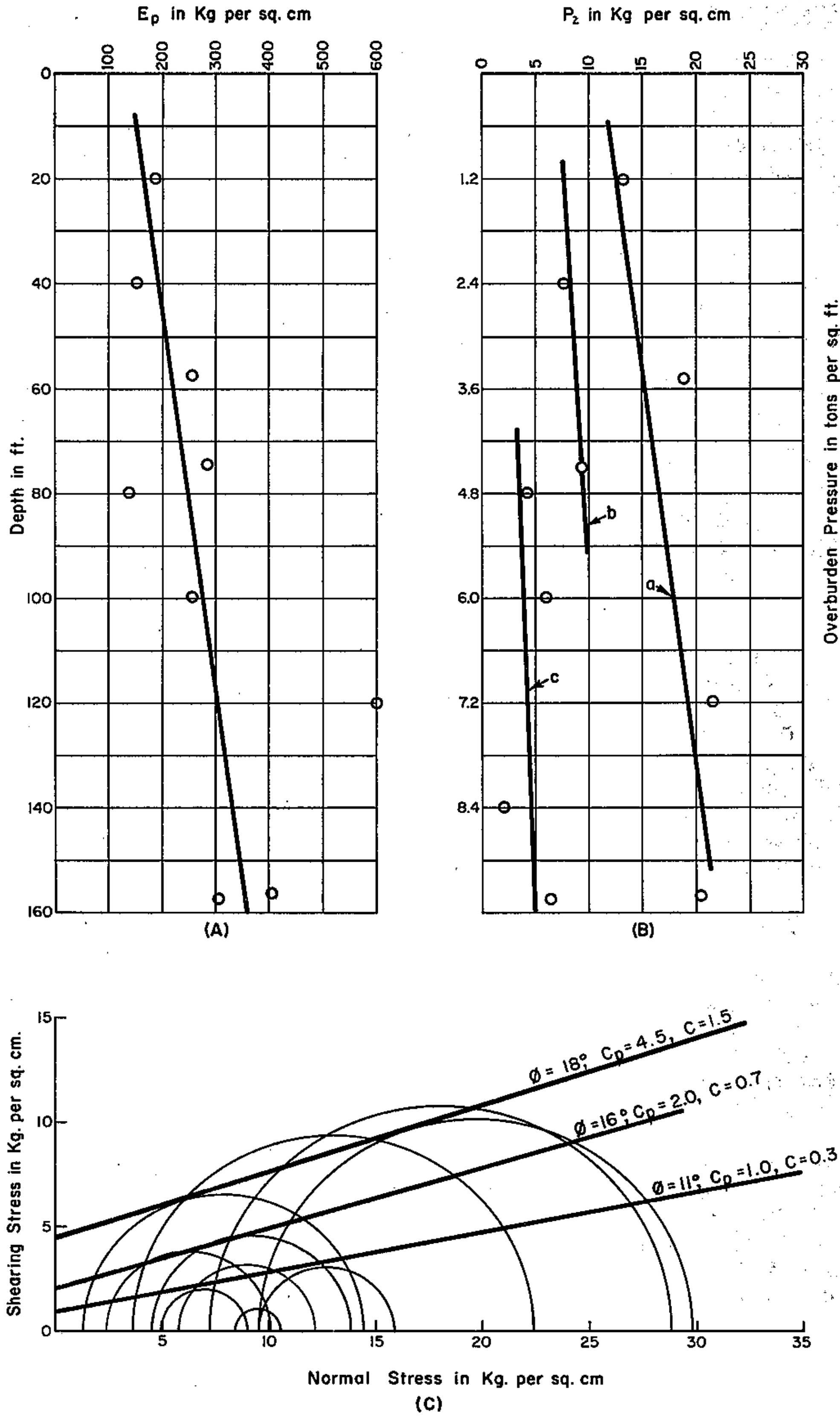
The pressuremeter modulus as calculated for the pressuremeter testing of the Liebre Gulch fill is plotted versus depth in the upper left graph, Figure 5-A. It appears to exhibit a linear correlation with depth, though somewhat erratic in the range of 120 to 140 ft. below the testing datum. A computed value of E_p for the test No. 8 is not plotted due to the peculairities of the data for that particular test, Figure C-5.

Poisson's Ratio

The effective Poisson's ratio for this material is estimated to be in the range from 0.45 to 0.50, depending on exposure to ground moisture.

Figure 5

LIEBRE GULCH EMBANKMENT - PRESSUREMETER



(C)

SQUAW CREEK

Introduction

The second high embankment to be tested with the pressuremeter was the Squaw Creek embankment on U.S. 101 just north of Cummings. This fill is labeled the "highest in California," approximately 383 ft. in height. Because of its height, its design required the material in the lower zones to be of higher quality than that in the upper zone. Figure C-7 in the Appendix illustrates the design concept. Zone C was to be constructed of selected rocky material. Zone B was to be of specially compacted high quality excavation with no clay or weathered detritus. Zone A was designed as roadway excavation compacted to 90 percent relative compaction.

The Materials and Research Department conducted the original testing program on which the design of the Squaw Creek embankment was based. (8) The soil contained boulder sizes to eleven inches and over, and required a large, high pressure triaxial device for analysis. This department did not have, at that time, the capability for large diameter triaxial testing. For this reason, arrangements were made to use the equipment available at the U.S. Army Corps of Engineers Division Laboratory, Sausalito, California.

The testing program was conducted on specimens prepared at varying moistures and degrees of compaction. Special attention was given to the material to be placed in Zone B. As a result of the testing, this zone was designed on the premise of achieving an in-place strength denoted by a ϕ of 25° (total stress) and a C of 0.25 tons per sq. ft. To this end it was specified that the selected material for this zone be placed at 93 percent relative compaction and at a moisture content of 1 to 3 percent less than optimum.

Additional Laboratory Testing

During the course of construction, additional triaxial tests were run on large diameter remolded specimens prepared from the soil actually incorporated in Zone B. Arrangements were made with Geo-Testing, an earth materials laboratory in San Rafael, and the California Department of Water Resources Materials Laboratory to conduct these tests.

The effective stress triaxial test values for the material sampled from Zone B compared well with those of the earlier program of tests on the proposed soils. This indicates that the material which was actually placed in this zone possessed shearing strength comparable to the soil of the original testing program.

Pressuremeter Strength Data

This fill was tested with the pressuremeter in March and early April of 1967 with the object of obtaining strength data for use in the verification of the in-place strength of the Zone B material. There was both snow and rain during the period of testing. Free moisture was encountered at several depths in the boreholes, and it was evident that the surface water could percolate into the embankment with relative ease.

The data curves for the pressuremeter testing at Squaw Creek are presented in the Appendix, Figures C-8, 14. The plan view and cross-section showing the location of the boreholes are shown on Figure C-7.

In the following summary data sheet, Figure 6, it is indicated that the Zone B embankment material at Squaw Creek has an angle of internal friction (Ø) of approximately 25° The cohesion (C) varies from 0.7 to 2.3 tons per sq. ft. An average for the embankment would be: Ø = 25°, C = 0.4 tons per sq. ft. On the basis of the analysis of the pressuremeter data, it is concluded that the strength requirement of the Zone B material was achieved in the Squaw Creek embankment.

Pressuremeter Modulus

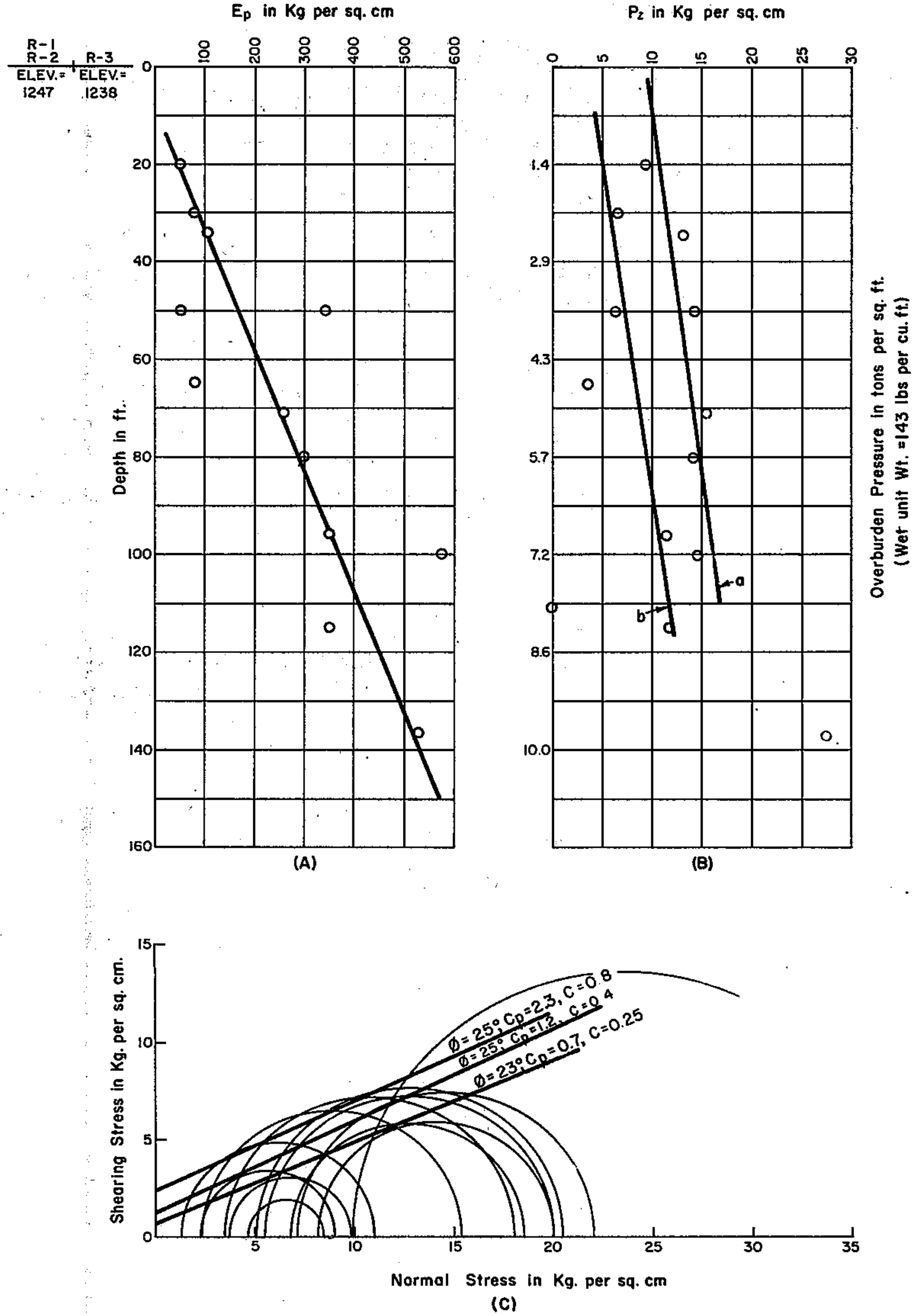
The pressuremeter modulus of the embankment at Squaw Creek, Figure 6-A, appears to have a linear relationship with depth. There is a pronounced scattering of this index in the vicinity of the 55 and 100 ft. depths.

Poisson's Ratio

The apparent or effective Poisson's ratio for this embankment soil is estimated to be in the range 0.40 to 0.45.

Figure 6

SQUAW CREEK EMBANKMENT - PRESSUREMETER



CHADD CREEK

Introduction

The embankment at Chadd Creek on U.S. 101, about 3 miles south of Pepperwood, was the third embankment tested with the pressuremeter. This embankment is about 100 ft. high over a multiplate culvert. This culvert is extensively instrumented for a research project being conducted by the Bridge Department. A borehole was placed in the Chadd Creek fill in April, 1967, and tested with the pressuremeter.

The objective in testing this embankment with the pressuremeter was to extend our range of experience in this type work. This could be done efficiently in conjunction with the investigation at Squaw Creek. Moreover, it was desired to obtain an in-place stress versus strain modulus of the fill. This modulus may be used in a computer analysis of soil stress based on a finite element incremental analysis concept, which will be conducted by the Bridge Department. The information will be correlated with other data gathered for the "Movement Within Large Fills" project.

No special problems were encountered at Chadd Creek in testing with the pressuremeter.

Pressuremeter Strength Data

Data curves for the Chadd Creek testing may be found in Appendix C with a sketch showing the location of the test borings, Figures C-15, 20.

The plot of P_z versus depth, Figure 6-B, seems to indicate either two materials, or one material exhibiting two strength conditions. The Mohr's construction indicates a ∅ of 32° and a C of 0.6 for the material of trend line (a). Trend line (b) representing the predominant material has a ∅ of 14° and a C of 0.6 tons per sq. ft.

Pressuremeter Modulus

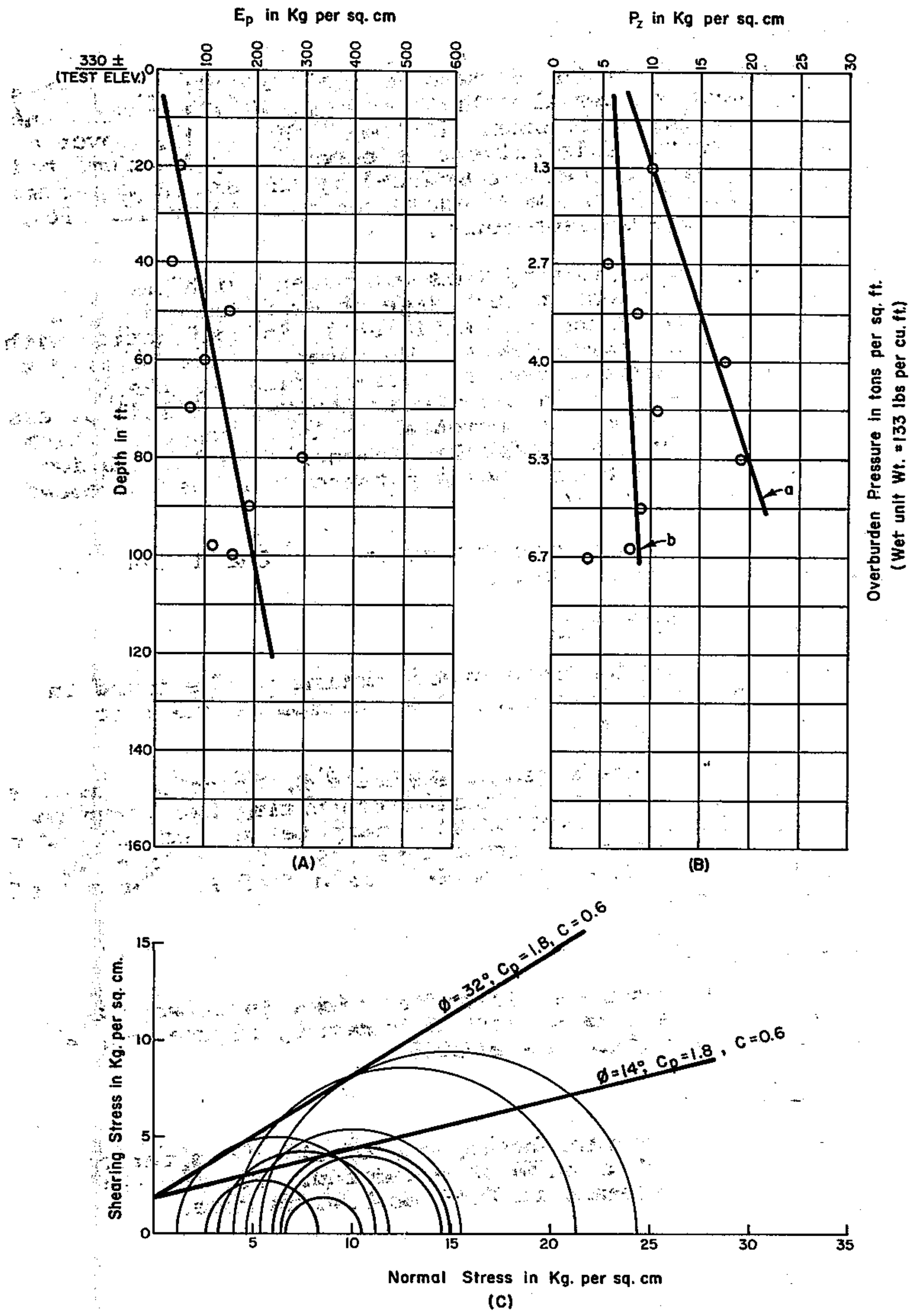
There is an approximate linear relationship between the pressuremeter modulus E_p and depth as shown by Figure 7-A, although considerable scatter is evident.

Poisson's Ratio

For the clayey material having a ∅ of 14°, an apparent Poisson's ratio of 0.45 to 0.50 is estimated. For the soil with a ∅ of 32°, an estimate of 0.40 to 0.45 is made.

Figure 7

CHADD CREEK EMBANKMENT - PRESSUREMETER



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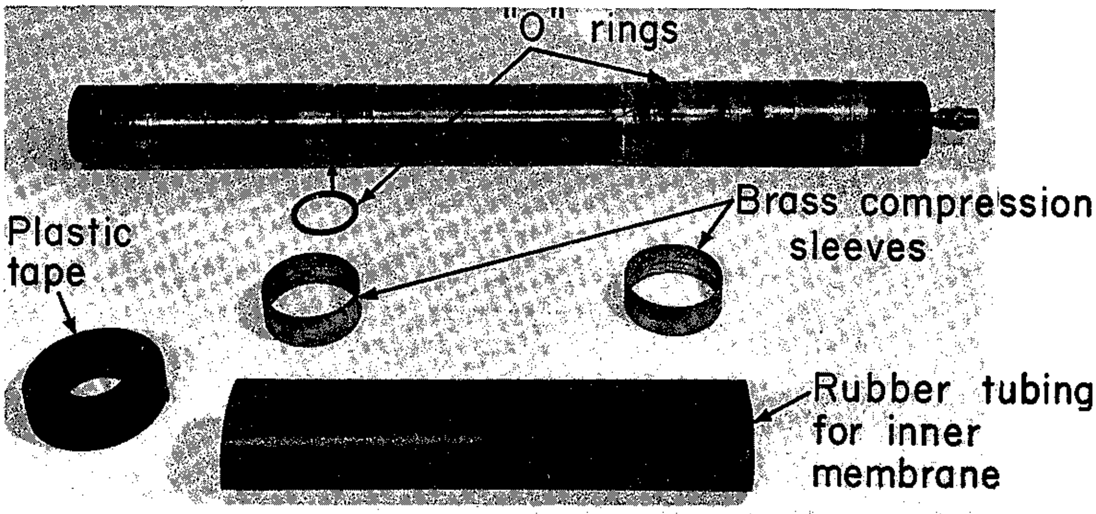
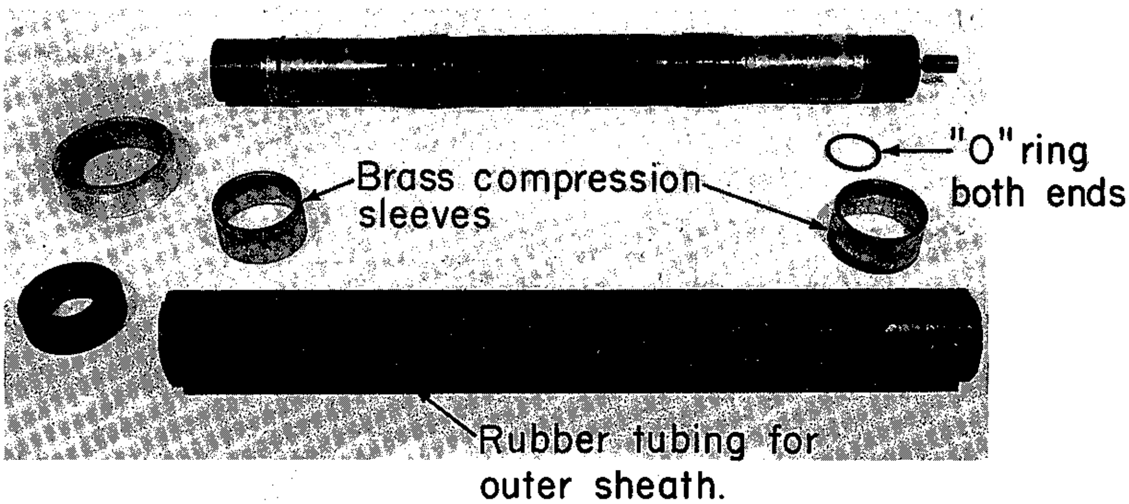
APPENDIX

PHOTO NO. A-1



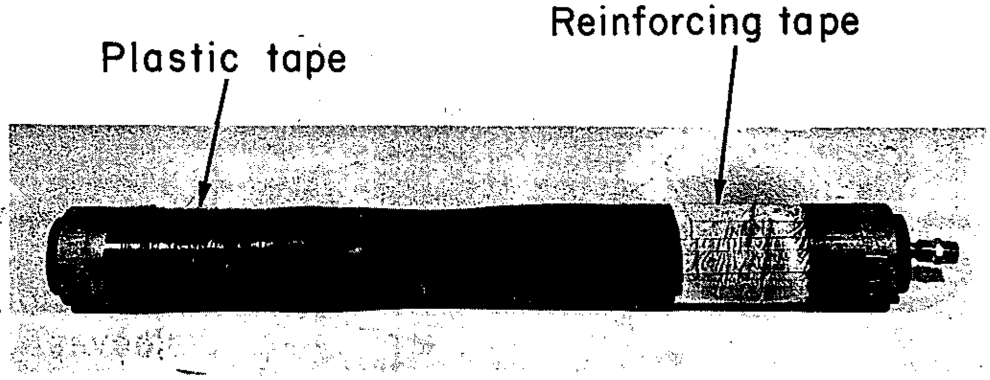
Bare probe with materials for fabrication of inner water cell.

PHOTO NO. A-2



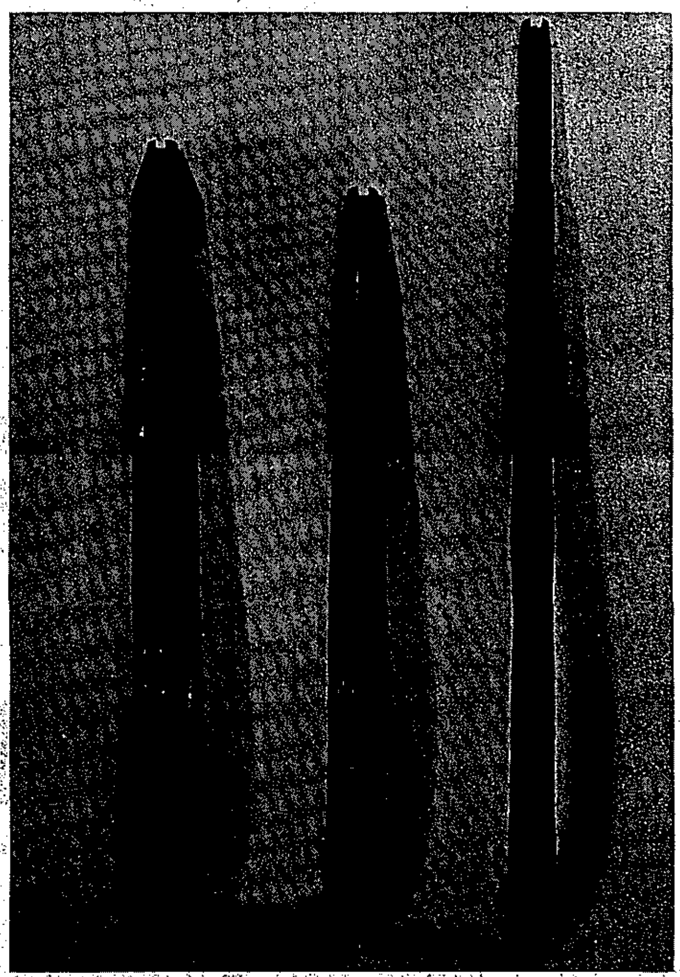
Probe with inner cell assembled and materials for outer cell.

PHOTO NO. A-3



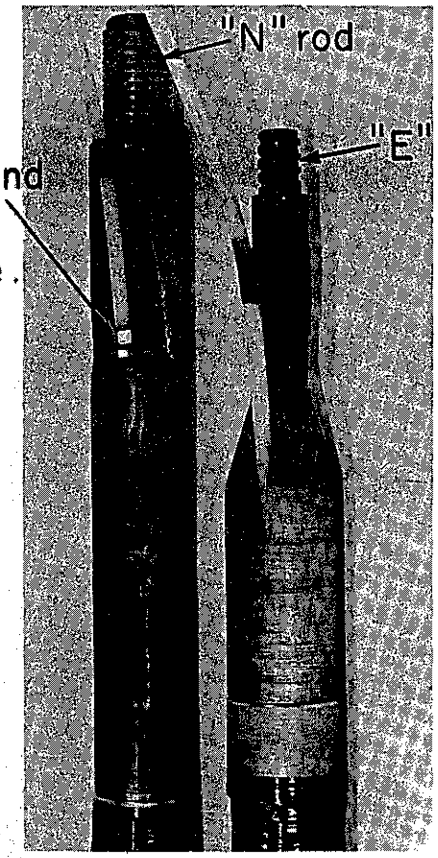
Assembled probe showing rayon or nylon reinforcing tape.

PHOTO NO. A-4



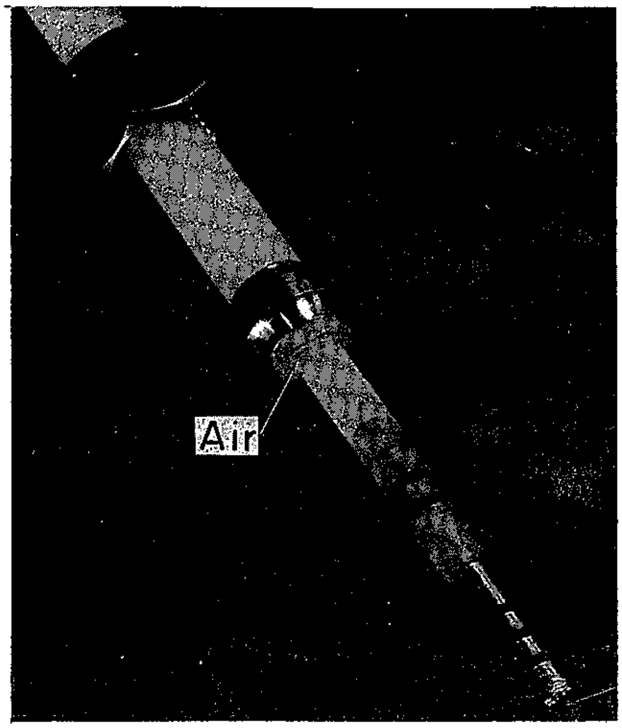
Probes with connectors for small diameter rod – also point covers.

PHOTO NO. A-5



Connectors for NX probe.

PHOTO NO. A-6



Coaxial tubing

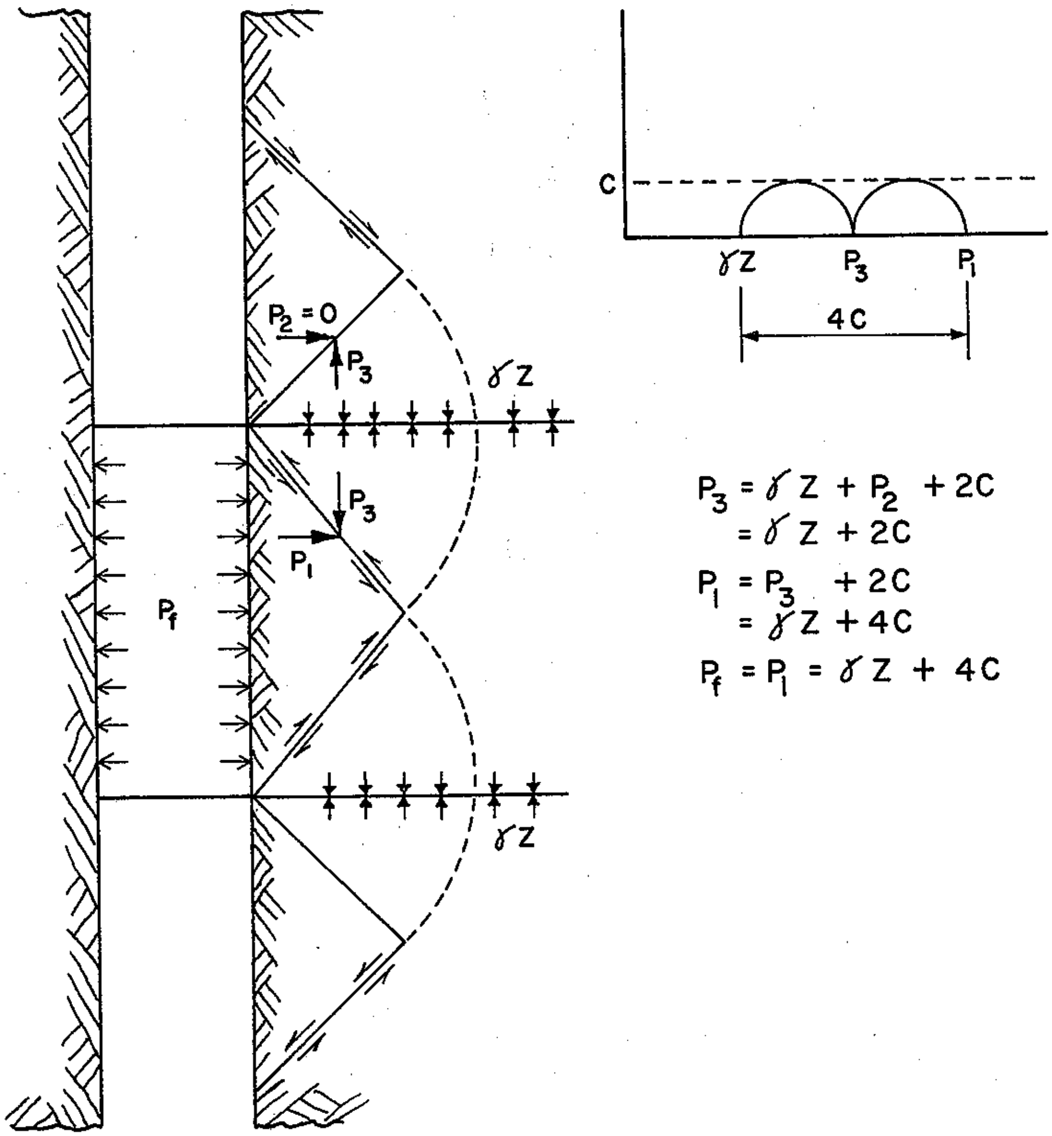
Connectors for HX probes

Coaxial tubes

B. DERIVATIONS

1. Pressuremeter Strength Equation

a. Soils having cohesion only:

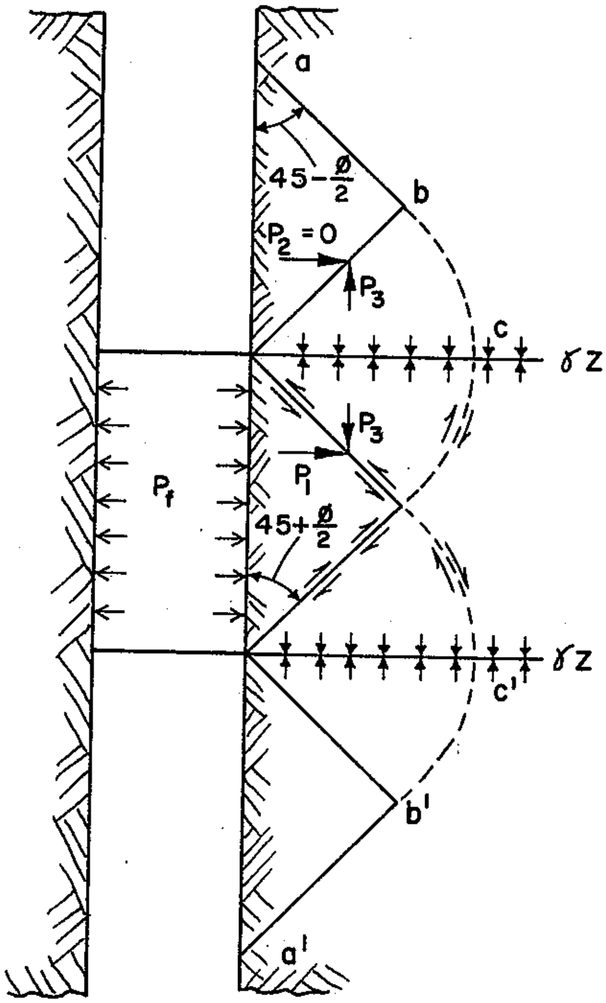


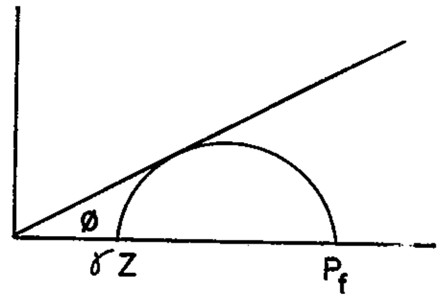
Experimental and field evidence by several investigators has shown that the maximum load which may be imposed by a strip footing on the surface of a plastic clay is in the order of 5.5C versus 4.0C based on an analysis similar to the above (9, p. 251). For a square or circular footing this increases to 7.4C due to the increased shearing area. (9, p. 251, 10, p. 172). Since the shearing area of a pressuremeter test is greater than that of an equivalent strip footing, a bearing value in shear for the borehole wall is presumed to be equal to 6C. The total stress at failure would then be equal to the overburden stress at that depth (γZ) plus 6C:

$$P_f = \gamma Z + 6C$$

(contd)

b. Cohesionless soils:





$$
\begin{aligned}
P_3 &= \gamma Z + P_2 \tan^2(45+\tfrac{\emptyset}{2}) \\
&= \gamma Z \\
P_1 &= P_3 \tan^2(45+\tfrac{\emptyset}{2}) \\
&= \gamma Z \tan^2(45+\tfrac{\emptyset}{2}) \\
P_f &= P_1 = \gamma Z \tan^2(45+\tfrac{\emptyset}{2})
\end{aligned}
$$

In this case, it is presumed that the simple Rankine relationship exists between the major and minor stress, (P_f, γZ). The shearing resistance of a. to c. is neglected since $P_2 = O$, thus:

$$P_f = \gamma Z \tan^2 (45+ \frac{\emptyset}{2}) \qquad (1\text{-}2)$$

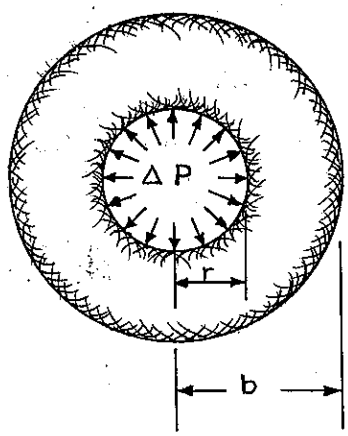
c. Mixed soils:

If the quantity C_p (pressuremeter cohesion) is introduced where $C_p = 3C$, the familiar parametric expression for the Mohr's strength envelope results.

$$P_f = \gamma Z \tan^2 (45+ \frac{\emptyset}{2}) + 2\, C_p \tan (45+ \frac{\emptyset}{2}) \qquad (1\text{-}3)$$

2. Derivation of pressuremeter modulus, Ep

A thick walled elastic cylinder with inner radius 'r' and outer radius 'b' is subjected to a differential inner pressure ΔP.



$$\Delta r = \frac{r \cdot \Delta P}{E}\left(\frac{r^2+b^2}{b^2-r^2}+\mu\right) \tag{2-1}$$

(3, p. 210, eq. 179)

Where: E = modulus of elasticity
μ = Poisson's ratio
Δr = change in radius 'r' due to ΔP

Transpose E and Δr and multiply through by $(b^2 - r^2)$:

$$E(b^2-r^2) = \frac{r \cdot \Delta P}{\Delta r}\left[r^2+b^2+\mu(b^2-r^2)\right]$$

Divide by b^2:

$$E\left(1-\frac{r^2}{b^2}\right) = \frac{r \cdot \Delta P}{\Delta r}\left[\frac{r^2}{b^2}+1+\mu\left(1-\frac{r^2}{b^2}\right)\right]$$

Let 'b' approach infinity, then:

$$E = \frac{r \cdot \Delta P}{\Delta r}(1+\mu) \tag{2-2}$$

Note:

The strain Δr occurs from r to $b=\infty$, not in a radial distance equal to r, as may be indicated by the form of the final equation. Essentially the same expression (2-2) may be found in references 2 and 4.

2. Contd.

A quantity 'Ep' is introduced which is defined as the pressuremeter (apparent) modulus of the in-place soil, in its proportional range, as determined by the pressuremeter tests.

$$Ep = \frac{\Delta P \cdot r}{\Delta r} \quad (2\text{-}3)$$

$$E = (1+\mu)E_p \quad (2\text{-}4)$$

Where: ΔP = the change in unit pressure in the proportional range. (P_1, P_2)

Δr = the radial deformation of the borehole in the proportional range = $f(\Delta V)$, the change in volume of the measuring cell.

r = the nominal radius of the borehole.

For small changes in radius:

$$\Delta V = (2r\pi\ell)\,\Delta r \quad (2\text{-}5)$$

$$\Delta r = \frac{\Delta V}{2r\pi\ell} \quad (2\text{-}6)$$

Here: ΔV = the amount of fluid added to the cell in the proportional range (V_1, V_2)

ℓ = length of measuring cell in probe

Substituting: equation (2-6) in (2-3)

$$Ep = 2r^2\pi\ell\frac{\Delta P}{\Delta V} \quad (2\text{-}7)$$

$$Ep = 2V_0\frac{\Delta P}{\Delta V} \quad (\underline{2}, \text{ p. } 6) \quad (2\text{-}8)$$

Where: V_0 = the calculated initial volume of the measuring cell with r = nominal radius of the borehole.

Constants used to calculate Ep

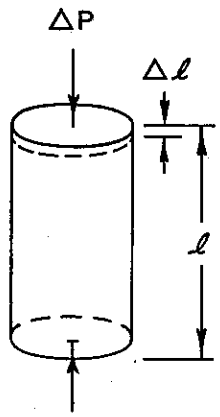
r = 4 cm

ℓ_1 = 20 cm (all except Squaw Cr., Test No. 4)

ℓ_2 = Squaw Cr., Test No. 4

3. FORMULAS FOR APPLICATION OF PRESSUREMETER MODULUS

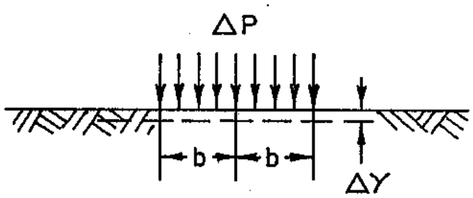
a. If the deformation $\Delta \ell$ of a freestanding specimen of length ℓ under a uniform unit load ΔP is desired:



$$\Delta \ell = \frac{\Delta P \cdot \ell}{E}$$

$$\Delta \ell = \frac{\Delta P \cdot \ell}{(1+\nu) E_p} \qquad (3\text{-}1)$$

b. If the average deformation of a uniformly loaded square area of side 2b on a simi-infinite body is desired:

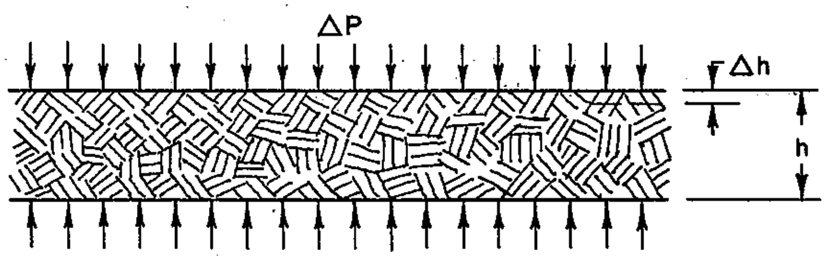


$$\Delta Y = \frac{1.90\ \Delta P \cdot b (1-\nu^2)}{E} \qquad [11, \text{p. } 291]$$

$$= \frac{1.90\ \Delta P \cdot b (1-\nu^2)}{1+\nu\ E_p}$$

$$\Delta Y = \frac{1.90\ \Delta P \cdot b\ (1-\nu)}{E_p} \qquad (3\text{-}2)$$

c. If the change in height Δh of a confined layer (no lateral strain permitted) is desired when subjected to a uniform load ΔP:



$$\Delta h = \frac{\Delta P \cdot h}{E}\left(1 - \frac{2\mu^2}{1-\mu}\right) \quad \text{(see below)} \tag{3-3}$$

$$\Delta h = \frac{\Delta P \cdot h}{(1+\mu)E_p}\left(\frac{1-\mu-2\mu^2}{1-\mu}\right)$$

$$\Delta h = \frac{\Delta P \cdot h}{E_p}\left(\frac{1-2\mu}{1-\mu}\right) \tag{3-4}$$

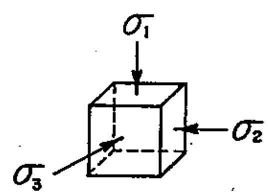
DERIVATION OF EQUATION (3-3)

The general equations of triaxial stress on a unit cube:

(1) $$\epsilon_1 = \frac{\sigma_1}{E} - \frac{\mu\sigma_2}{E} - \frac{\mu\sigma_3}{E}$$

(2) $$\epsilon_2 = \frac{\sigma_2}{E} - \frac{\mu\sigma_1}{E} - \frac{\mu\sigma_3}{E}$$

(3) $$\epsilon_3 = \frac{\sigma_3}{E} - \frac{\mu\sigma_1}{E} - \frac{\mu\sigma_2}{E}$$



Let $\sigma_2 = \sigma_3$: (1a) $\epsilon_1 E = \sigma_1 - 2\mu\,\sigma_3$

(3a) $\epsilon_3 E = \sigma_3(1-\mu) - \mu\sigma_1$

In the case of no lateral strain, $\epsilon_2 = \epsilon_3 = 0$:

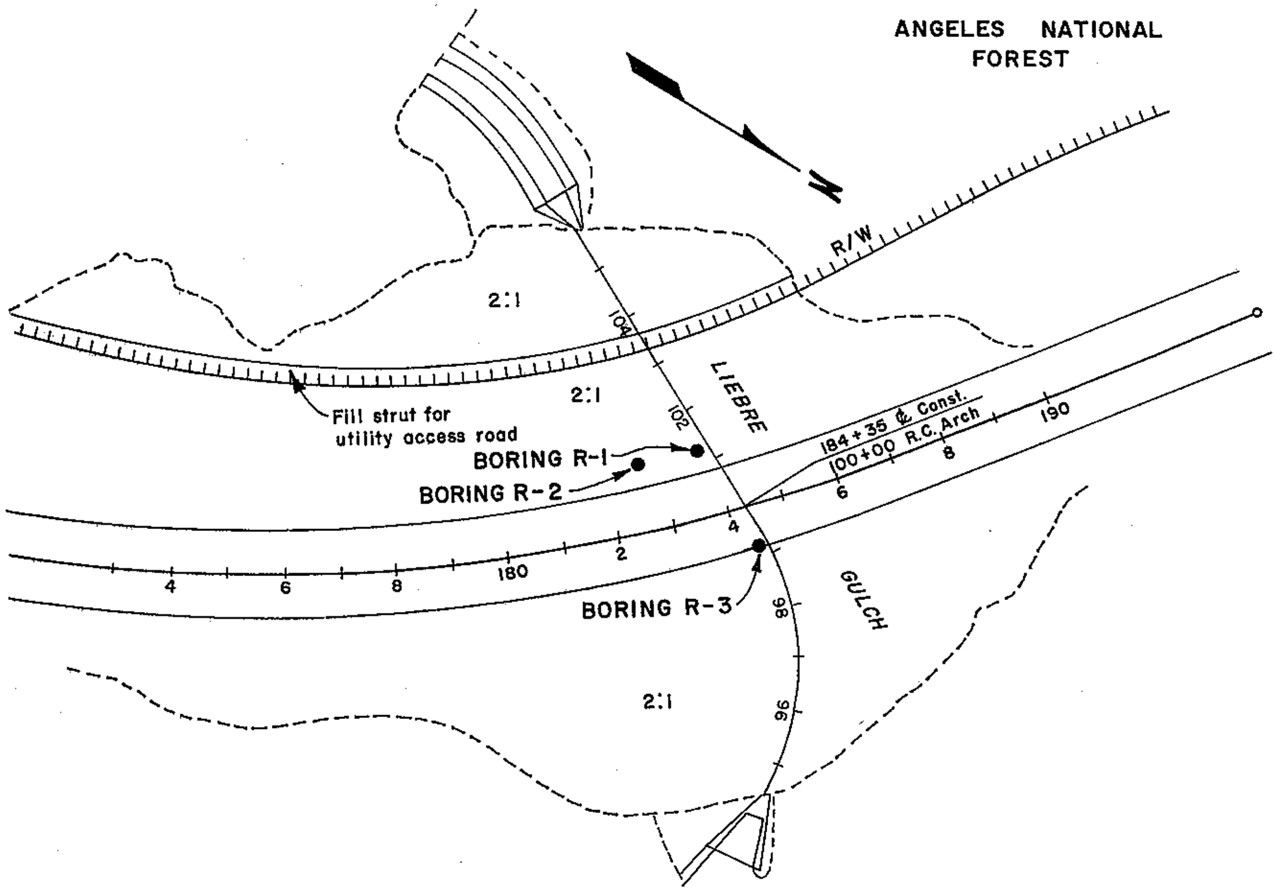
(3b) $$\sigma_3 = \frac{\mu}{1-\mu}\sigma_1 \tag{3-5}$$

Substitute (3b) in (1a):

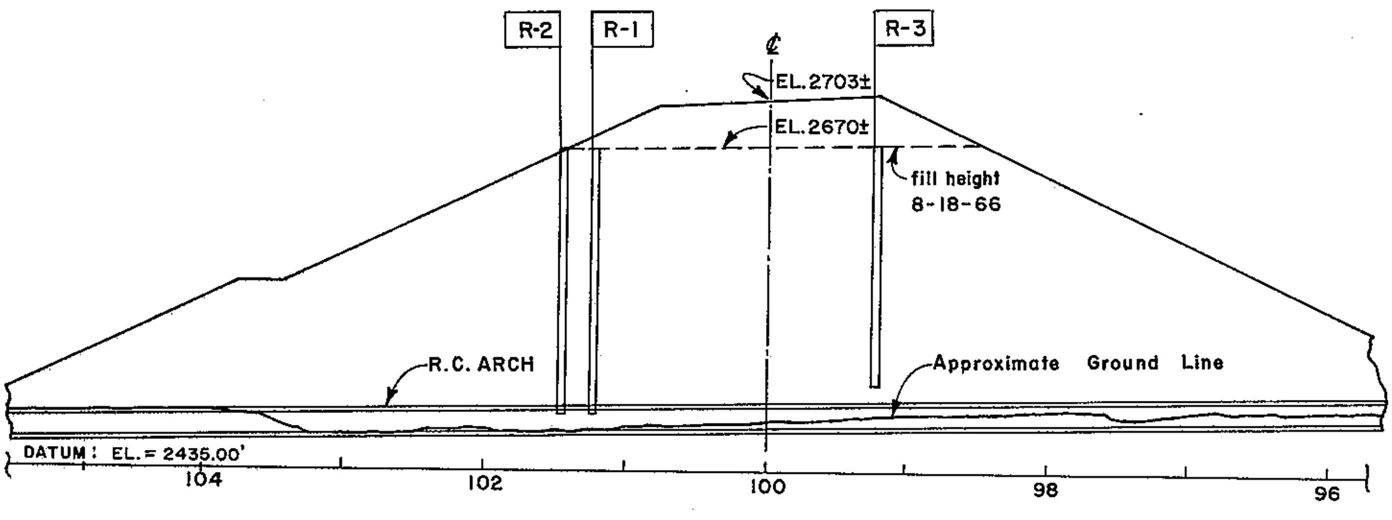
$$\epsilon_1 = \frac{\sigma_1}{E}\left(1 - \frac{2\mu^2}{1-\mu}\right)$$

From which: $$\Delta h = \frac{\Delta P \cdot h}{E}\left(1 - \frac{2\mu^2}{1-\mu}\right) \tag{3-3}$$

Figure C-1



07-LA-5
P.M. R70.9/R74.5



STA. 184 + 35 ℄ CONSTRUCTION
SKEW 14° LEFT

LIEBRE GULCH EMBANKMENT
PRESSUREMETER

Figure C-2

LIEBRE GULCH EMBANKMENT - PRESSUREMETER

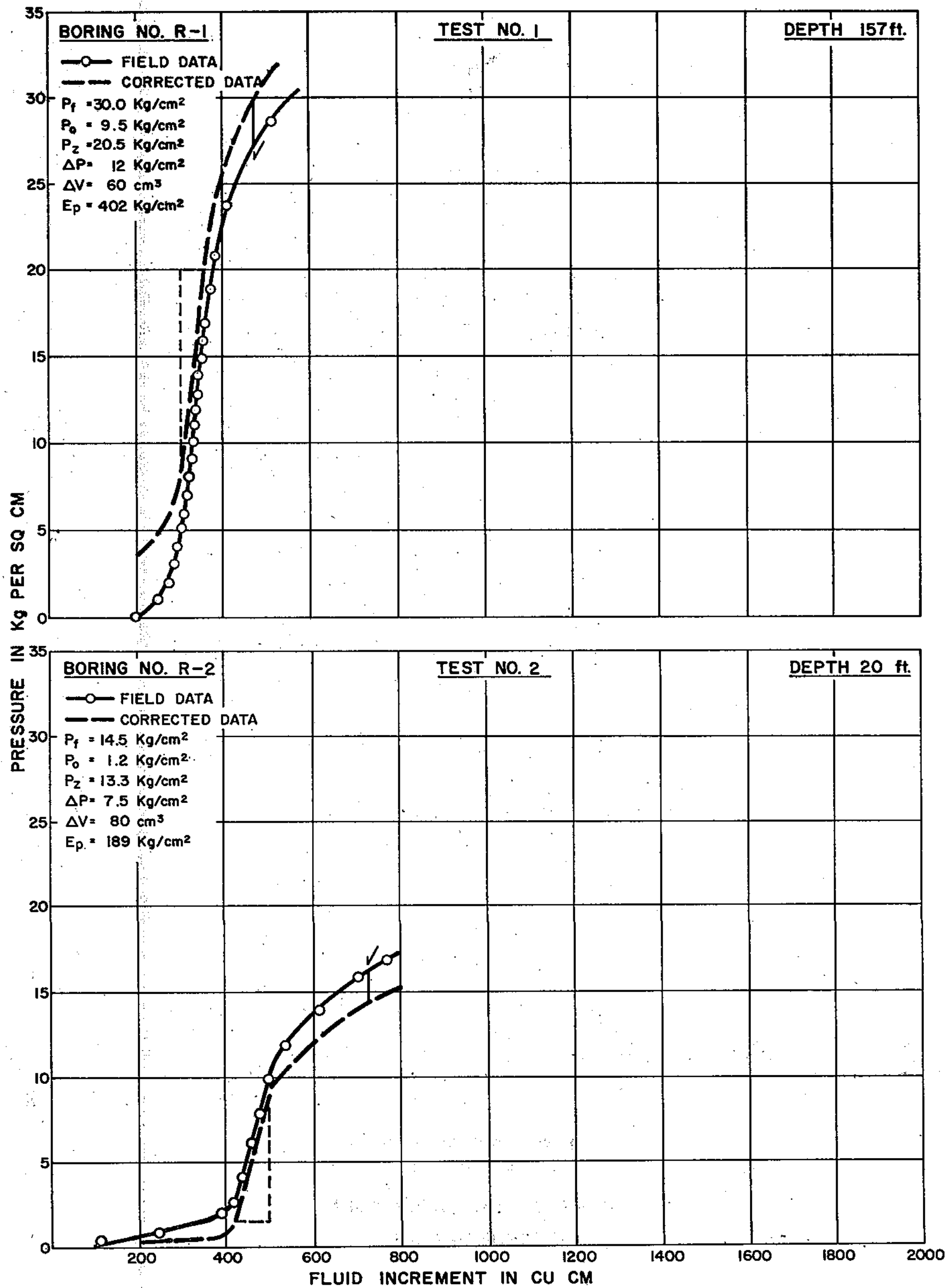


Figure C-3

LIEBRE GULCH EMBANKMENT - PRESSUREMETER

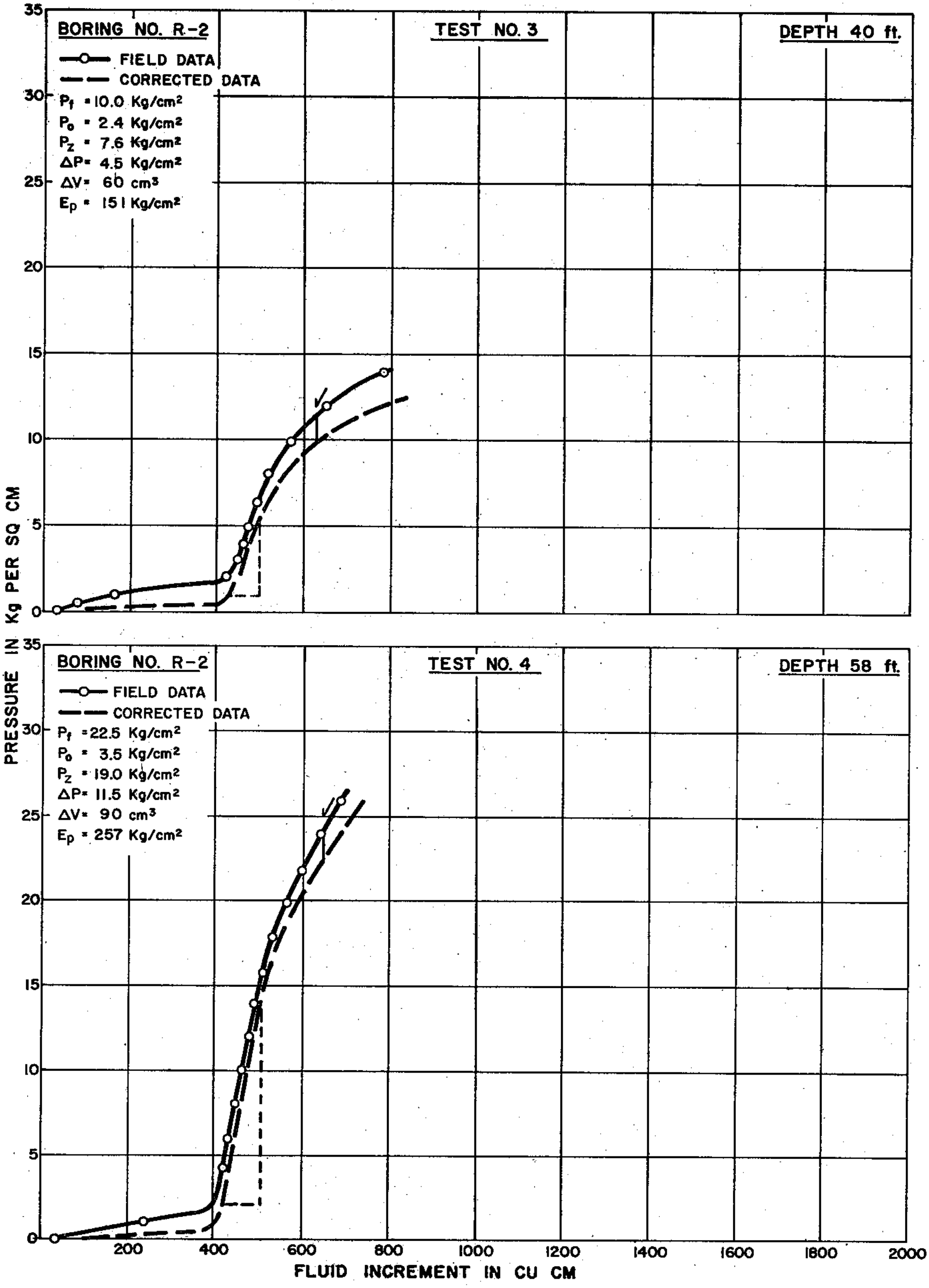


Figure C-4

LIEBRE GULCH EMBANKMENT - PRESSUREMETER

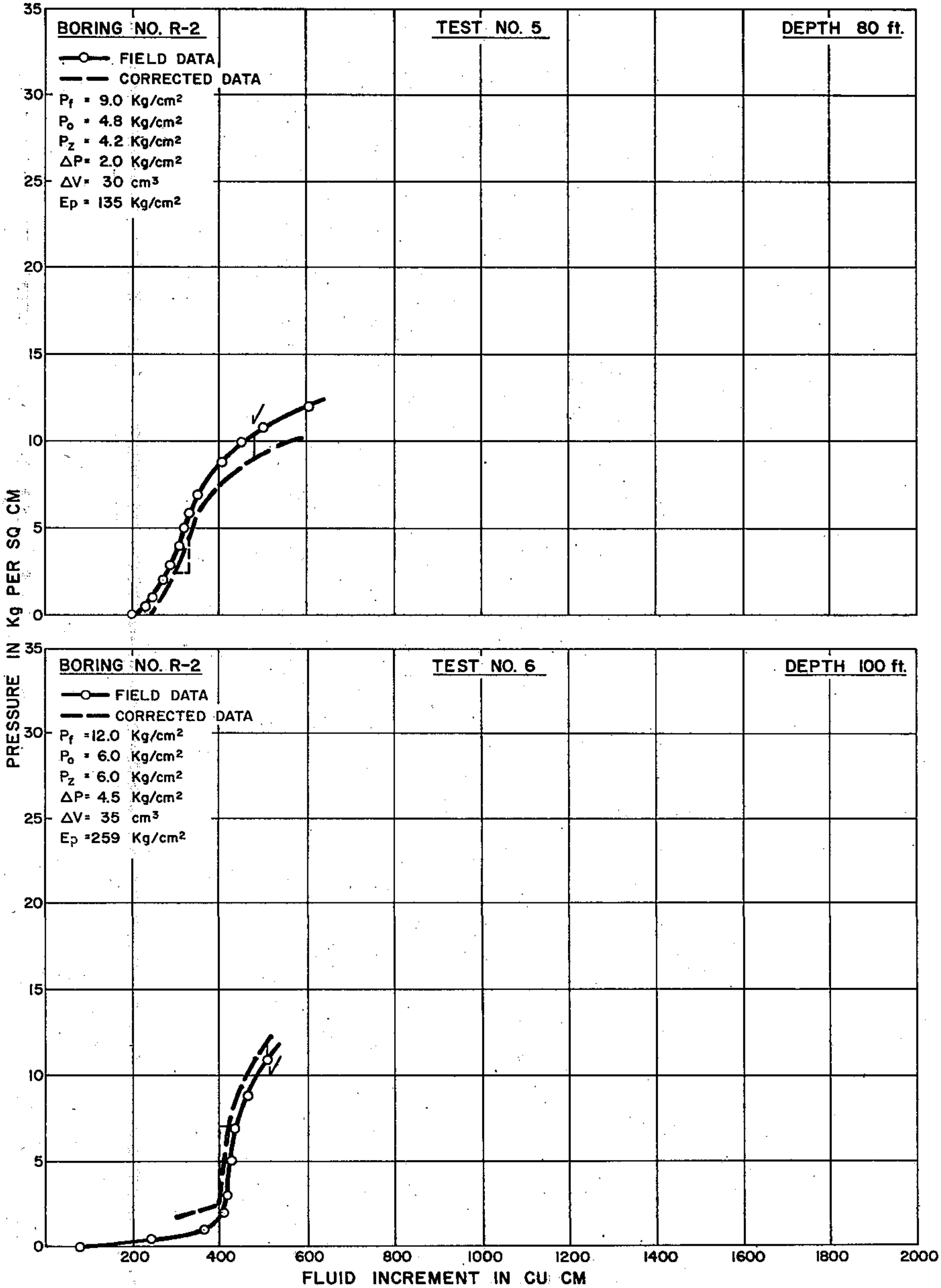


Figure C-5

LIEBRE GULCH EMBANKMENT - PRESSUREMETER

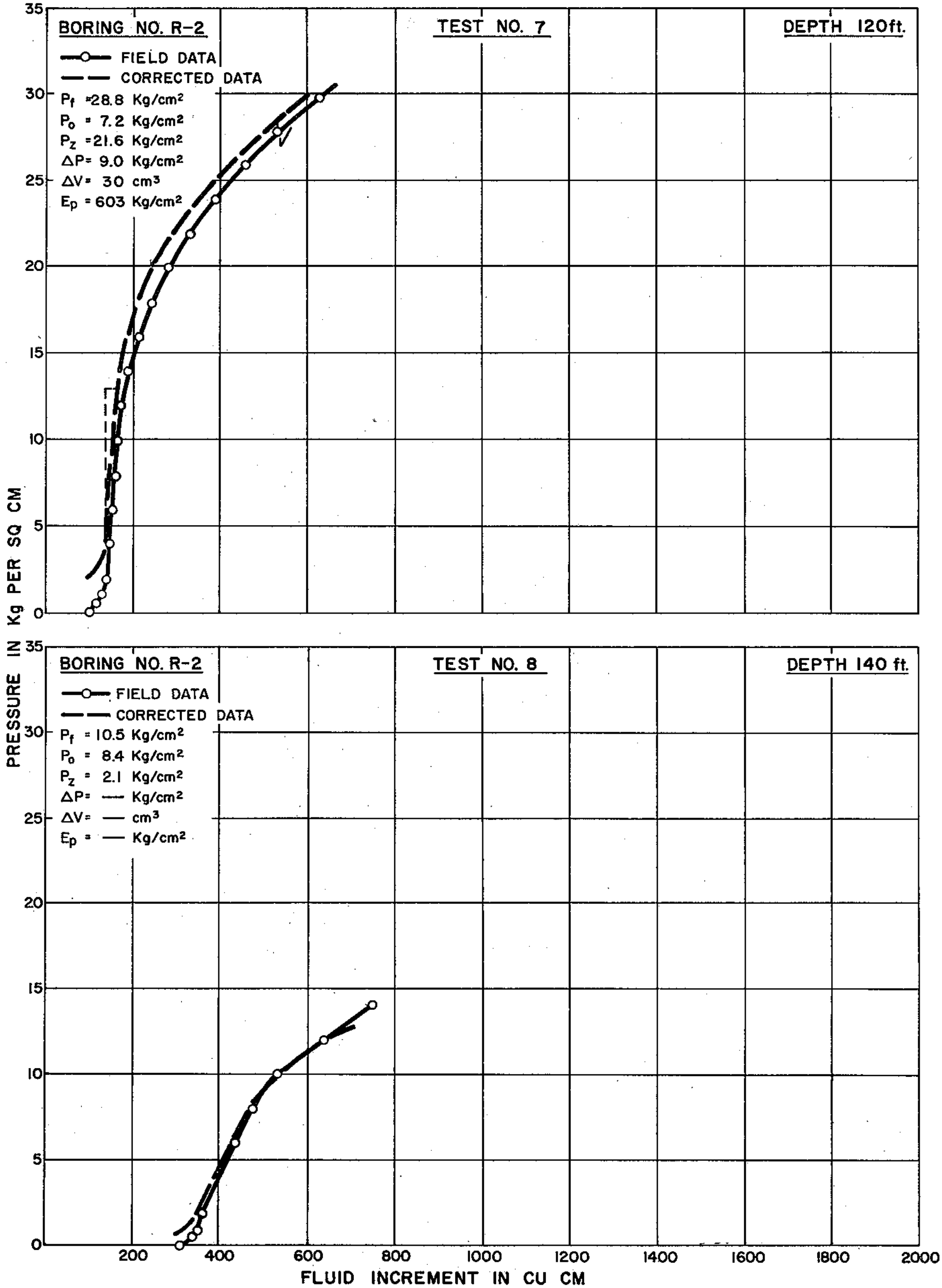


Figure C-6

LIEBRE GULCH EMBANKMENT - PRESSUREMETER

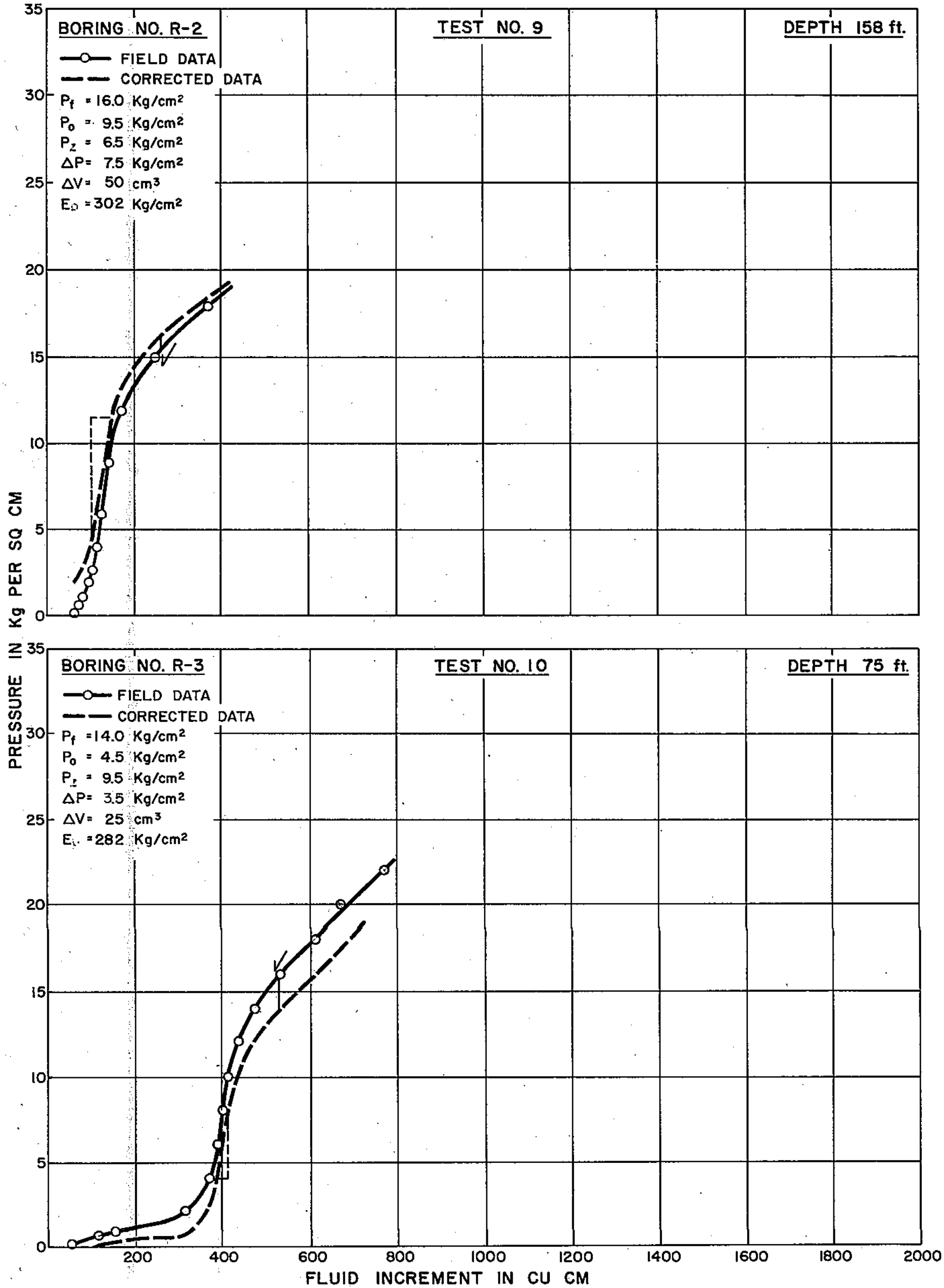
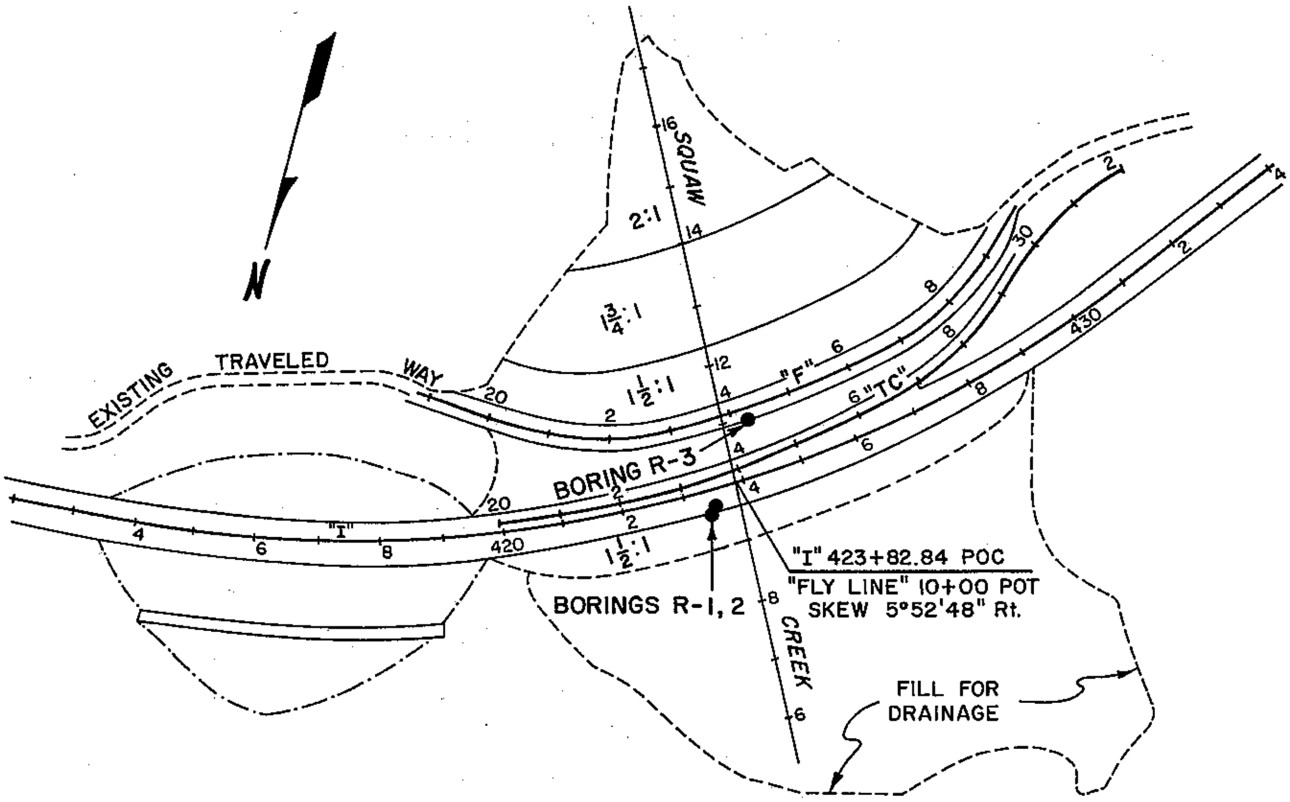
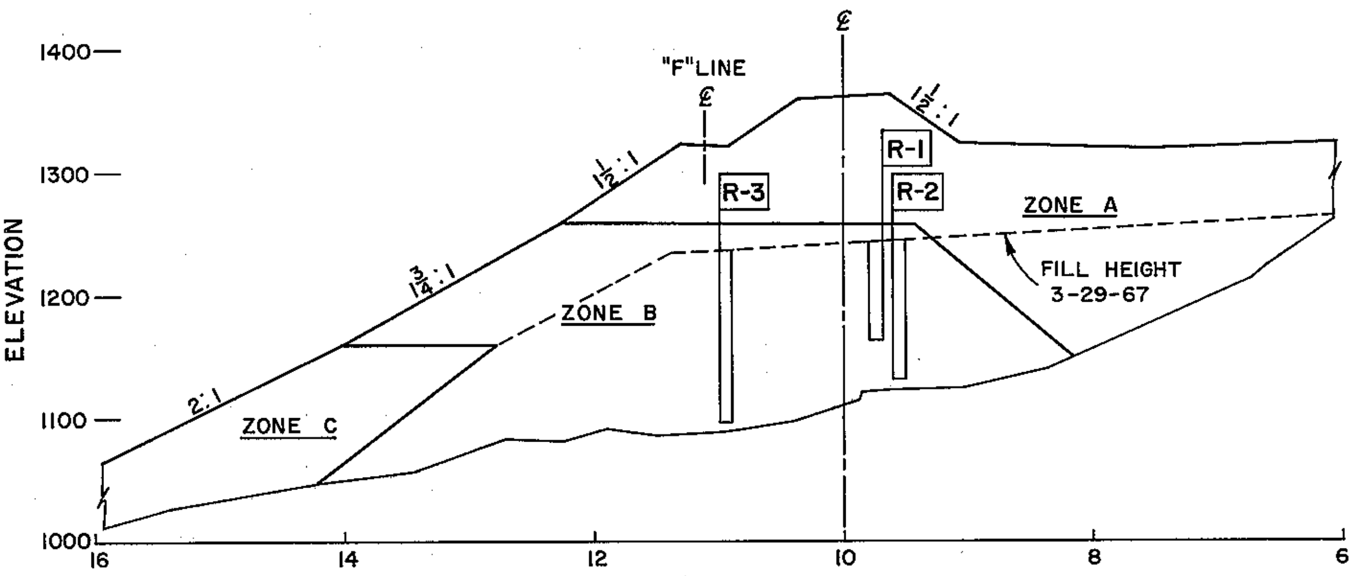


Figure C-7



01-MEN-101
P.M. 83.8/88.3



STA. "I" 423+82.84 POC
SKEW 5° 52' 48" RIGHT

SQUAW CREEK EMBANKMENT
PRESSUREMETER

Figure C-8

SQUAW CREEK EMBANKMENT - PRESSUREMETER

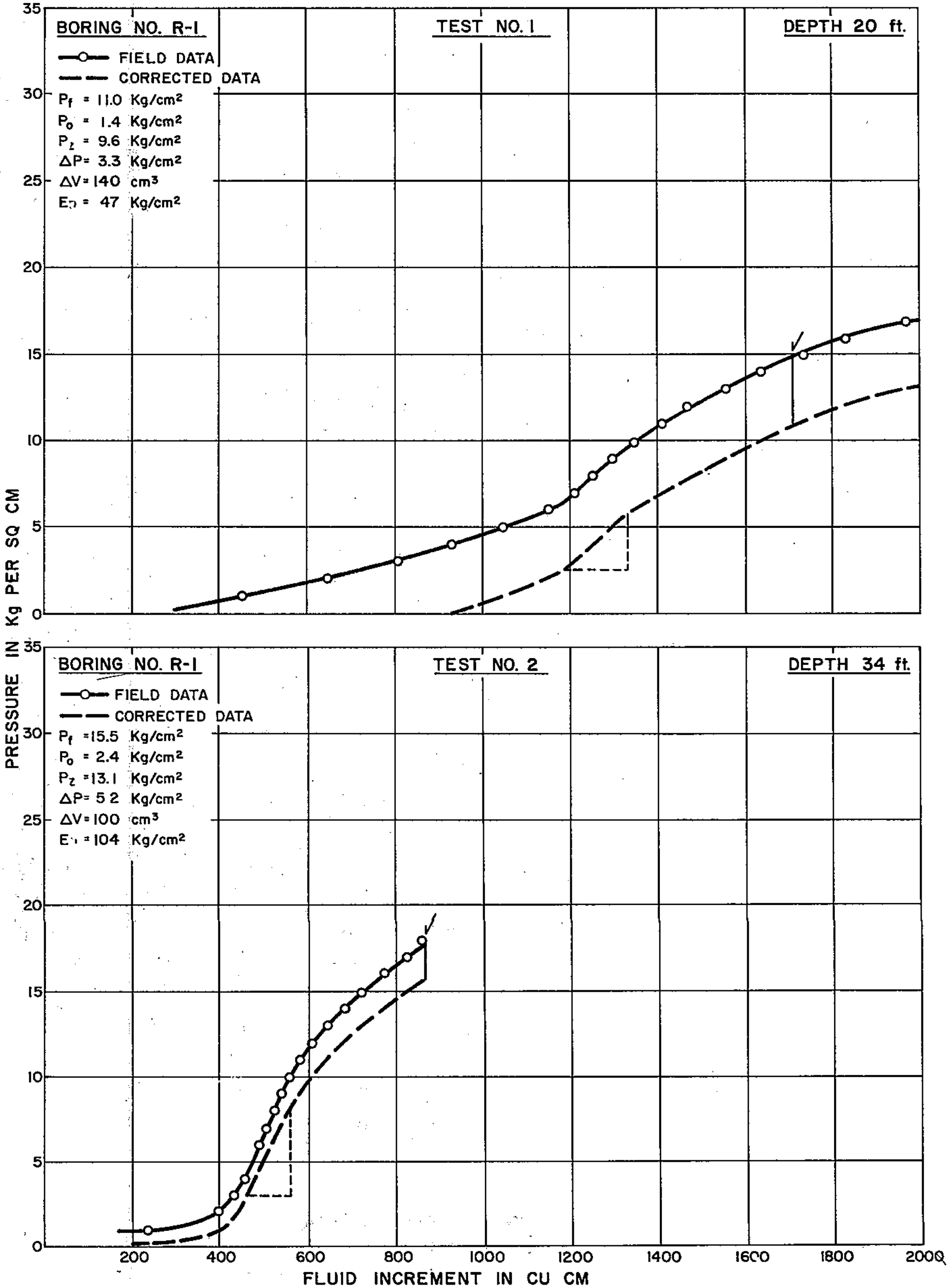


Figure C-9

SQUAW CREEK EMBANKMENT - PRESSUREMETER

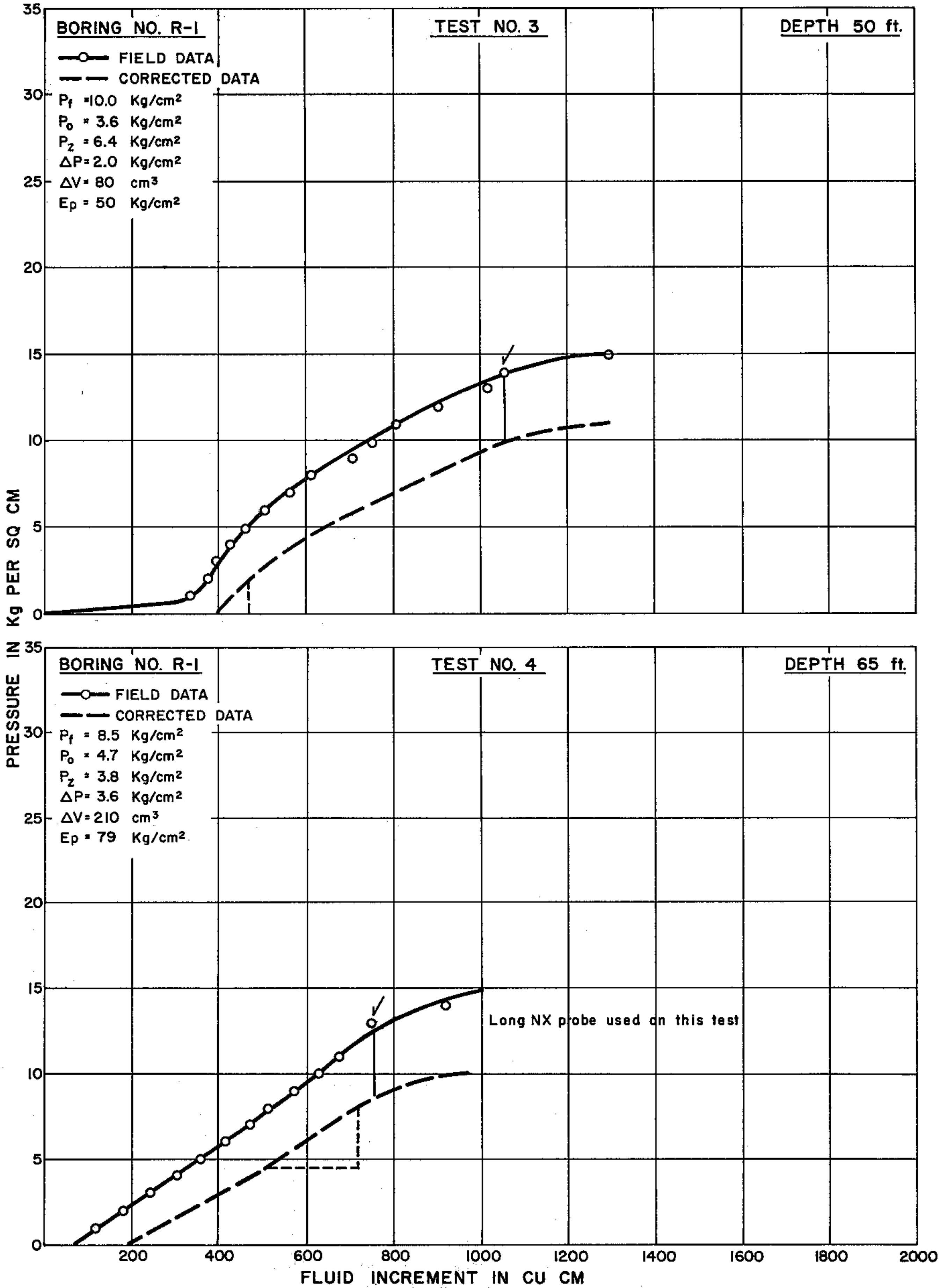


Figure C-10

SQUAW CREEK EMBANKMENT - PRESSUREMETER

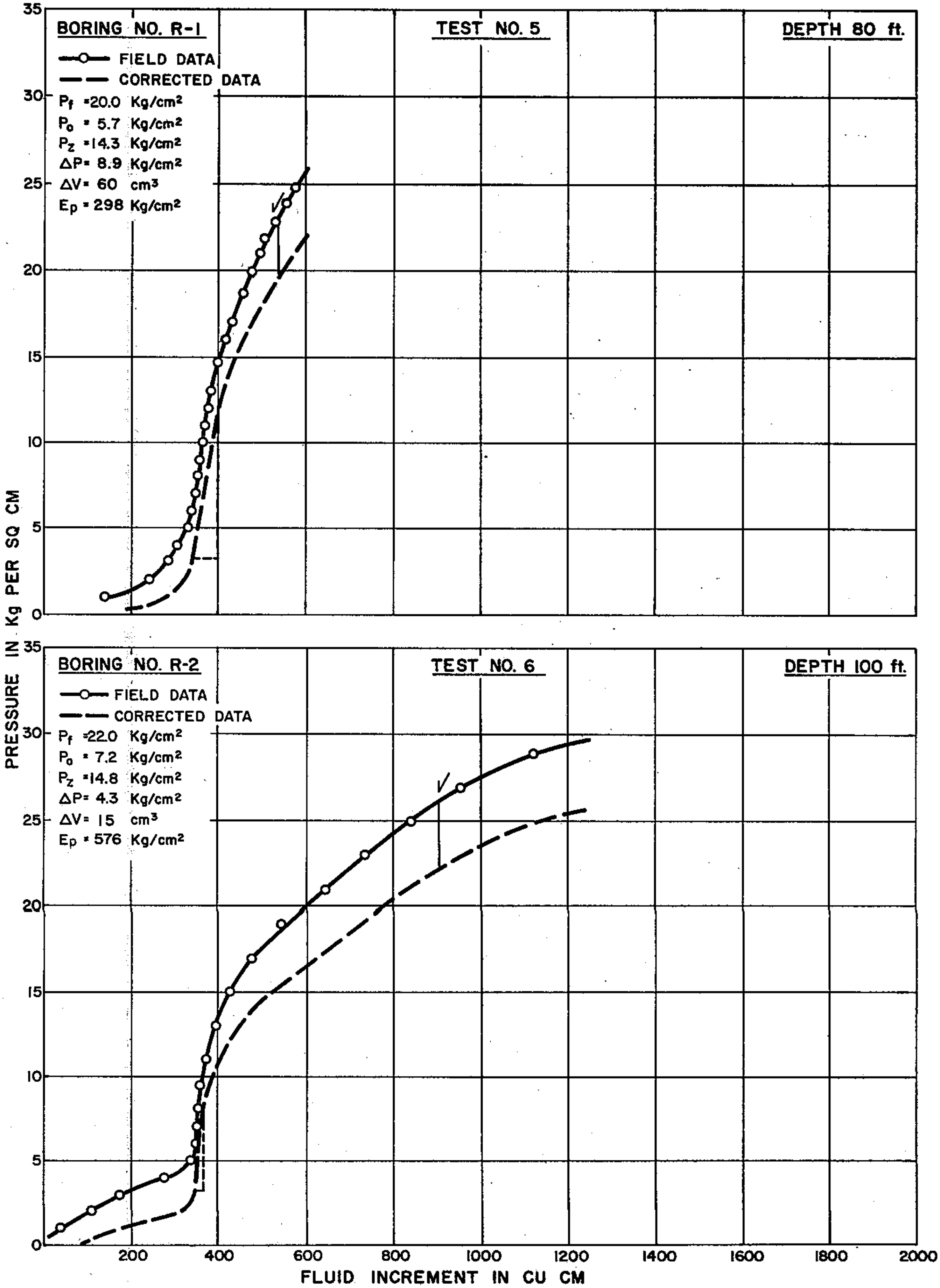


Figure C-11

SQUAW CREEK EMBANKMENT - PRESSUREMETER

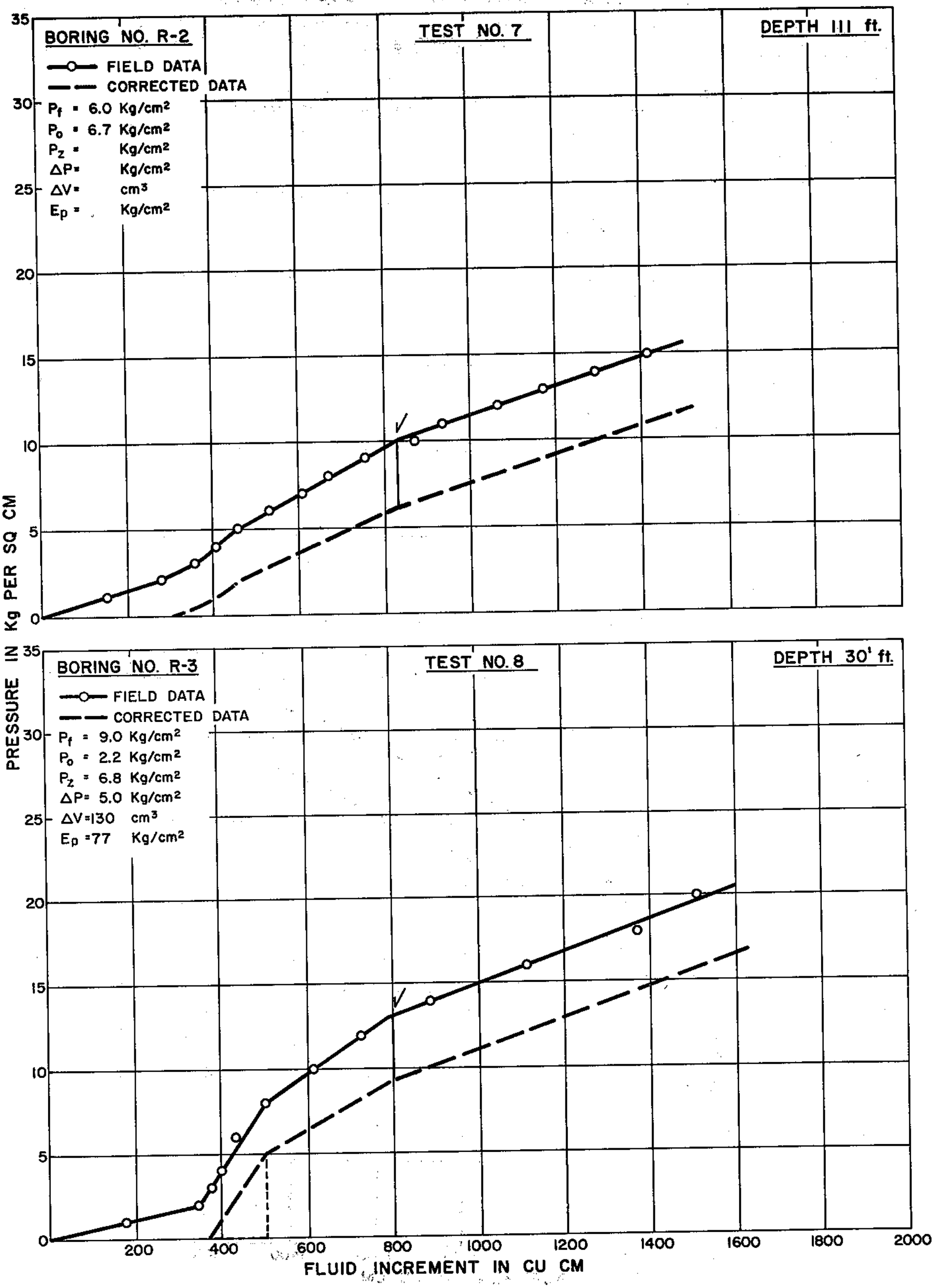


Figure C-12

SQUAW CREEK EMBANKMENT - PRESSUREMETER

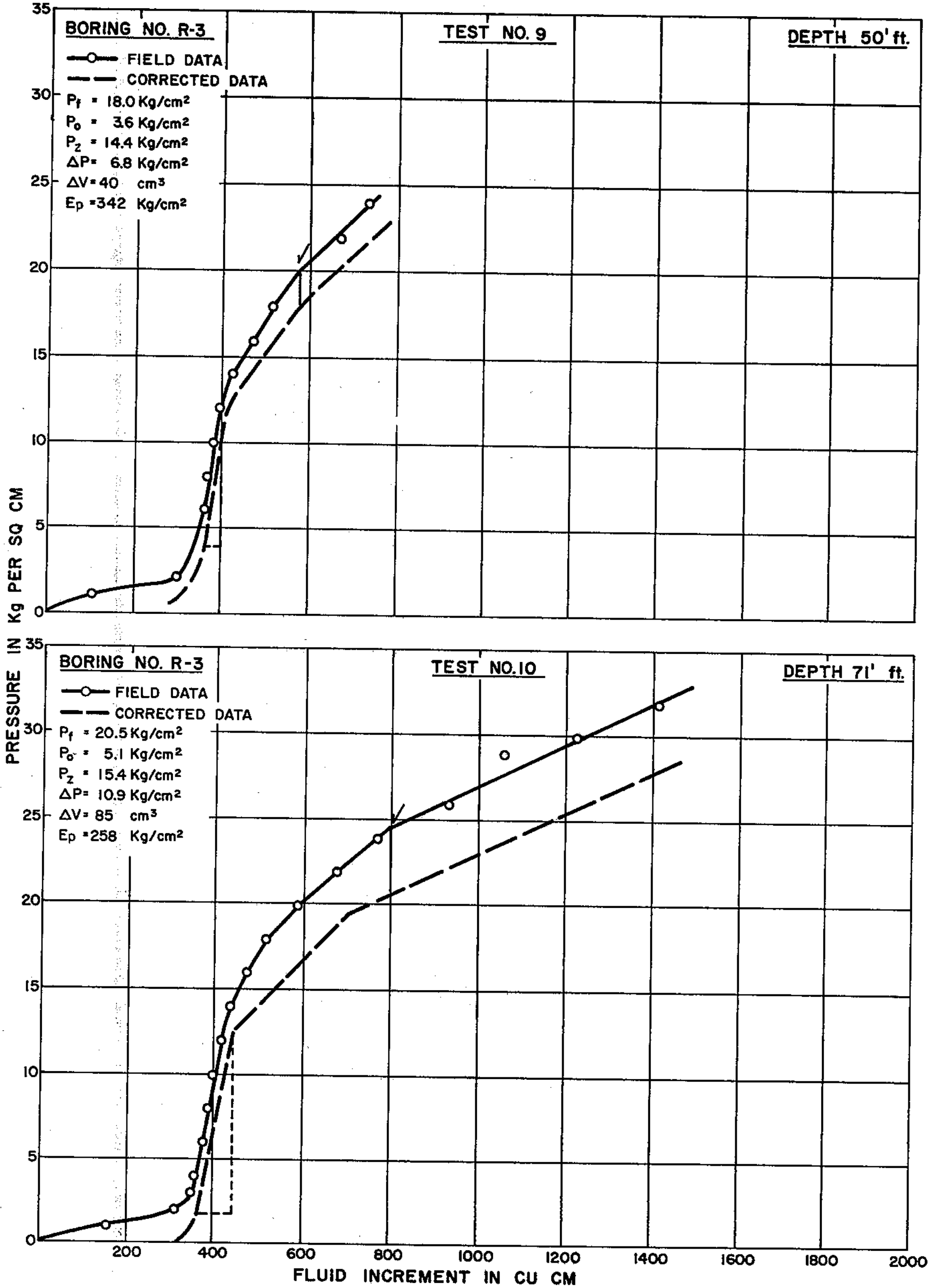


Figure C-13

SQUAW CREEK EMBANKMENT - PRESSUREMETER

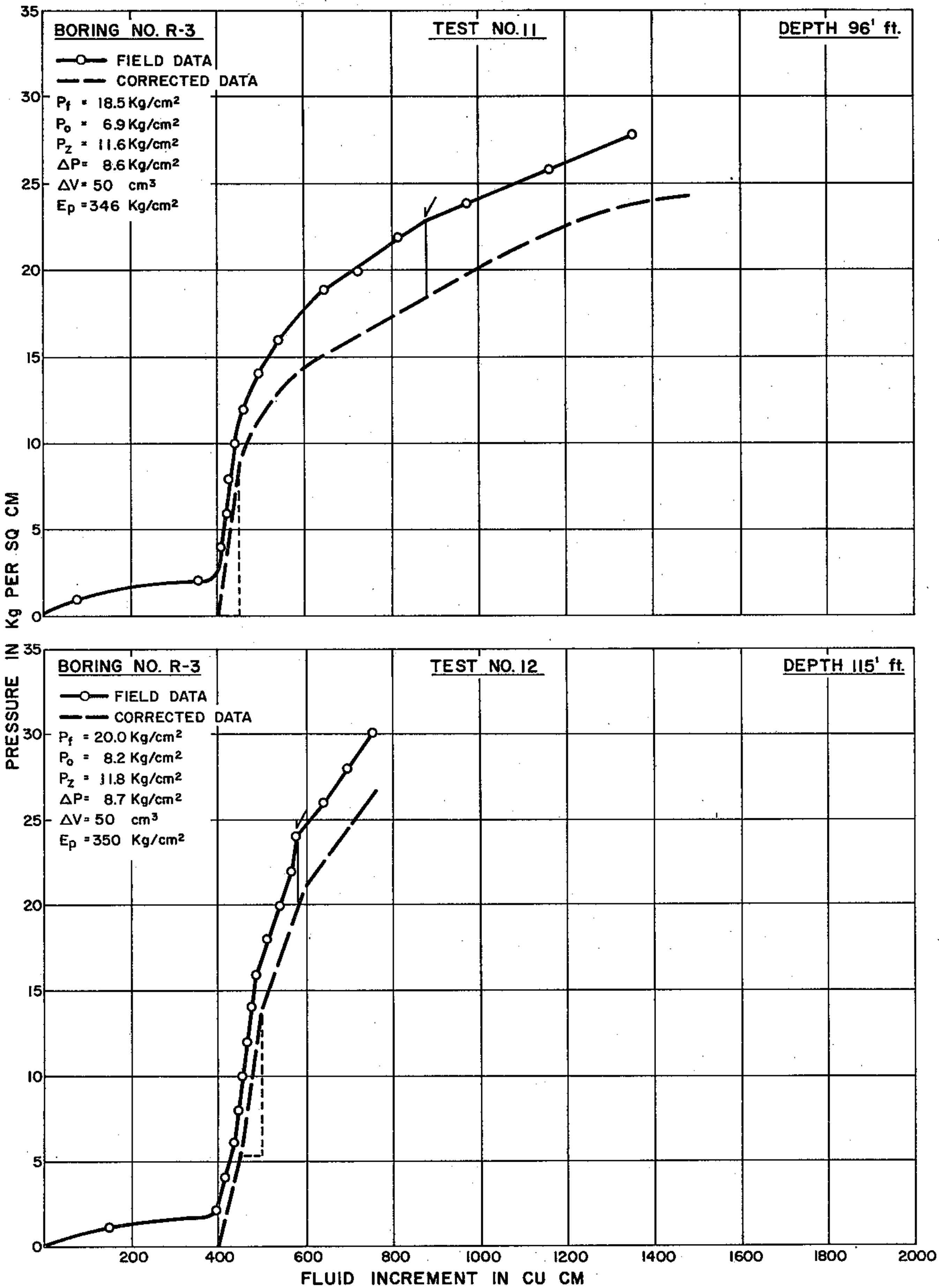


Figure C-14

SQUAW CREEK EMBANKMENT - PRESSUREMETER

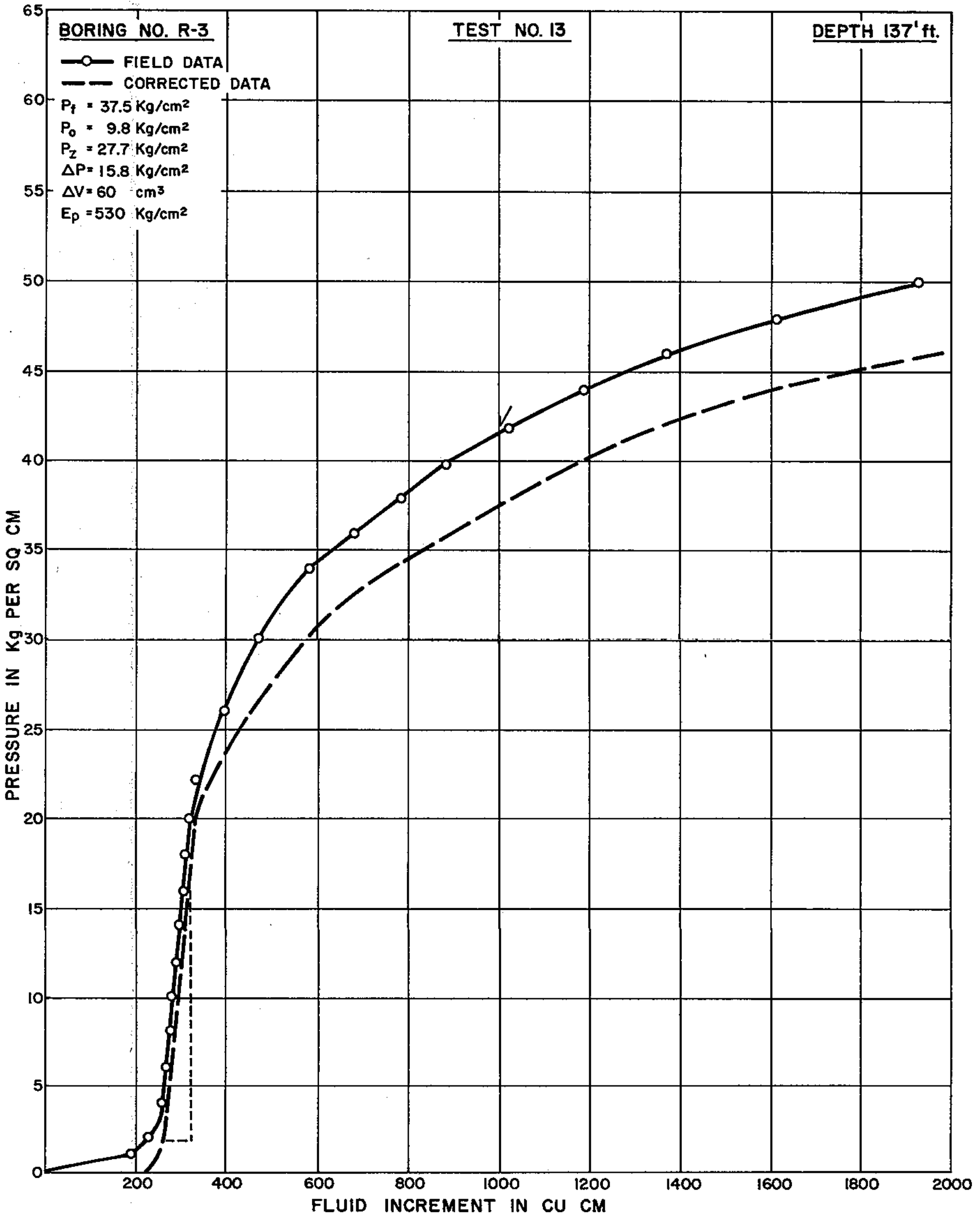
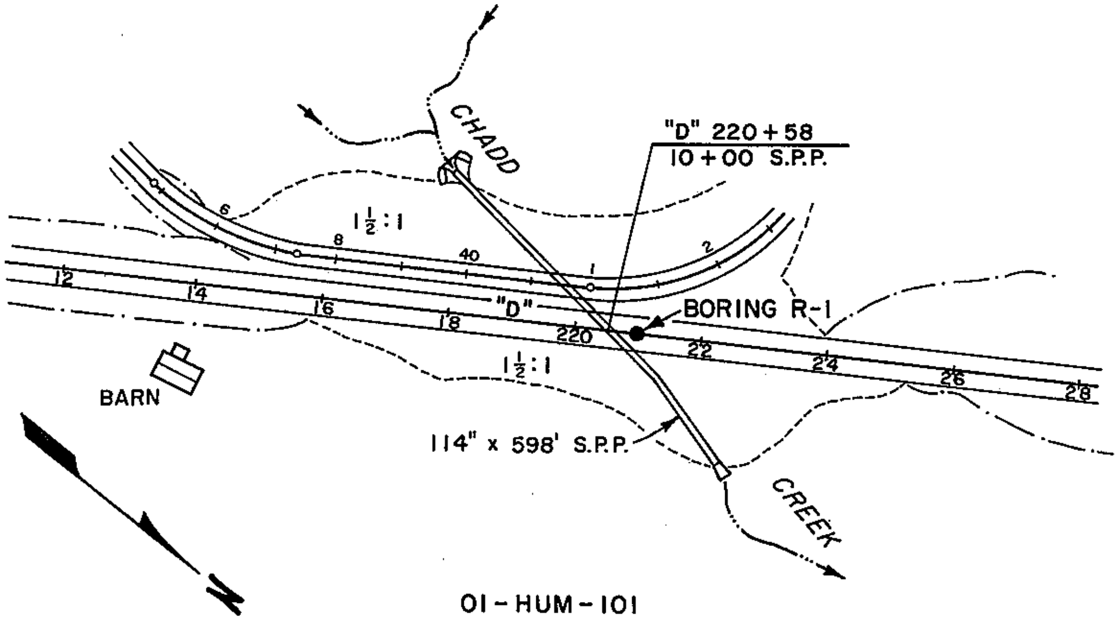
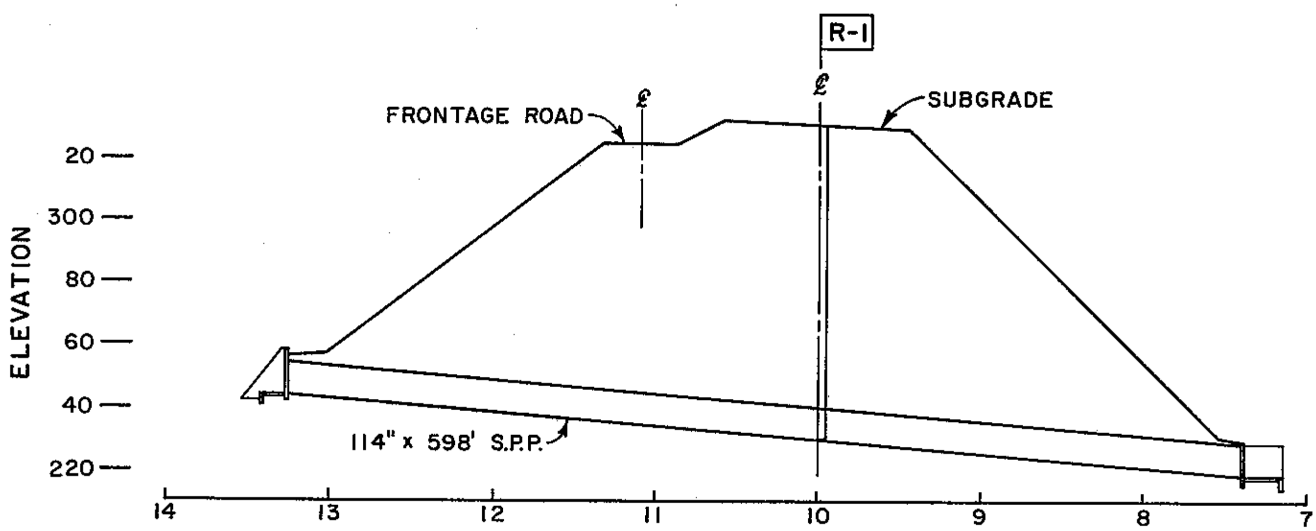


Figure C-15



01 - HUM - 101
P.M. R39.2 / R46.9



STA. "D" 220 + 58
SKEW 50° LEFT

CHADD CREEK EMBANKMENT
PRESSUREMETER

Figure C-16

CHADD CREEK EMBANKMENT - PRESSUREMETER

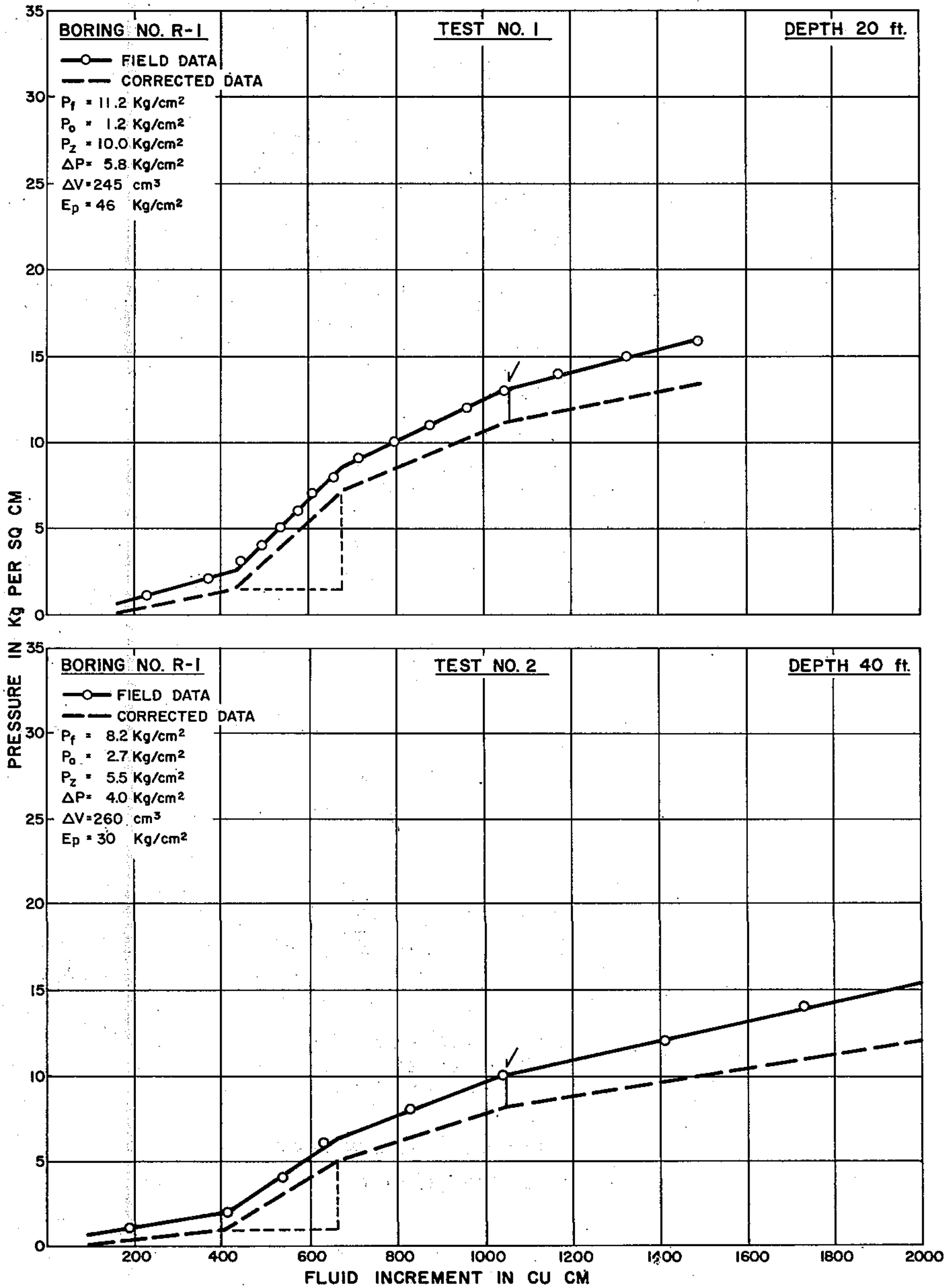


Figure C-17

CHADD CREEK EMBANKMENT - PRESSUREMETER

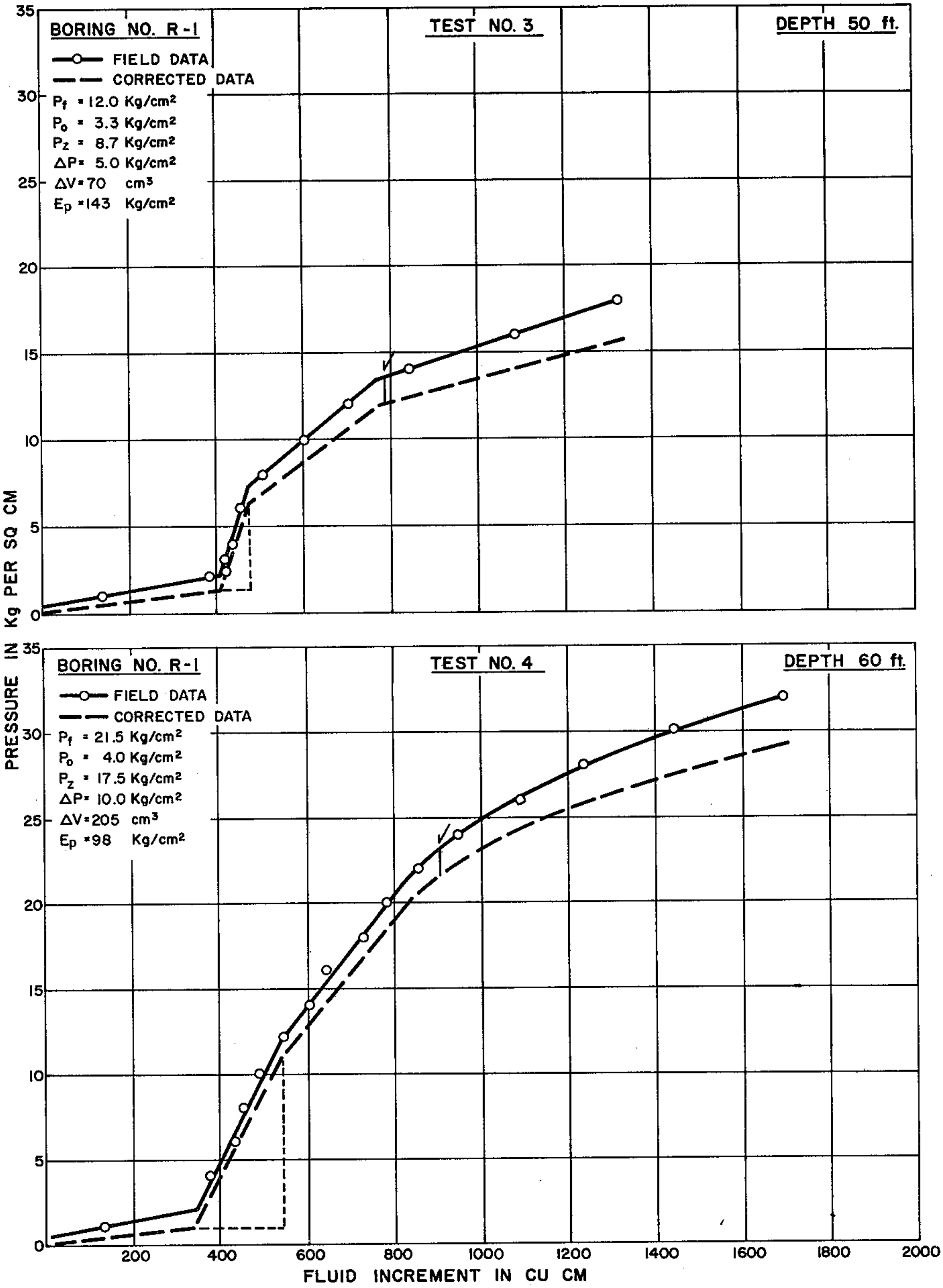


Figure C-18

CHADD CREEK EMBANKMENT - PRESSUREMETER

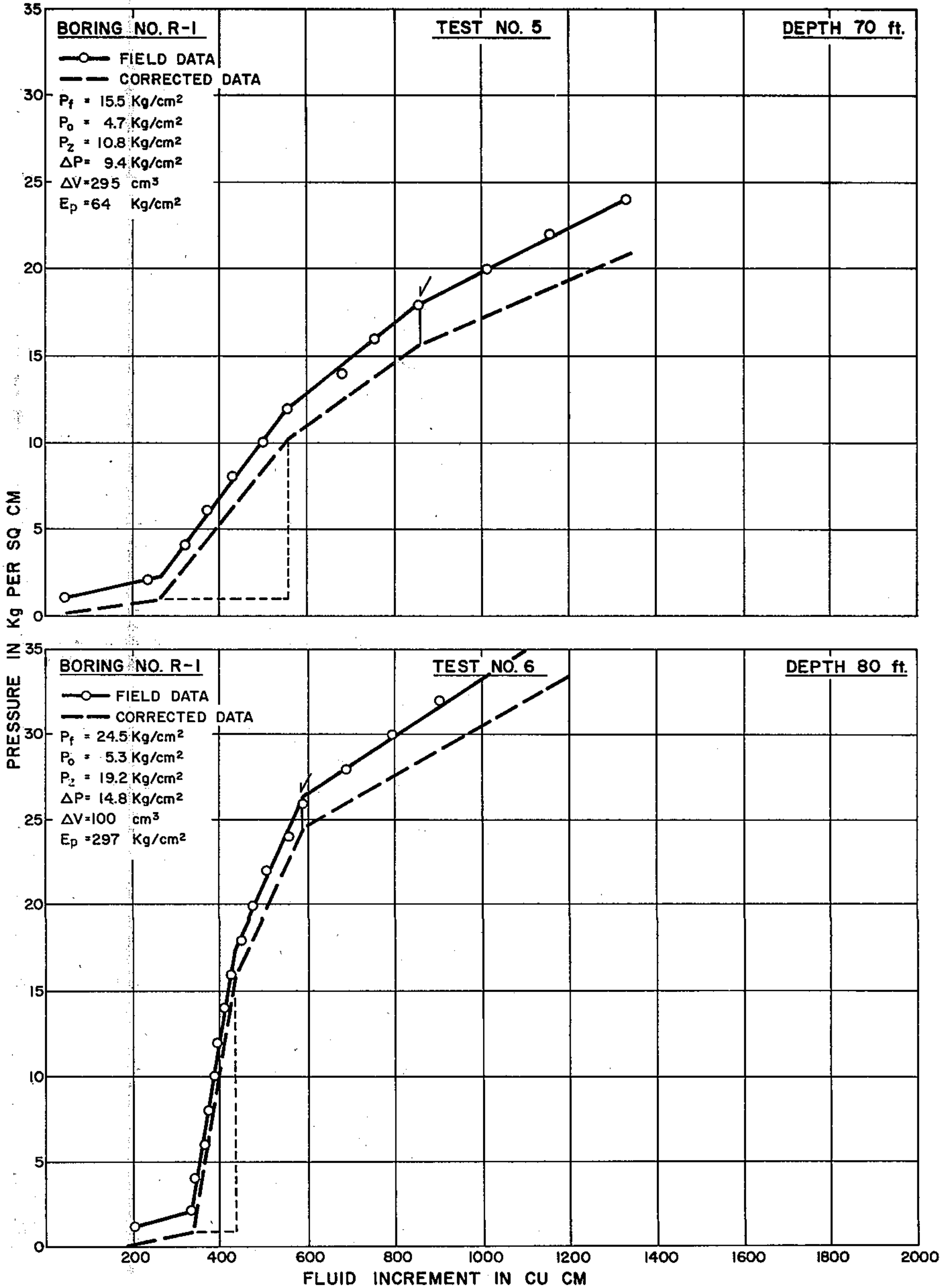


Figure C-19

CHADD CREEK EMBANKMENT - PRESSUREMETER

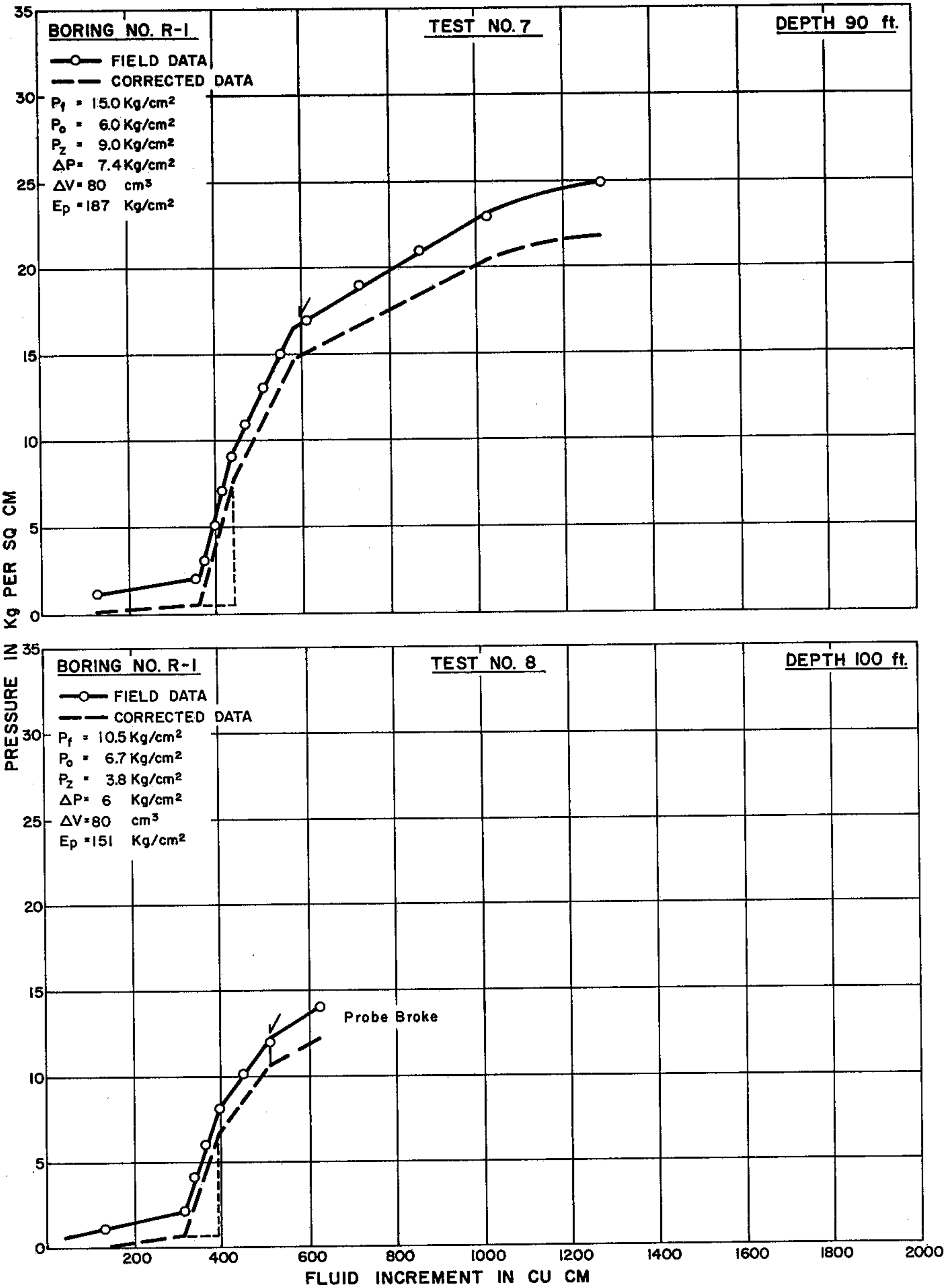


Figure C-20

CHADD CREEK EMBANKMENT - PRESSUREMETER