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A STUDY OF THE SHEAR CHARACTERISTICS OF THE SOILS OF THE PIEDMONT REGION DETERMINED BY ROTATING VANES

A THESIS

Presented to

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Master of Science in Civil Engineering

By

Edmond Trowbridge Miller

March, 1957

A STUDY OF THE SHEAR CHARACTERISTICS
OF THE SOILS OF THE PIEDMONT REGION
DETERMINED BY ROTATING VANES

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Date Approved by Chairman: March 16, 1957

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ABSTRACT

The rotating vane was developed to provide a simple and accurate device for determining the shear characteristics of soil. Its initial tests, made in sedimentary glacial clays, bear out its usefulness in these soils.

The equipment used in the series of experiments described in this thesis consists of two primary parts, the vanes, and the torque head. The vanes themselves are made of two steel plates attached to each other in the form of a cross and then welded to a central shaft. The torque head or the device used to rotate the vanes, was fabricated from steel and aluminum for ease of construction and light weight. Torque was applied to the vane through soil drill rods by an aluminum disc, a winch, and a cable. The force applied to the disc was measured by a proving ring. The torque required to produce a unit rotation was recorded in the tests and the soil stress developed by the maximum torque recorded in each test was considered the shear strength of the soil.

Standard laboratory tests of the soil at the location of each vane test provided the laboratory control data needed for an evaluation of the vane shear apparatus. These tests included triaxial shear tests, unconfined compression tests, and consolidation tests. The triaxial shear tests and unconfined compression tests provided information concerning the shear characteristics of the soils encountered.

The consolidation tests were performed to determine the preconsolidation load previously imposed on the soil.

When the soil shear strength was determined from the Mohr envelope by considering the preconsolidation load as the confining pressure, a definite relationship was found between the laboratory tests and the vane shear tests. The soil strength as determined by the vanes was approximately 1.5 times that determined by the laboratory tests. The usefulness of the device, however, is limited because supplementary information, which can be obtained only from laboratory testing, is required. The vane shear apparatus would be a valuable tool in stability investigations or in similar investigations where the ultimate strength of the soil in its present condition is of primary concern, but its usefulness in routine soil investigation for foundation design would appear to be small in the soils of the Piedmont Region.

Approved:

Ben F Jaurn

CHAPTER I

INTRODUCTION

In the year 1948, Lyman Carlson in Sweden(1) and A. W. Skempton in England(2) reported on experiments with a new device for determining the shear strength of clay soils. It appears that the idea was developed simultaneously in the two countries. In both of these investigations, a bladed vane was forced into a clay and rotated to determine the shear strength of the soil. Both investigators show that the shear strengths thus obtained are somewhat greater than those obtained through the conventional laboratory tests performed on "undisturbed" samples.

This discovery by Skempton and Carlson stimulated interest in this unconventional method of shear strength determination. The Messrs. Vey and Schlesinger(3) reported on similar tests in the Chicago area of the United States in the year 1949. This series of tests indicated that the shear strengths determined by the rotating vane very closely paralleled those determined through conventional laboratory methods. These tests were conducted closer to the surface than those mentioned previously, and thus were subjected to a lesser overburden pressure. The tests made by Skempton and Carlson show that the increase over the conventional tests as shown by the vane is less in tests made at the shallower depths, or in the tests with smaller overburden pressures.

In 1955, Mr. Hamilton Gray(4) reported on tests using the rotating vane which also indicate a marked increase in the strength over that determined through laboratory testing. His results also indicate the increase in the differential with increasing depth that was noted in earlier tests.

Even more recently (in 1955) tests of this nature were reported by Mr. Carl W. Fenske(5). These tests again bear out the validity of the previous test results.

It has been noted that the strengths indicated by the vane shear test closely approximate the estimates of strength made in post mortems of certain actual landslides. It would then appear that the vane shear testing device gives a better index to the soil strength than any device in use today. The increase in indicated strength is due in part to the absence of disturbances which are unavoidable in sampling for standard laboratory testing. Obviously, there is a certain amount of disturbance involved in the trimming operation during the preparation of the shear specimen. Not so obvious is the disturbance involved when the stresses applied to the soil in its natural location are released as the soil is removed from the ground. Both of these disturbances tend to weaken the specimen tested.

All writers on the subject agree that the value of the results of the vane shear test cannot be accurately determined without extensive research and much more substantiating evidence. One of the major reasons for this is that there is at this time no way to express the amount of disturbance in a soil sample mathematically. Therefore, it is only possible to relate two techniques of testing to each other, because the norm, or the true strength of the soil cannot be determined. However, as it has been said, the strength of the soil is probably not less than the values determined by the vane shear tests.

All tests made in the previously mentioned experimental series were performed in sedimentary, glacial clays. An attempt to utilize a rotating vane for the shear testing of residual soils was made by William C. Hill(6) of the Oregon State Highway Department. By using statistical methods, Mr. Hill arrived at an equation relating the shear strengths obtained through the use of the vane to those obtained through standard laboratory procedures. While the equation may be statistically correct, the points which were plotted on a graph of these strength values are so widely scattered that they fail to visually support the conclusions reached in the research. While further testing in the ranges not included in the research of the Oregon State Highway Department may bear out the validity of the equation, it is not apparent in the data provided.

These articles and their findings, as well as the challenge they presented, led to the selection of this thesis topic. The Atlanta area is geologically located in the Piedmont Plateau. The soils found in the Piedmont Region are primarily residual, formed by the in-place weathering of the underlying crystalline rock. This thesis further explores the possibility of using the rotating vane as a means of determining the shear strength of soils with the properties of those found in the Piedmont Region.

CHAPTER II

TESTING EQUIPMENT

The equipment used in the performance of the actual vane shear tests for this thesis was designed using features of several of the apparatuses used in previous investigations.

The vanes themselves were formed from 1/8 inch steel plates. Two blades were cut to the proper shape and welded together at right angles to each other along their centerlines. The blades were then welded to a central shaft. At the upper end of this shaft was welded a standard size "E" soil drill rod coupling (Fig. 1 and Fig. 3).

The width of the smaller vane was 1.52 inches and its length was 2.98 inches. The bottom end of the vane was tapered to a point on a slope of 1.25 to 1 with respect to a line perpendicular to the axis of the vane and the top end on a slope of 2 to 1. The tapered portions of the large vane were cut to the same slopes, but the width and length were increased to 2.50 inches and 5.11 inches, respectively. The shaft attached to the upper end of the small vane was 1/2 inch in diameter, and the shaft attached to the large vane was 3/4 inch in diameter.

The vane to be used in the test was attached to the appropriate length of size "E" soil drill rod through the coupling at the end of its own shaft. The top end of the size "E" rod was attached to a 7/8 inch diameter steel shaft. A metal key was used to attach this shaft to an aluminum disc of one foot radius, through which the torque was applied to the sample.

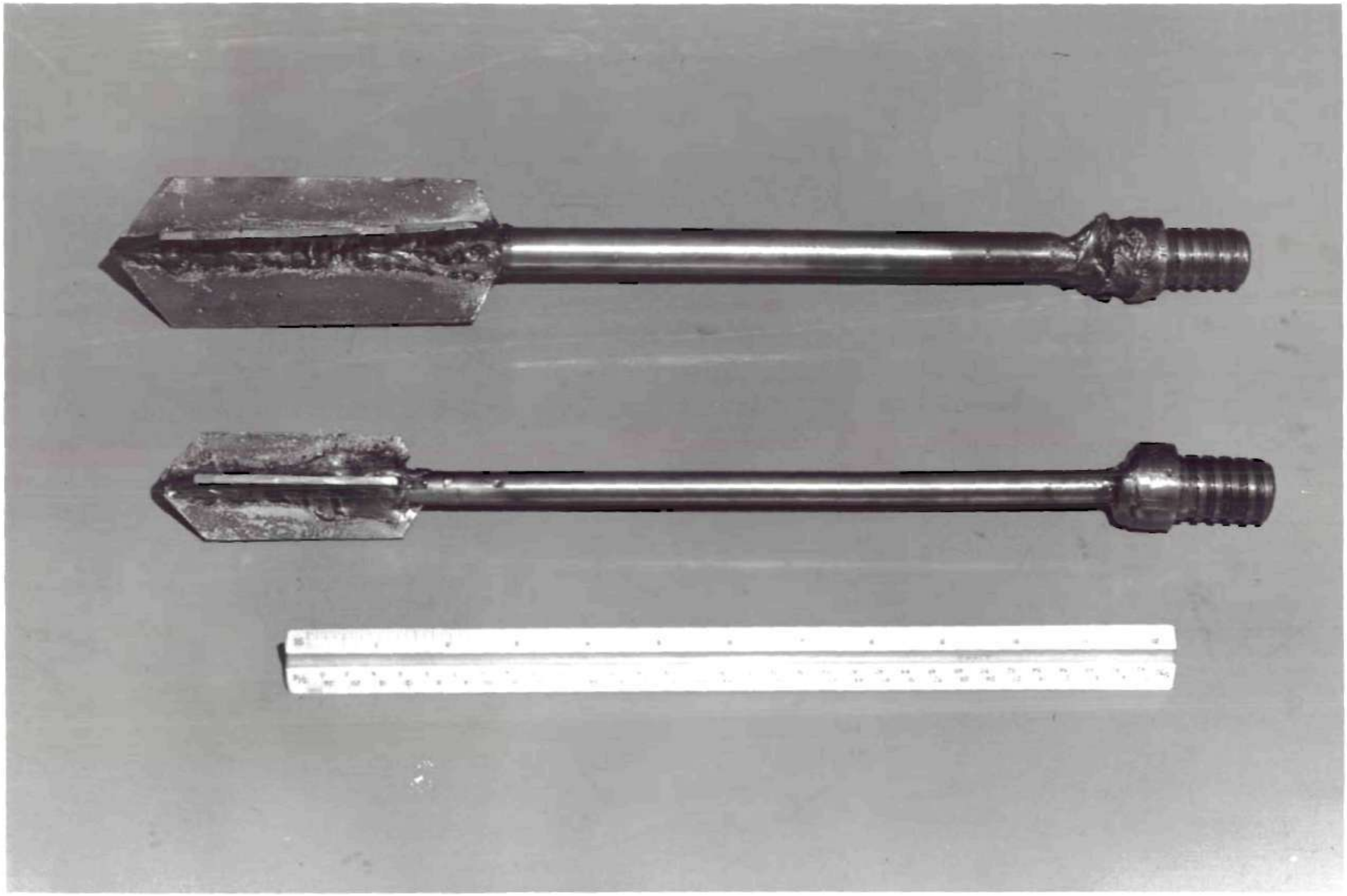
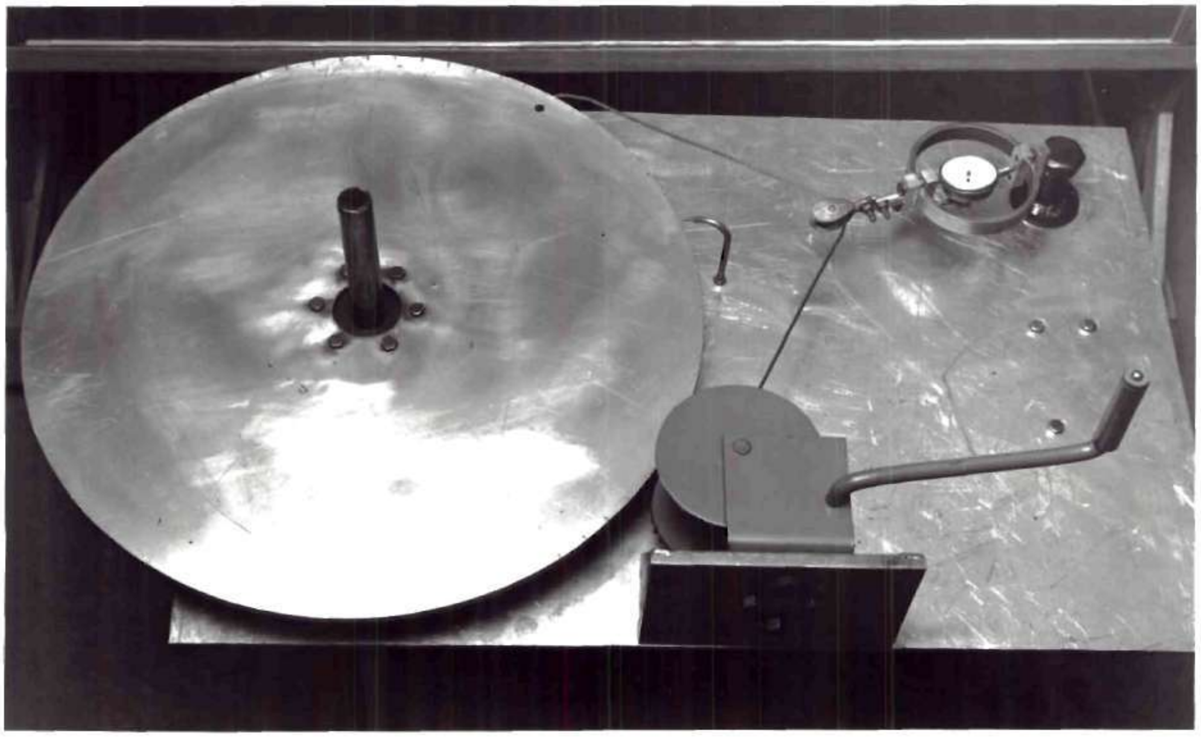
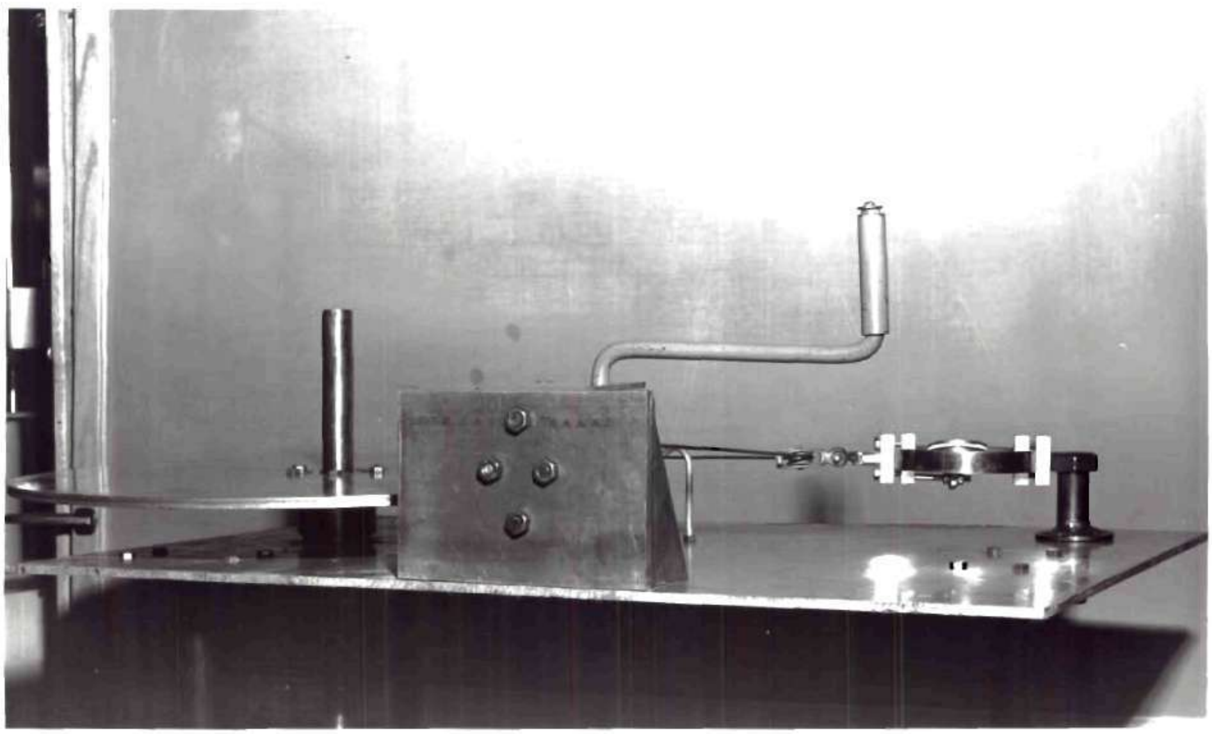


Fig. 1 ROTATING VANES



TOP VIEW



SIDE VIEW

Fig. 2 TORQUE HEAD

This aluminum disc was a portion of the "torque head" (Fig. 2). The base of the torque head was a rectangular aluminum plate which was raised two feet from the ground by three steel legs for convenience. A winch was mounted on the side of the plate, and a proving ring with a pulley attached was mounted on a corner of the plate. A steel cable was stretched from a point on the rim of the disc to the winch through a pulley attached to the proving ring. Rotation of the winch handle tightened the cable and caused the disc to be rotated. The stress applied to the proving ring through this procedure was measured with the micrometer gauge in the ring, and a pointer mounted on the base indicated the amount of rotation of the aluminum disc, which had been marked in degrees.

To resist the torque applied to the head of the apparatus, an aluminum angle section was attached to the legs of the device and then anchored to the ground with steel pins.

The equipment used for the laboratory shear testing of undisturbed samples consisted of a portable triaxial chamber and a beam scale loading machine. The triaxial chamber was capable of accepting either 1.4 inch or 2.8 inch diameter cylindrical soil samples at confining pressures of up to 100 pounds per square inch. The triaxial shear tests were performed by the controlled stress method of testing because of the nature of the loading machine.

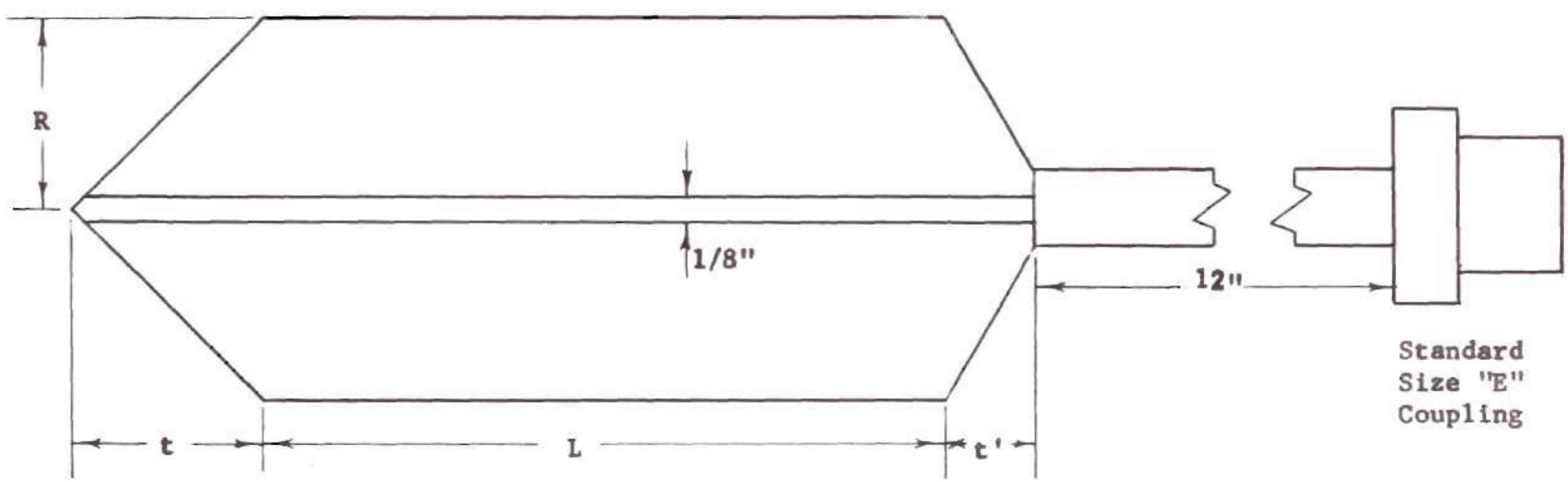


Fig. 3 SKETCH OF ROTATING VANE

Consolidation tests were performed in the consolidometers of the Georgia Institute of Technology, which will accept samples 2.375 inches in diameter. The load is applied to the samples through a lever arrangement by placing calibrated weights on suspended weight hangers.

CHAPTER III

CALIBRATION OF VANE SHEAR DEVICE

The equipment was designed to produce the key to an unknown relationship, making a physical calibration of the device impossible. The mathematical calibration was, therefore, accepted as wholly accurate. In the mathematical derivations which follow, the parts of the vanes which are referred to by letters amy be identified from Fig. 3.

Two primary assumptions were made in the calibration of the vane. First, it was assumed that the surface over which shear would be developed was the same as the surface of revolution of the vane itself. Second, it was assumed that the shear stress was constant along the conical sections at the top and bottom of the vane, regardless of the radius.

The total moment required to shear the soil may be represented as:

$$M = M_c + M_b + M_t - M_s , \qquad (1)$$

where M = total moment,

M_c = moment required to rotate the cylindrical section,

M_b = moment required to rotate the bottom conical section,

M_t = moment required to rotate the top conical section,

and M_s = moment previously included but not required to rotate the shaft.

The moment which is developed along the cylindrical surface of the vane may be expressed as:

$$M_c = 2\pi R^2 LT. \tag{2}$$

The moment developed on the conical sections then becomes:

$$m = M_b = M_t = M_c \left(\frac{\sqrt{R^2 + t^2}}{3L}\right). \tag{3}$$

Substituting into equation (1):

$$M = 2\pi R^2 LT \left\{1 + \frac{\sqrt{R^2 + t^2}}{3L} + \frac{\sqrt{R^2 + t'^2}}{3L}\right\}. \tag{4}$$

When this equation is solved for the unit shear stress (T), a constant may be determined for individual vanes. The small vane equation becomes:

$$T = 134.61\ M, \tag{5}$$

and the large vane equation becomes:

$$T = 28.85\ M. \tag{6}$$

The loading applied to the vane was measured by the proving ring placed at the apex to a triangle. The angle thus formed at the apex by the cable stretched from winch to torque wheel was measured during each test. A geometric factor is introduced into the total shear equation, which is a function of this angle (B). Therefore:

$$M = KSR, \tag{7}$$

where K = the reciprocal of cos $(B/2)$,

S = one half of the load as measured by the proving ring,

and R = the radius of the torque wheel or one foot.

When equation (7) is substituted into equations (5) and (6), the equation for the shear stress in the small vane becomes:

$$T = 134.61\ (K)\ (S), \qquad (8)$$

and the corresponding equation for the large vane becomes:

$$T = 28.85\ (K)\ (S). \qquad (9)$$

CHAPTER IV

TEST PROCEDURE

Four auger borings was made at the site selected for the field tests. These borings were made by manually twisting a sharpened augering tool into the ground. At depths of 3, 6, 8.5, and 11 feet, the augering operating was temporarily halted in each boring and a vane shear test was performed.

For these tests, the vane was driven into the soil for a distance of 12 inches beyond the bottom of the hole. This insured that the vane was firmly imbedded in soil which had not been disturbed by the augering operation. It was felt that, while some of the soil along the surface of the vane blades would certainly be disturbed to a large extent; the actual shear surface, which is defined by the cylinder encompassing the outer extremities of the blades, would remain relatively undisturbed.

The vane was then rotated at the rate of four degrees per minute. Each fifteen seconds, the stress measured by the proving ring was recorded.

After the failure of the soil was definitely recognized, the vane was rotated through an additional 180° to destroy the soil structure; the vane was then withdrawn, and the augering operation resumed.

Conventional undisturbed samples were taken in separate borings near the vane tests, at elevations corresponding to those of the vane tests. Portions of these samples were trimmed to a diameter of 1.4 inches and a height of approximately 3 inches with a tubular specimen cutter. Both ends of the sample were covered with porous stone discs. The specimen was then extruded from the cutting tube, placed on the sample base of the triaxial chamber, and encased in a thin rubber membrane. The chamber was assembled, placed in the loading machine, the micrometer dial attached to measure vertical sample deformation, and the confining pressure applied to the sample with compressed air. Load was applied to the sample in increments and the deformation measured until it became impossible to maintain a constant load on the sample. This final load was considered the failure load. From the data thus obtained, the stress-strain curves and Mohr diagram for each sample were drawn.

Another portion of the sample was cut into a disc 2.4 inches in diameter and 1.0 inches thick and placed in the consolidometer. Porous plates were placed on both ends of the sample, and the assembled consolidometer was placed in the loading frame. Loads of 100, 500, 1000, 2000, 4000, 8000, 16000, and 32000 pounds per square foot were applied by means of suspended weights to the samples thus prepared. The deformation of the sample under each load was measured by a micrometer dial gauge, and the next higher load was applied to the sample after all settlement had ceased. The data thus obtained provided the basis for the pressure-void ratio curves. The preconsolidation load applied to the soil was approximated by the method proposed by Doctor Arthur Cassagrande(7).

CHAPTER V

DISCUSSION OF RESULTS

Several notations were made during the performance of the tests, which help to explain the results which were obtained. Test number 4-2-1, made at a depth of three feet with the larger of the two vanes, showed an extremely high failure stress. The failure stress in this test was very close to the limit of the proving ring used. When the vane was recovered, the blades of the vane were bent slightly out of line, indicating that this was also the limit of the stress which could be absorbed by the vane. In this test, a quarter cylinder of soil sheared out by the testing operation was recovered. This segment of soil was of the same radius as the vane, thus bearing out the accuracy of the theory that the surface of the solid of revolution of the vane is in fact the shear surface.

Tests 4-2-4 and 5-1-3 also indicated very high failure stresses. The behavior of the vanes as they were driven into the ground, as well as the results of the tests performed, indicates that the vanes had pierced one of the many thin rock seams which were observed in several of the undisburbed samples obtained at the site. This explains the difficulty experienced in driving the vane, as well as the abnormally high stresses encountered.

If the strain involved in the vane tests is considered as function of the rotation of the vanes, the amount of strain measured at the failure stress is quite high. When compared with the results of the triaxial tests, the failure strains involved in the vane tests are approximately 300 to 400 per cent greater. This discrepancy could possibly be explained by the presence of the disturbed layer of soil along the face of the vanes, which undoubtedly presented different characteristics from those of the undisturbed segments of the soil mass.

The triaxial shear tests indicate that the apparent cohesive force varied from 0.2 to 2.7 kips per square foot and that the angle of internal friction varied from 14.4° to 30.1° through the several samples tested. The consolidation tests performed indicate that the preconsolidation loads varied from 1300 to 6600 pounds per square foot. The range of these soil characteristics allowed comparison of the two test methods on several bases. The results of these tests are tabulated in the TABLE OF TEST RESULTS in the Appendix.

The first relationship which was considered was the comparison of the shear strength indicated by the vane tests with the shear strength indicated by the triaxial shear tests. For the purpose of determining the approximate shear strength of the soil from the triaxial tests, the existing overburden pressure was calculated, using the unit weights of the samples taken. The shear strength was taken from the Mohr diagram for each sample, using this confining pressure. These values are graphically presented in Fig. 4.

Since there was no apparent relationship involved in this direct comparison, the ratio of the vane shear strength to the strength indicated by the triaxial test at a confining pressure equal to the weight of overburden was determined. This ratio was then plotted against values of angle of internal friction determined from the Mohr diagrams (Fig. 5). This relationship again failed to indicate a trend.

Values of the vane shear strength were plotted against the depth at which they were made (Fig. 6) and against the unconfined compressive strength of the soil as determined in the triaxial shear tests (Fig. 7). Neither of these comparisons indicated a constant or a consistently varying relationship.

Preconsolidation load is the term applied to the residual stresses in the soil mass. These may be caused by previous loading of the soil by any of several means, including the placing and subsequent removal of a surcharge load. If the preconsolidation load on a sample of soil is different from the overburden pressure, the shear characteristics of the soil will be those determined from the Mohr diagram at the preconsolidation load, rather than from the existing overburden pressure, because of the residual stresses remaining in the grain structure due to the previous load.

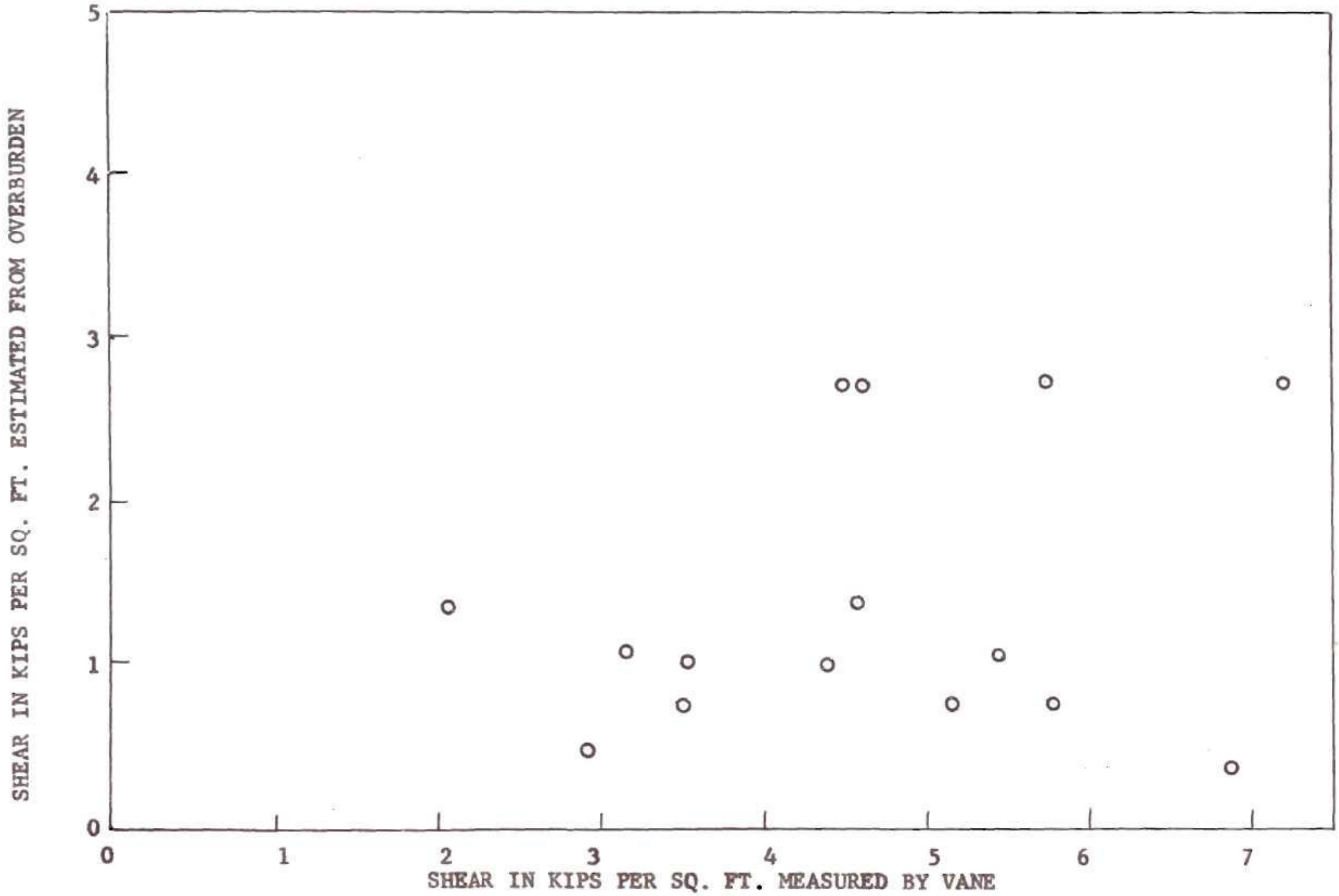


Fig. 4 SHEAR MEASURED BY VANES VS. SHEAR ESTIMATED FROM OVERBURDEN

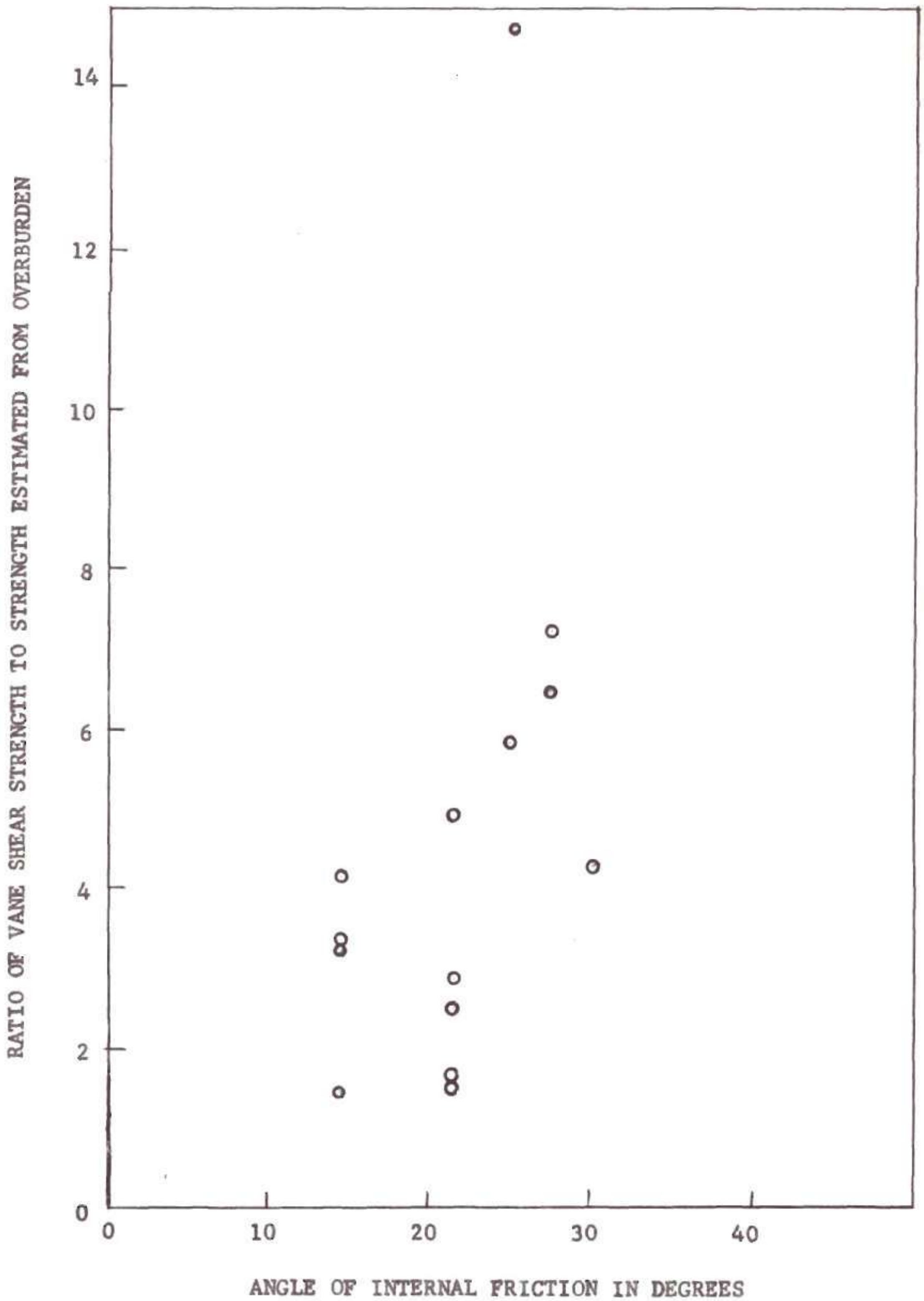


Fig. 5 RATIO OF VANE SHEAR STRENGTH TO STRENGTH ESTIMATED FROM OVERBURDEN VS. ANGLE OF INTERNAL FRICTION

A graphical comparison of the shear strengths thus obtained and the vane shear strengths (Fig. 8) seems to indicate that the vane shear results are approximately 1.5 times the corresponding value determined through standard laboratory testing.

The ratio of the vane shear strength to the laboratory shear strength was again graphically related to the angle of internal friction (Fig. 9). Again there was no visible correlation between the shear strength and the angle of internal friction.

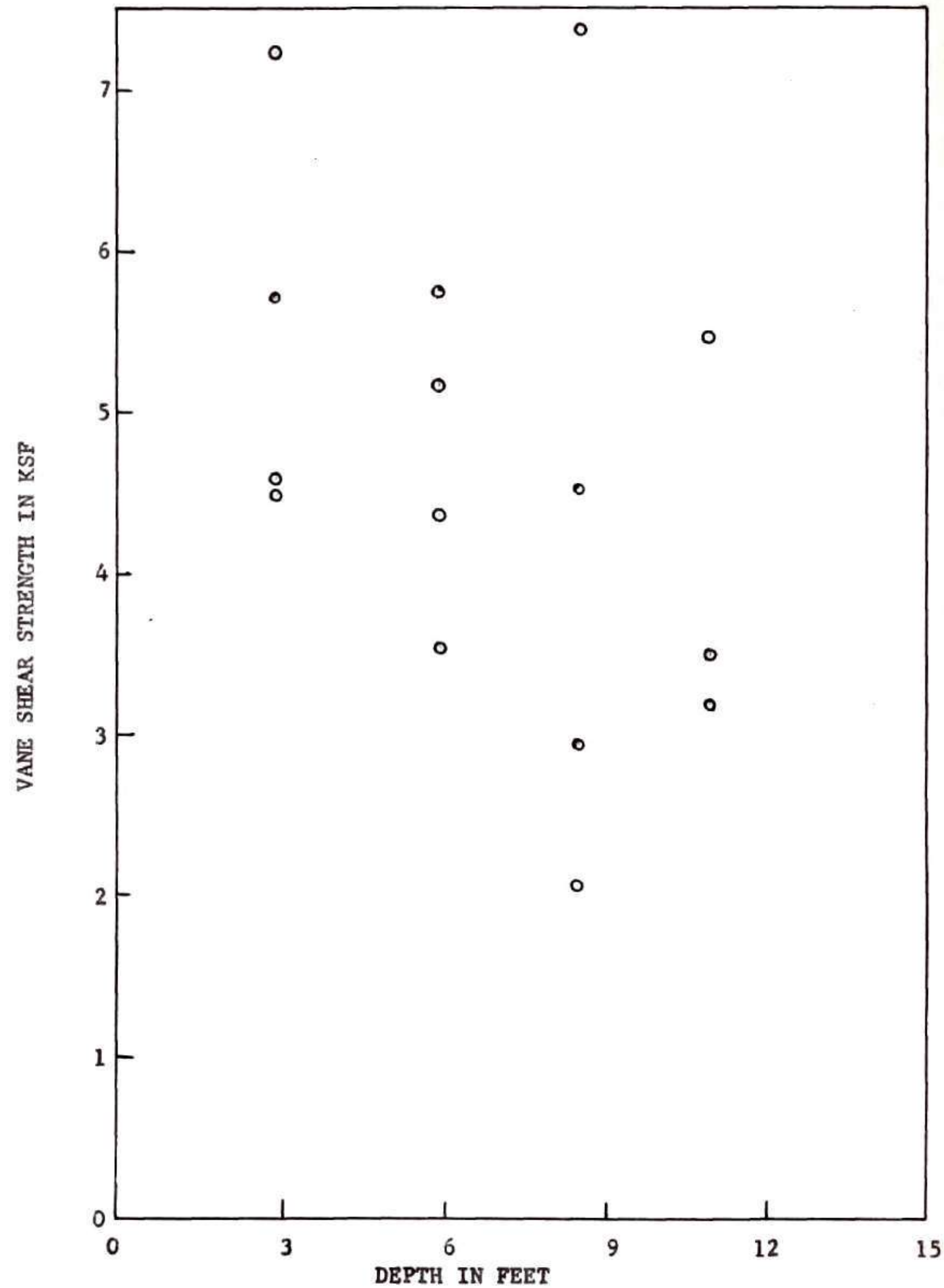


Fig. 6 VANE SHEAR STRENGTH VS. DEPTH

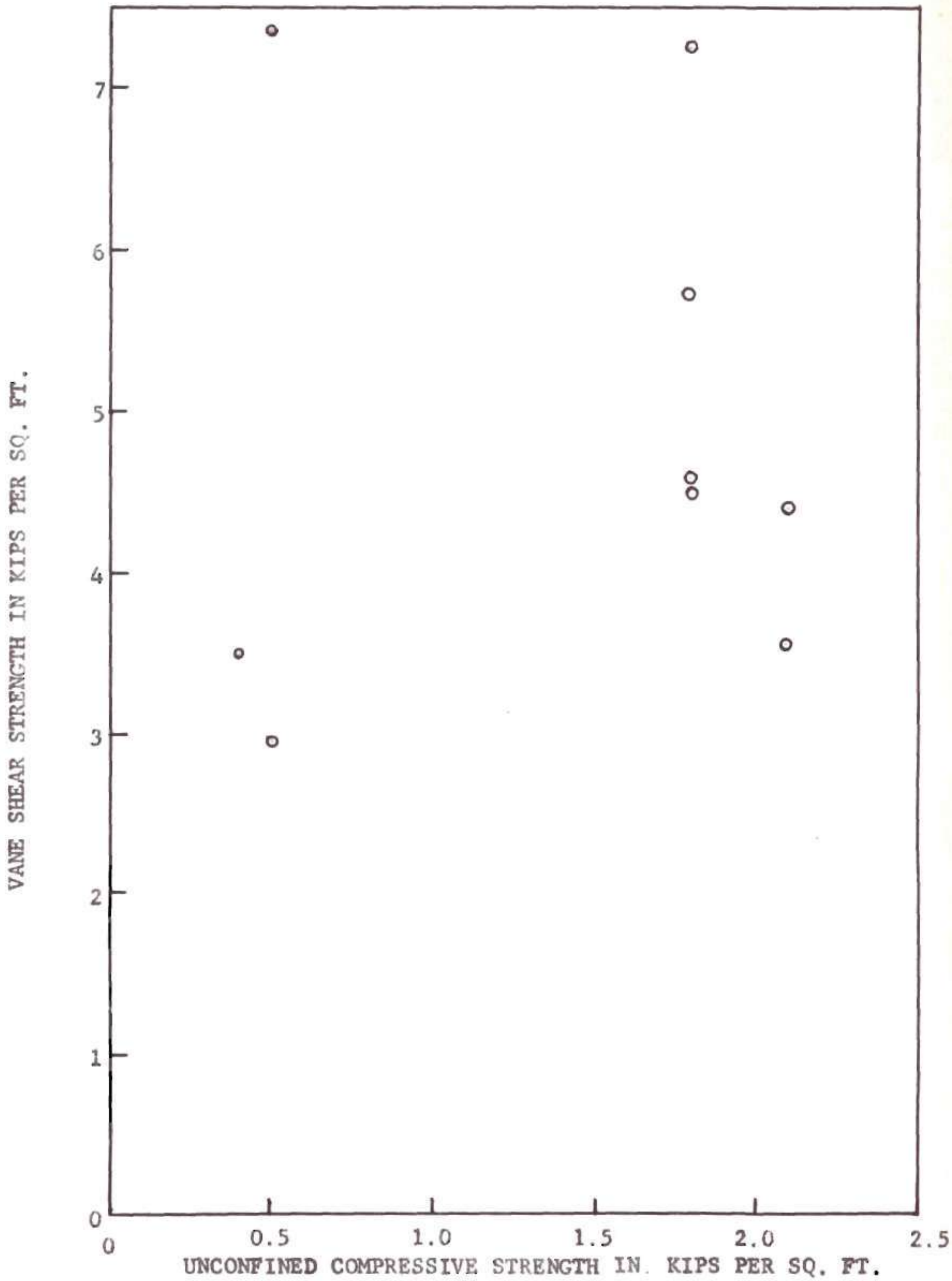


Fig. 7 VANE SHEAR STRENGTH VS. UNCONFINED COMPRESSIVE STRENGTH

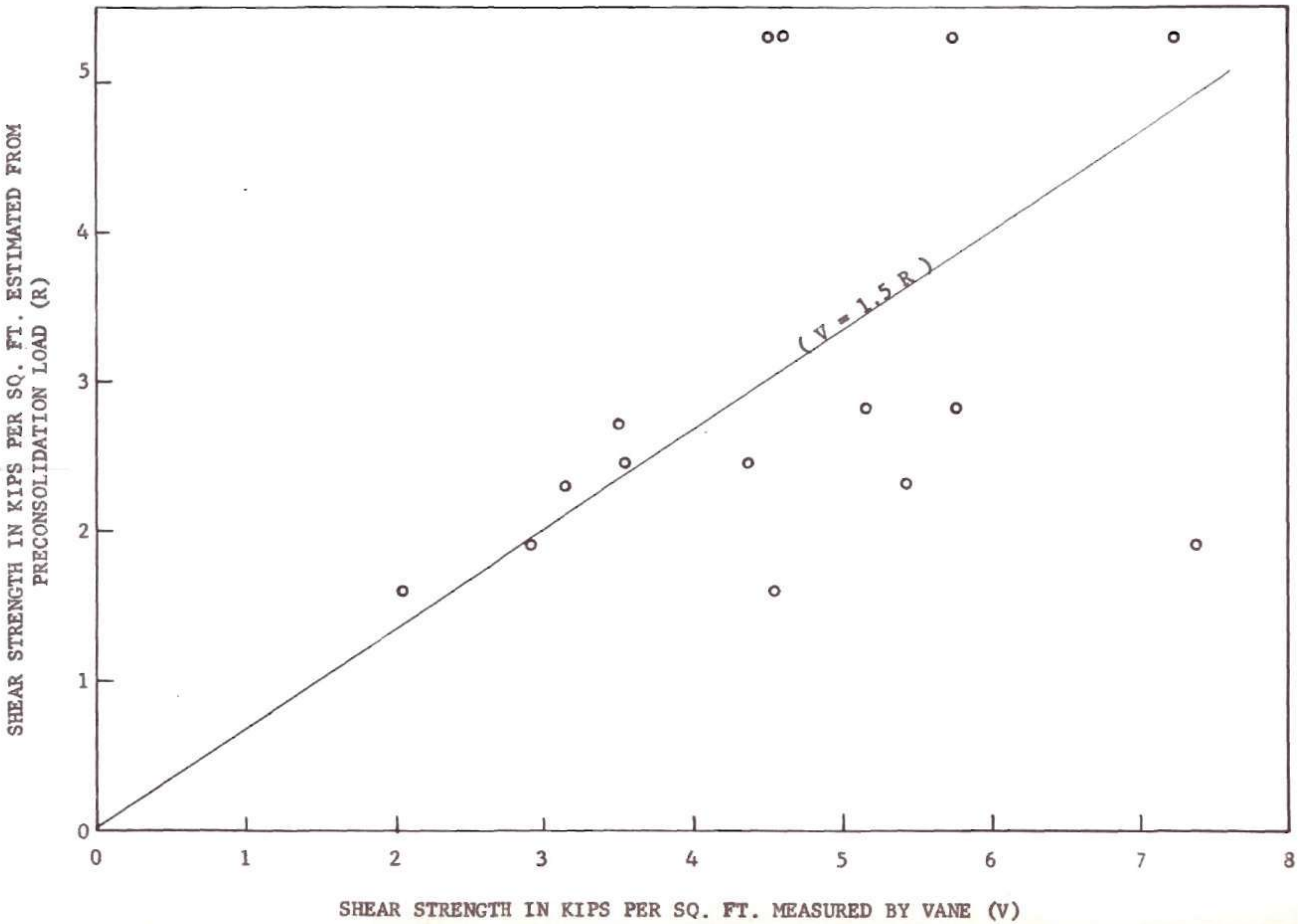


Fig. 8 SHEAR MEASURED BY VANES VS. SHEAR ESTIMATED FROM PRECONSOLIDATION LOAD

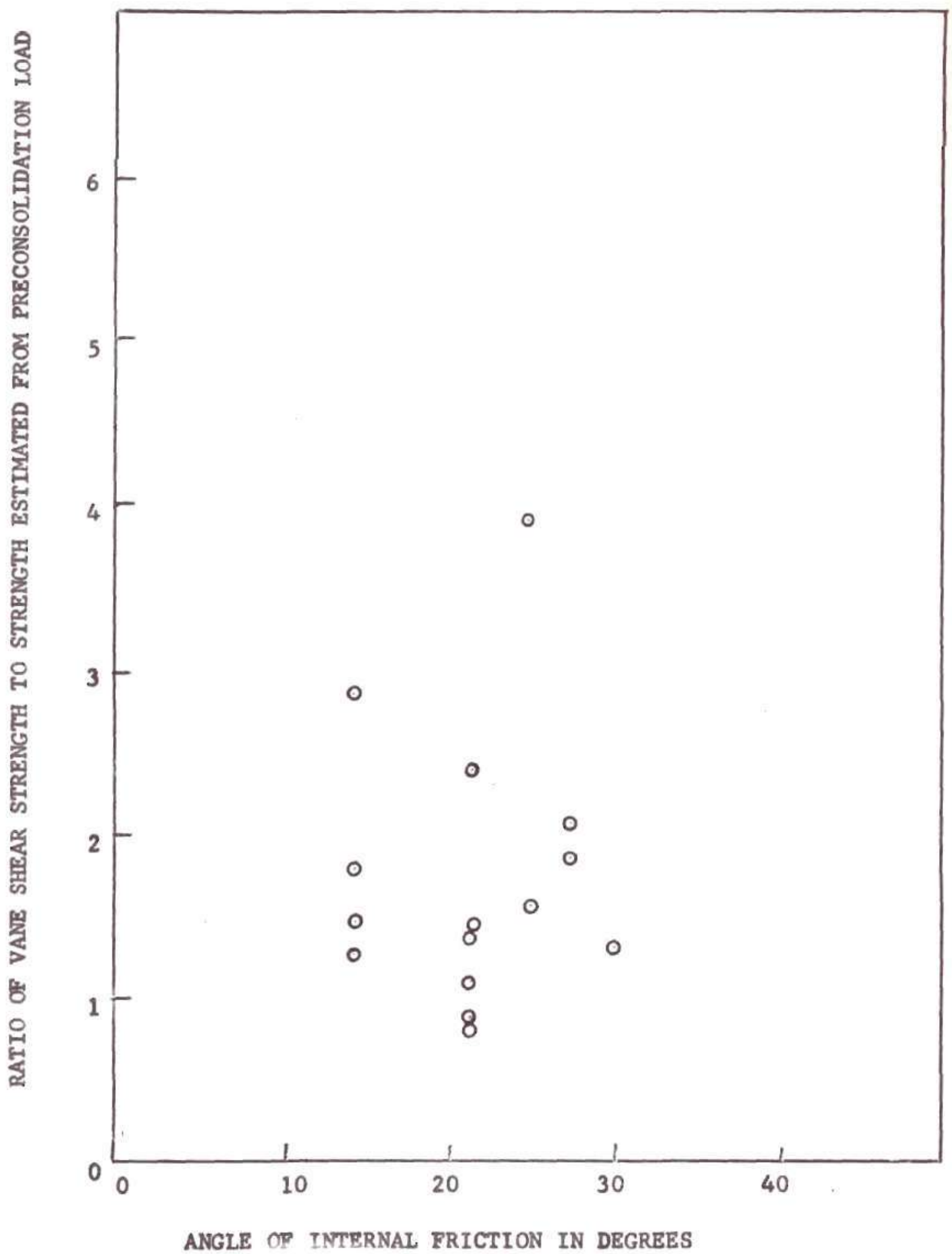


Fig. 9 RATIO OF VANE SHEAR STRENGTH TO STRENGTH ESTIMATED FROM PRECONSOLIDATION LOAD VS. ANGLE OF INTERNAL FRICTION

CHAPTER VI

CONCLUSIONS

When this series of tests was begun, it was hoped that a direct relationship could be found between the shear strengths determined through use of the rotating vane and the strengths determined in the laboratory tests. While the test results do indicate that a relationship does exist, it is not as simple as was originally hoped. However, it does appear that there is a strong possibility that the rotating vane can be used in exploratory work and that its use will give a result which is greater than the strength indicated in the laboratory tests. This ratio of vane strength to laboratory strength appears to be in the magnitude of approximately 1.5:1, for the range of preconsolidation loads encountered in this soil.

The exact calibration of the device in a particular soil would include, as does this thesis, a triaxial shear test and a consolidation test. However, it can be stated that the results of the vane test will probably be higher than the laboratory tests. How much closer the vane shear results are to the actual strength of the soil in a totally undisturbed state cannot be determined, but they do seem to give a better indication than any previous method of testing.

Further research into the vane shear apparatus is needed, and, based on the existing data, it seems that more research is justified. Not only does the device appear to be basically more accurate, but the tests are much easier to perform than the complex laboratory tests. The greatest weakness of the vane test, as observed by this writer, is the fact that the results of the test give only the shear strength of the soil at the point of the test and under the same stress conditions as the soil now experiences. No information can be derived with regard to the angle of internal friction or the apparent cohesion of the soil. Information regarding these soil characteristics can only be determined through the standard laboratory tests.

Because of its limitations, the vane may not be readily applicable to foundation investigations in the Piedmont Region, but it should be valuable in investigating shear type soil failures.

CHAPTER VII

RECOMMENDATIONS

Improved instrumentation could possibly provide better information in further testing. Refinement of the torque head should include redesigning the head for increased portability and greater accuracy. The first refinement could possibly be made by making the components act around a central shaft as a compact unit. The second refinement in design should include a provision for applying the torque in a more constant manner than was possible through the apparatus used in this thesis. The vane itself could be refined to reduce the disturbance of the soil through reducing the thickness of the vane itself, but the vanes would have to be carefully designed to insure adequate structural strength.

If possible, much information regarding the relationship of the shear strength measured by the vanes could be gained through the plan of research which follows. A sandy silt or clay similar to the soils in this area could be completely remolded in the laboratory. Samples of the soil could be pre-loaded to a known preconsolidation load until all settlement under the load had ceased. Then, a vane shear test could be made in the soil, and this could be correlated with a triaxial shear test made in the same material. This type of testing program would provide a more controlled situation than was possible in the field tests performed in connection with this thesis. Not only could the preconsolidation load be accurately controlled, but there would be a uniformity of soil conditions which cannot be duplicated in field tests in this region.

A P P E N D I X

TABLE OF TEST RESULTS

Test No.	Vane Failure Stress (V)	Vane Failure Strain	Unconfined Compressive Strength	Angle of Internal Friction	Estimated Failure Stress (from the overburden pressure) (T)	Ratio (V/T)	Precon- solidation Pressure	Estimated Failure Stress (from the precon- solidation pressure (R)	Ratio (V/R)
2-1-1	4500	0.30	1800	21.5	2850	1.58	6600	5300	0.85
2-1-2	5180	0.21	**	27.5	800	6.48	4600	2800	1.85
2-1-3	2050	0.26	**	14.4	1400	1.46	1300	1600	1.28
2-1-4	3500	0.21	400	30.1	800	4.38	4400	2700	1.30
3-2-1	4600	0.26	1800	21.5	2850	1.65	6600	5300	0.87
3-2-2	3550	0.27	2100	14.5	1050	3.38	6100	2450	1.45
3-2-3	2920	0.38	500	25.0	500	5.85	3800	1900	1.54
3-2-4	5460	0.48	**	21.5	1100	4.96	4400	2300	2.38
4-2-1	7230	0.44	1800	21.5	2850	2.54	6600	5300	1.36
4-2-2	5770	0.39	**	27.5	800	7.22	4600	2800	2.06
4-2-3	4540	0.19	**	14.4	1400	3.24	1300	1600	2.84
4-2-4	*	*	400	30.1	800	*	4400	2700	*
5-1-1	5730	0.16	1800	21.5	2850	1.98	6600	5300	1.08
5-1-2	4390	0.21	2100	14.5	1050	4.17	6100	2450	1.79
5-1-3	7380	0.22	500	25.0	500	14.76	3800	1900	3.89
5-1-4	3160	0.27	**	21.5	1100	2.87	4400	2300	1.37

NOTE: The test numbers are assigned as follows: The first number is the boring number, the second number is the vane number (1 = small vane, 2 = large vane), the third number refers to the number of the test in the boring.

* Test halted to avoid overstressing proving ring.
** No unconfined compression test performed.

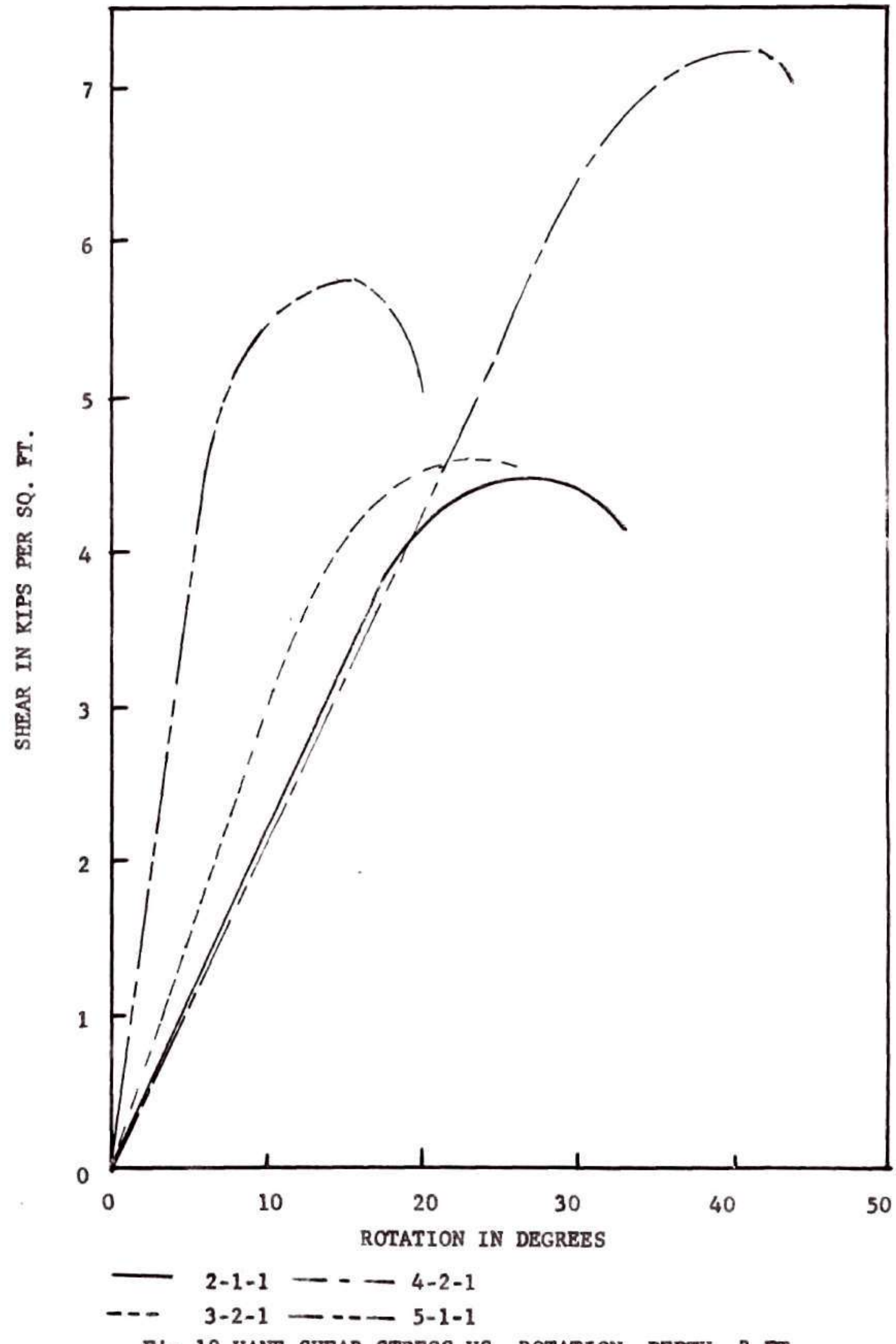


Fig.10 VANE SHEAR STRESS VS. ROTATION, DEPTH, 3 FT.

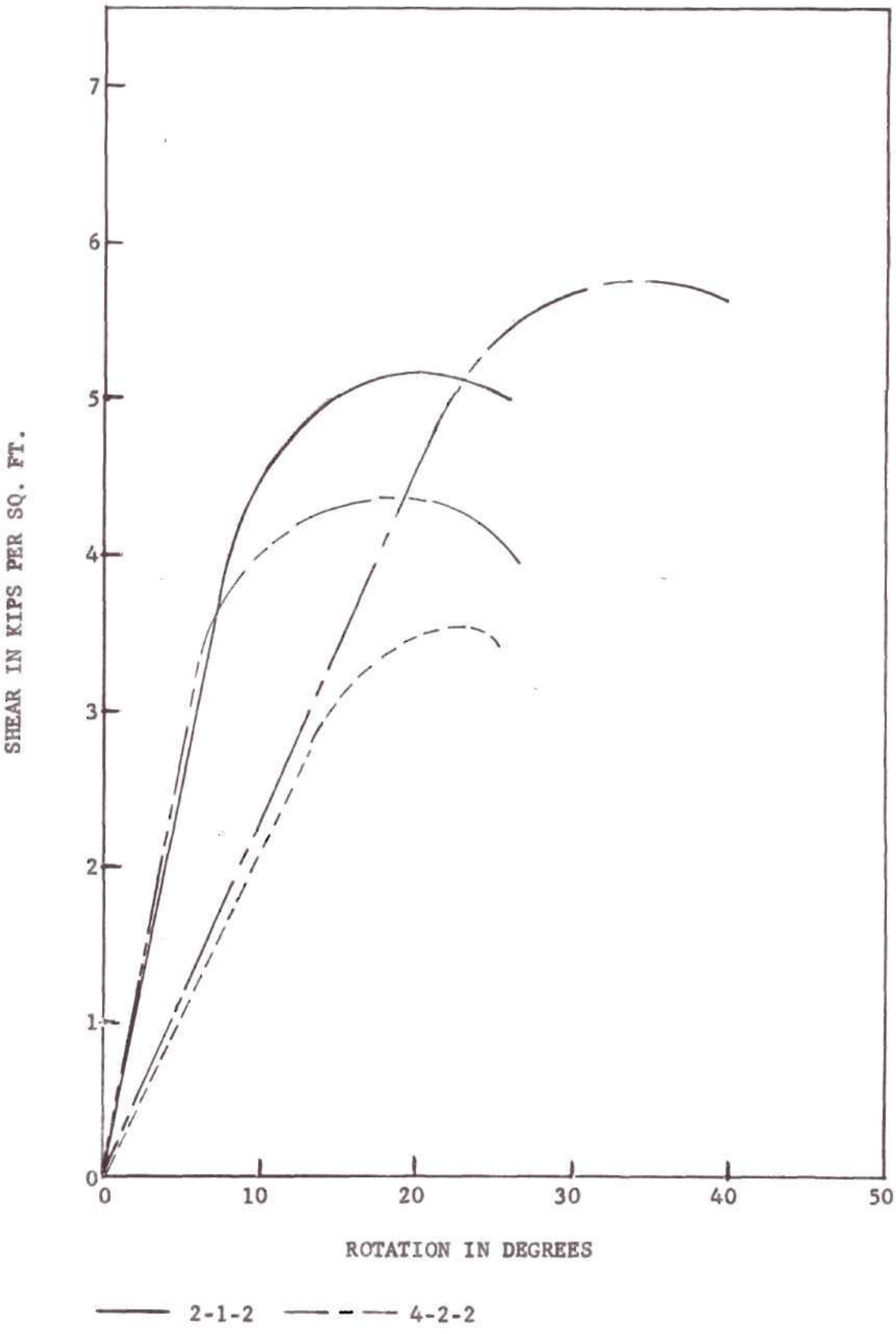


Fig. 11 VANE SHEAR STRESS VS. ROTATION, DEPTH, 6 FT.

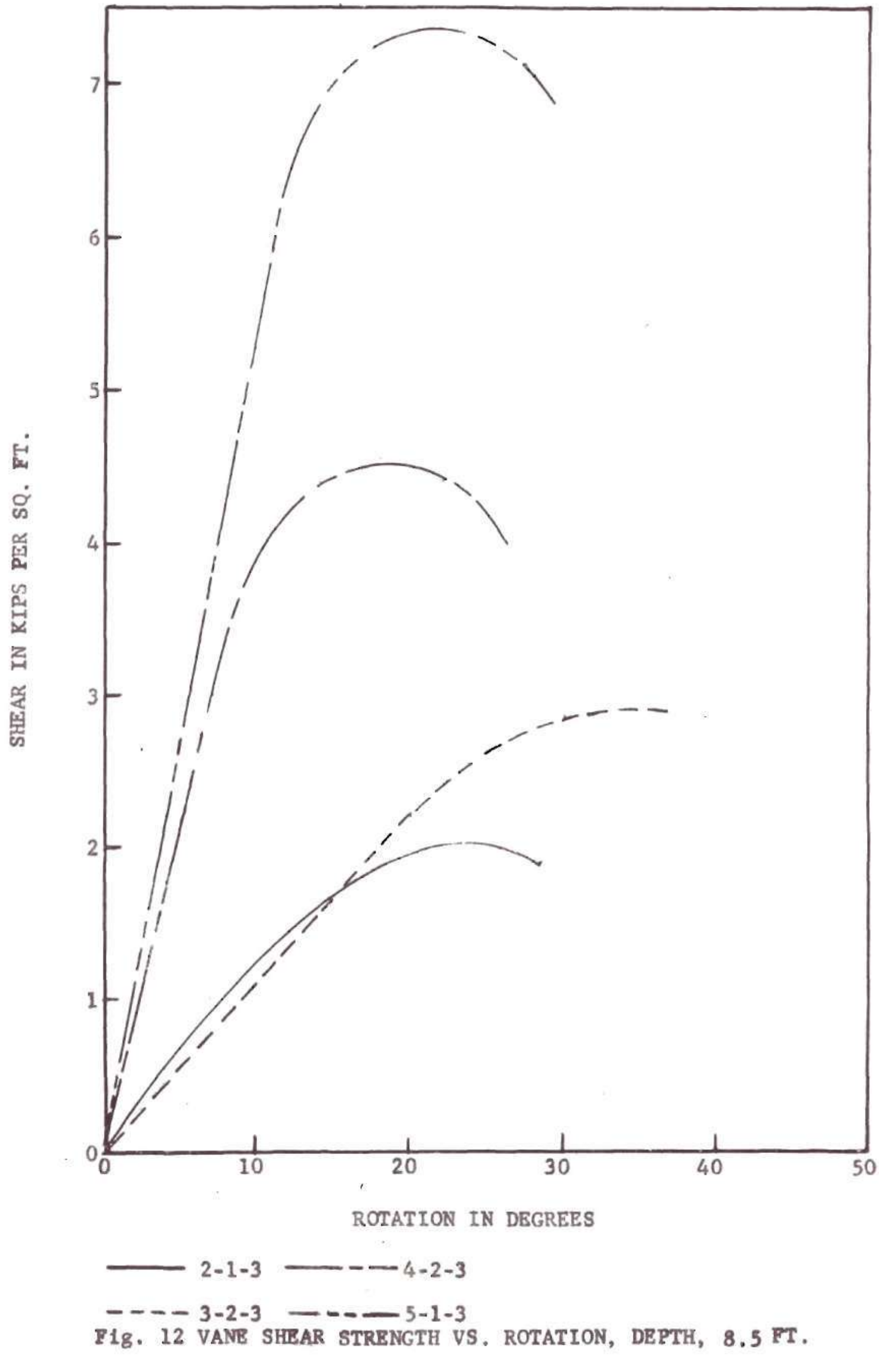


Fig. 12 VANE SHEAR STRENGTH VS. ROTATION, DEPTH, 8.5 FT.

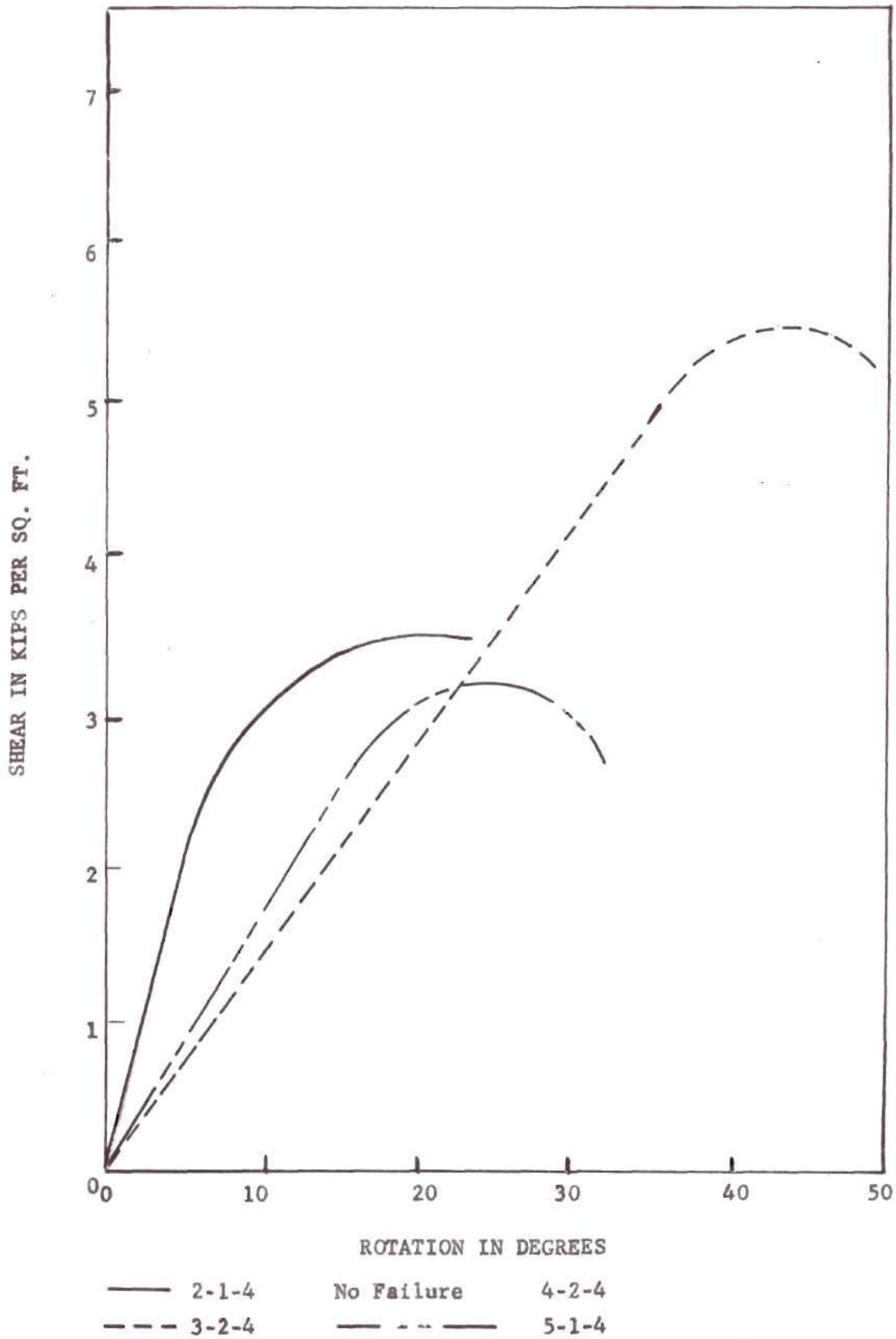
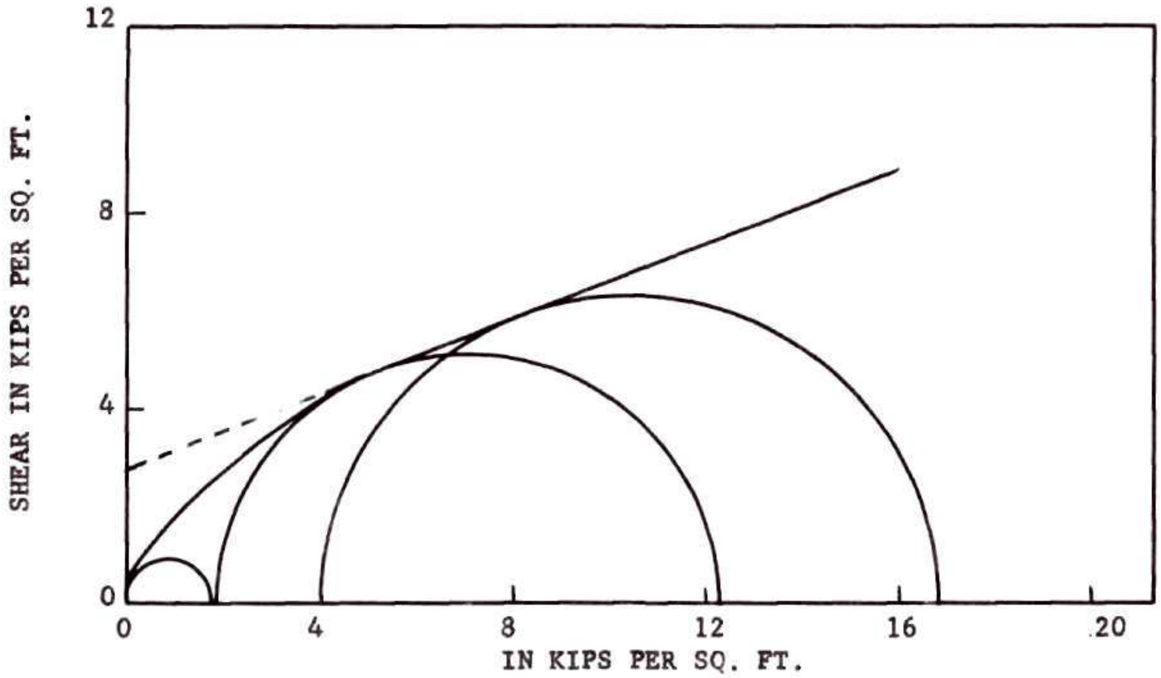


Fig. 13 VANE SHEAR STRENGTH VS. ROTATION, DEPTH, 11 FT.



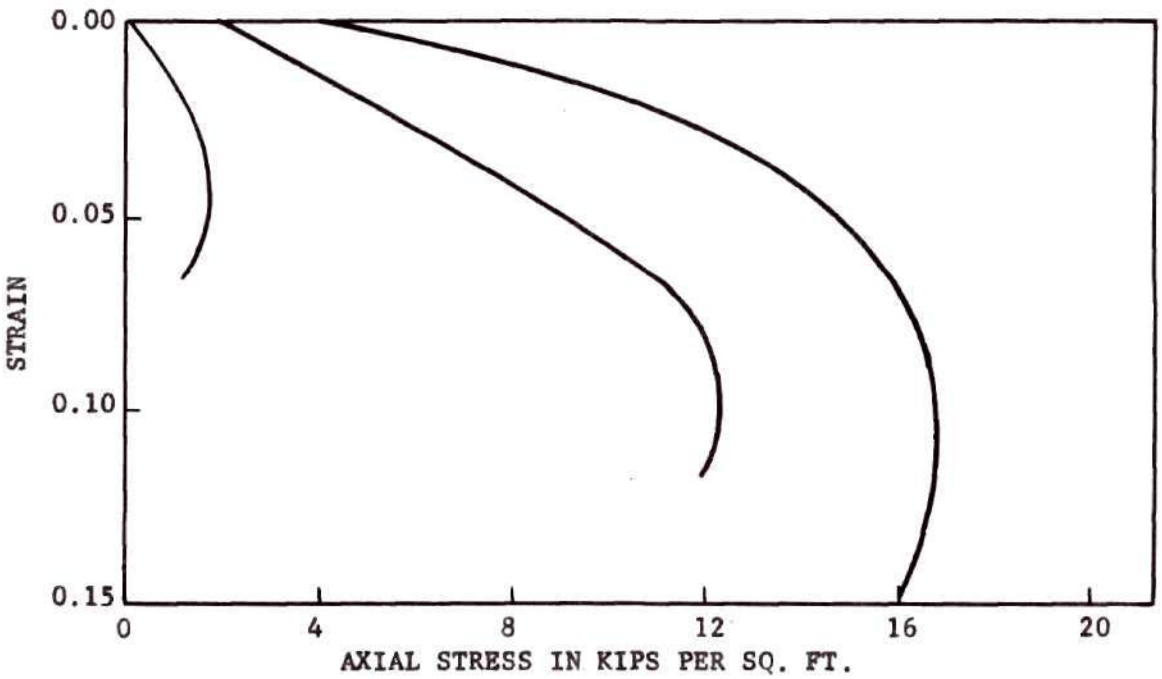


Fig. 14 STRESS VS. STRAIN AND MOHR DIAGRAM, DEPTH, 3 FT.

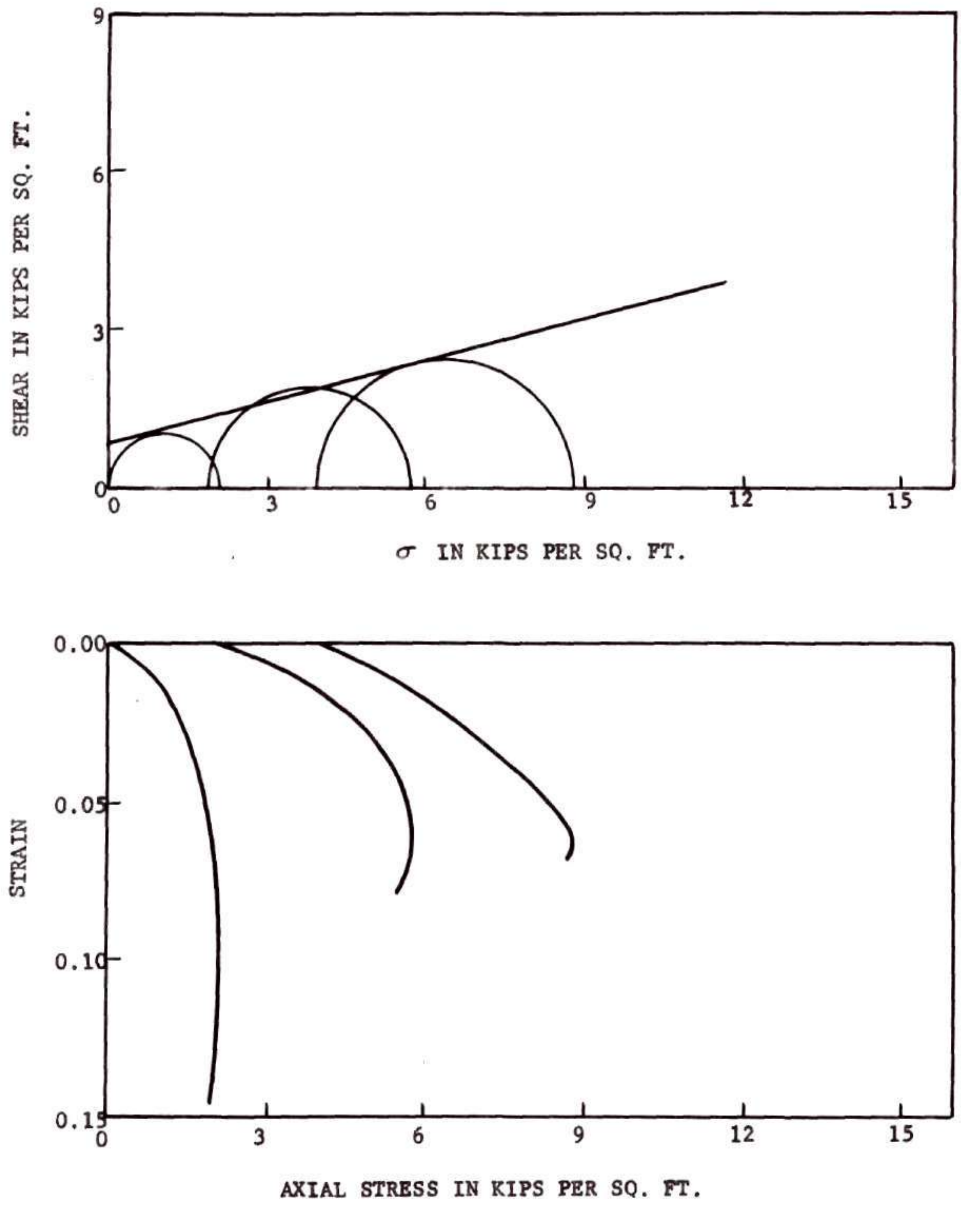


Fig. 15 STRESS VS. STRAIN AND MOHR DIAGRAMS, DEPTH, 6 FT., BORINGS 3 AND 5

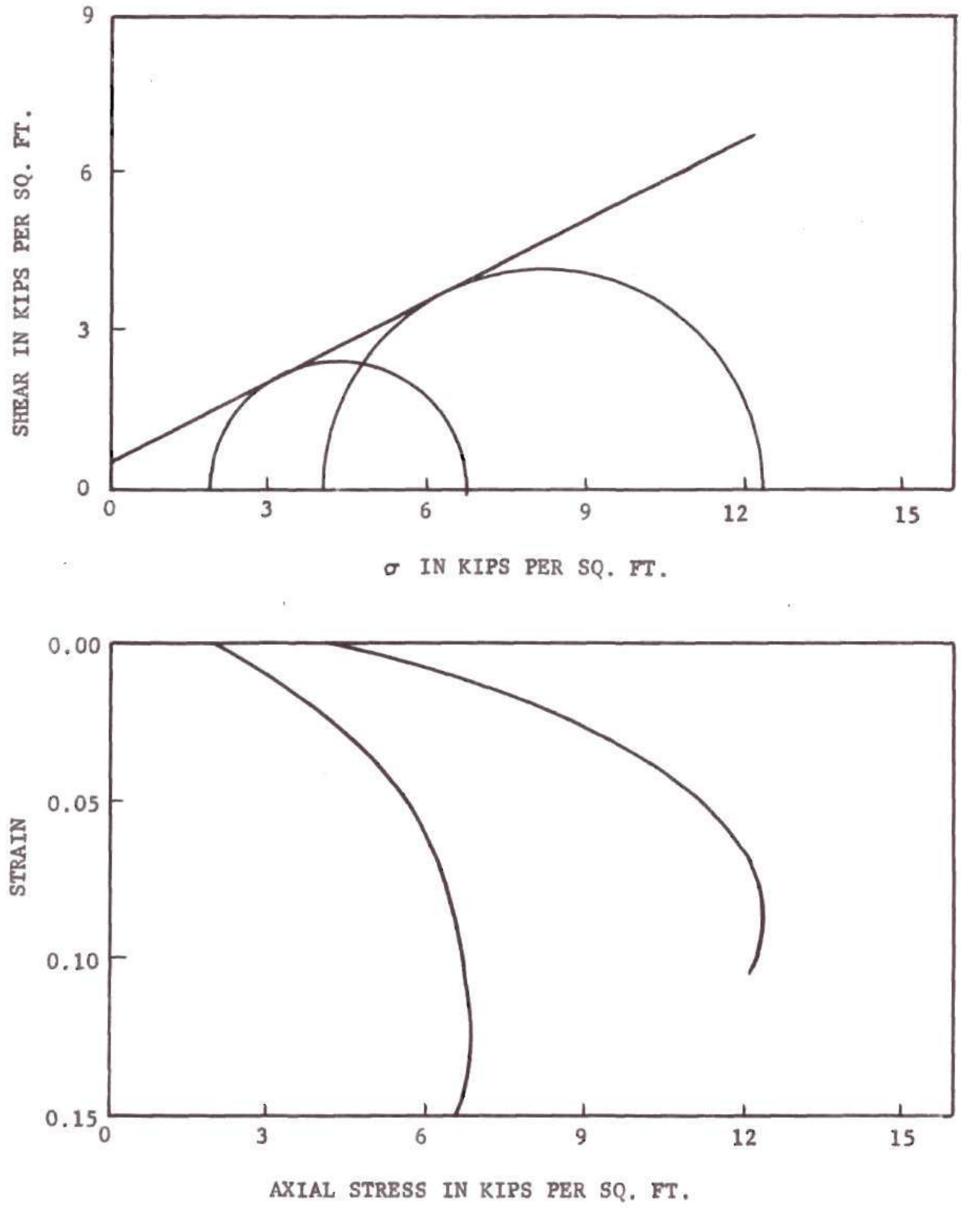


Fig. 16 STRESS VS. STRAIN AND MOHR DIAGRAM, DEPTH, 6 FT., BORINGS 2 AND 4

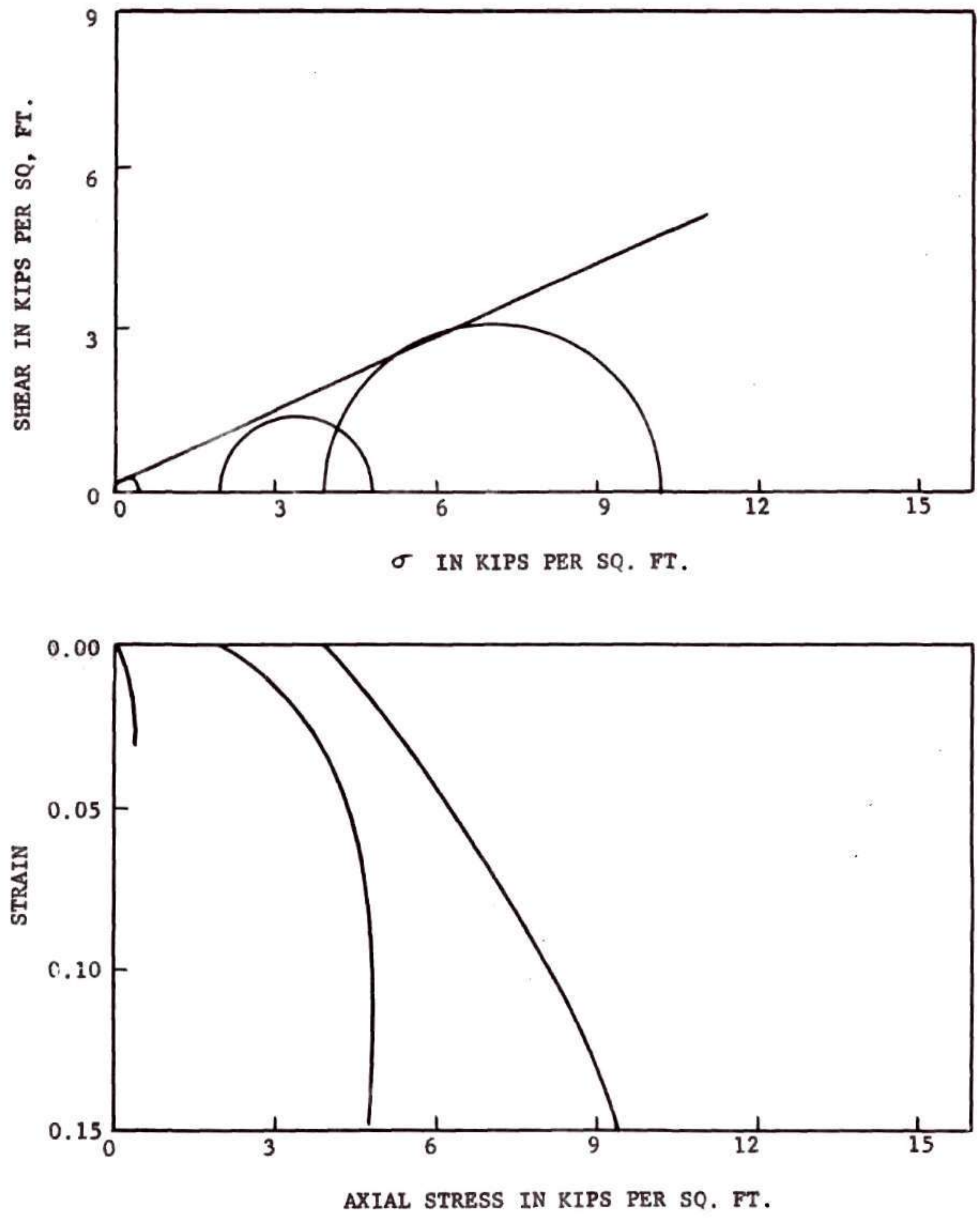


Fig. 17 STRESS VS. STRAIN AND MOHR DIAGRAMS, DEPTH, 8.5 FT., BORINGS 3 AND 5

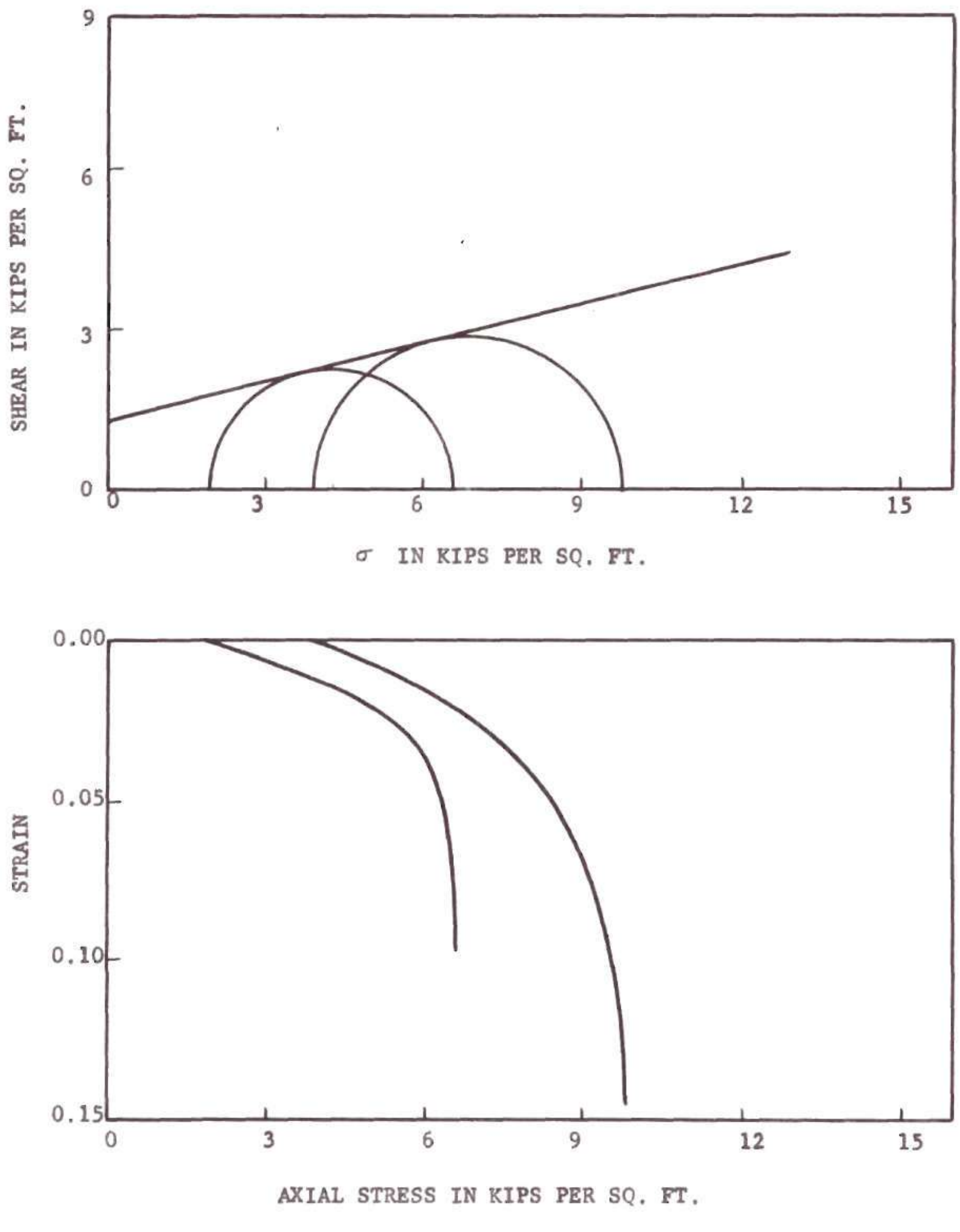


Fig. 13 STRESS VS. STRAIN AND MOHR DIAGRAM, DEPTH, 8.5 FT., BORINGS 2 AND 4

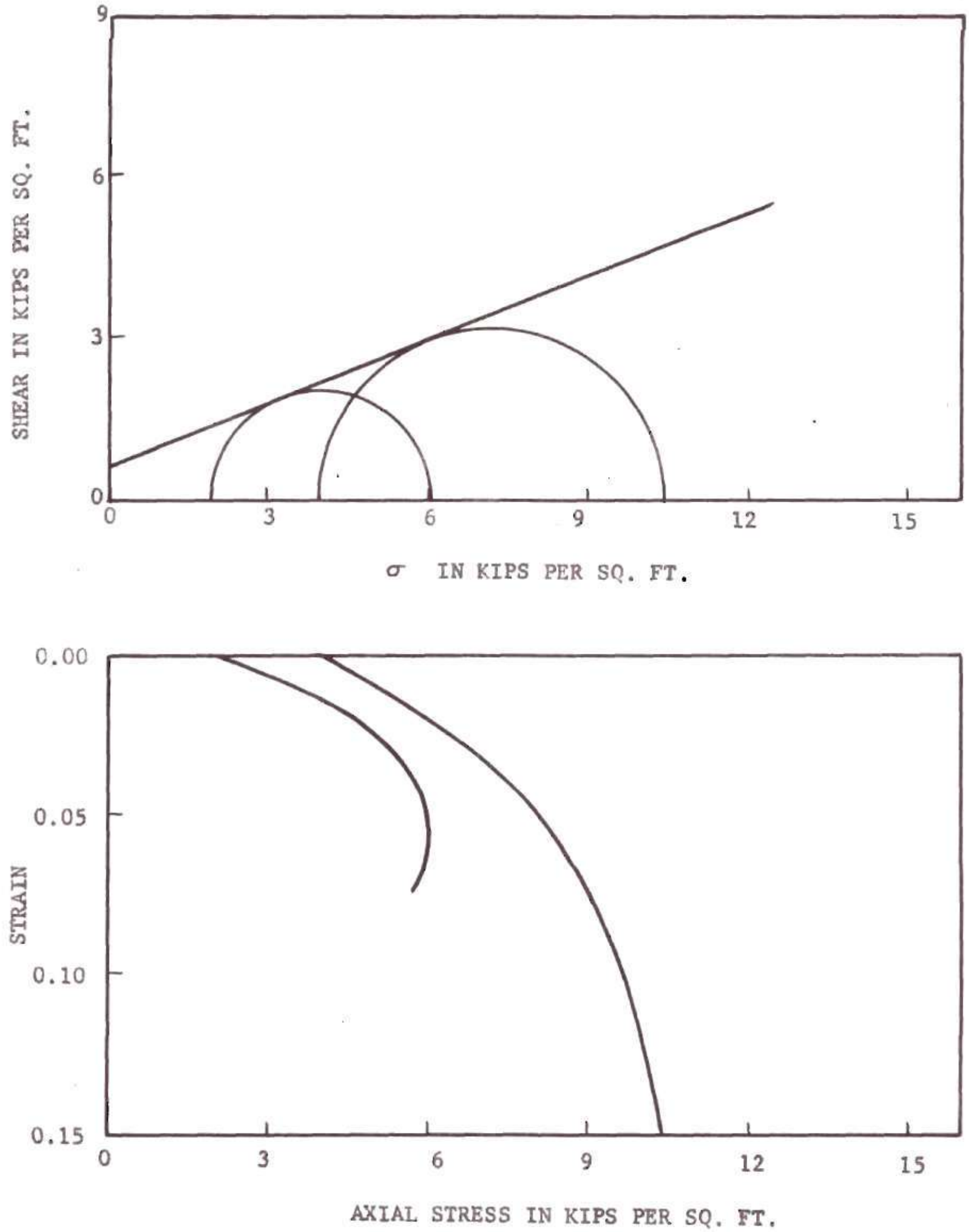


Fig. 19 STRESS VS. STRAIN AND MOHR DIAGRAMS, DEPTH, 11 FT., BORINGS 3 AND 5

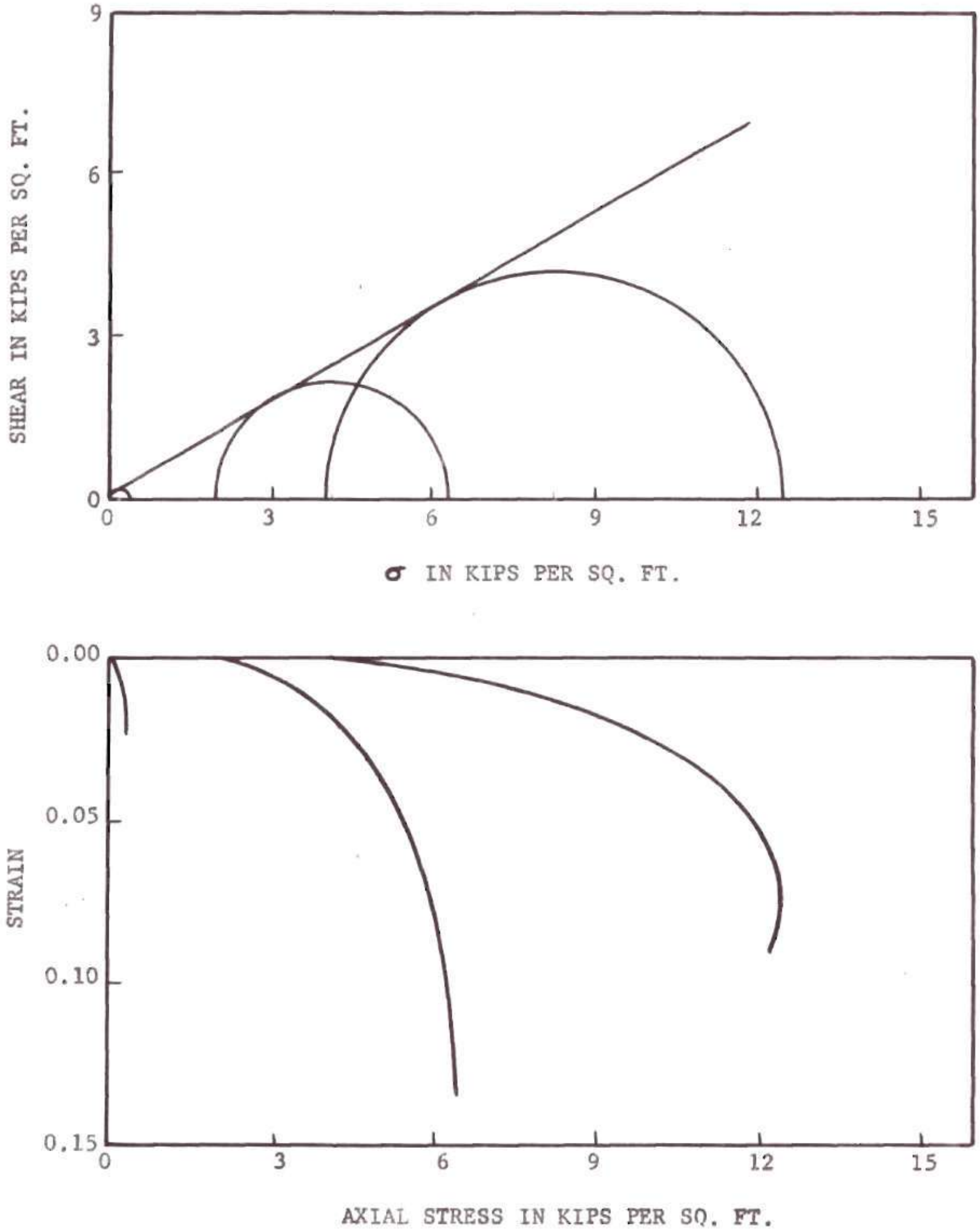


Fig. 20 STRESS VS. STRAIN AND MOHR DIAGRAM, DEPTH 11 FT., BORINGS 2 AND 4

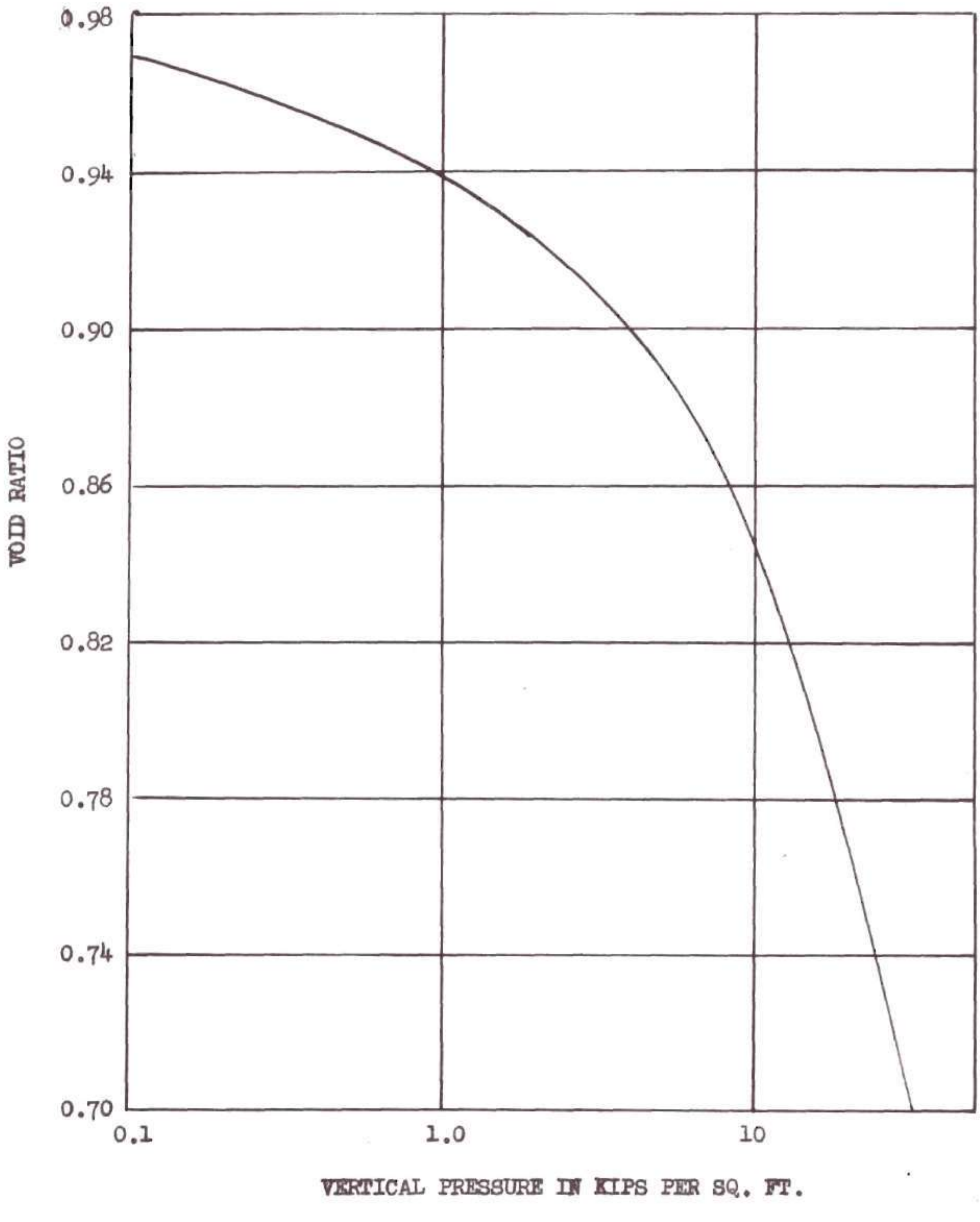


Fig. 21 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 3 FT.

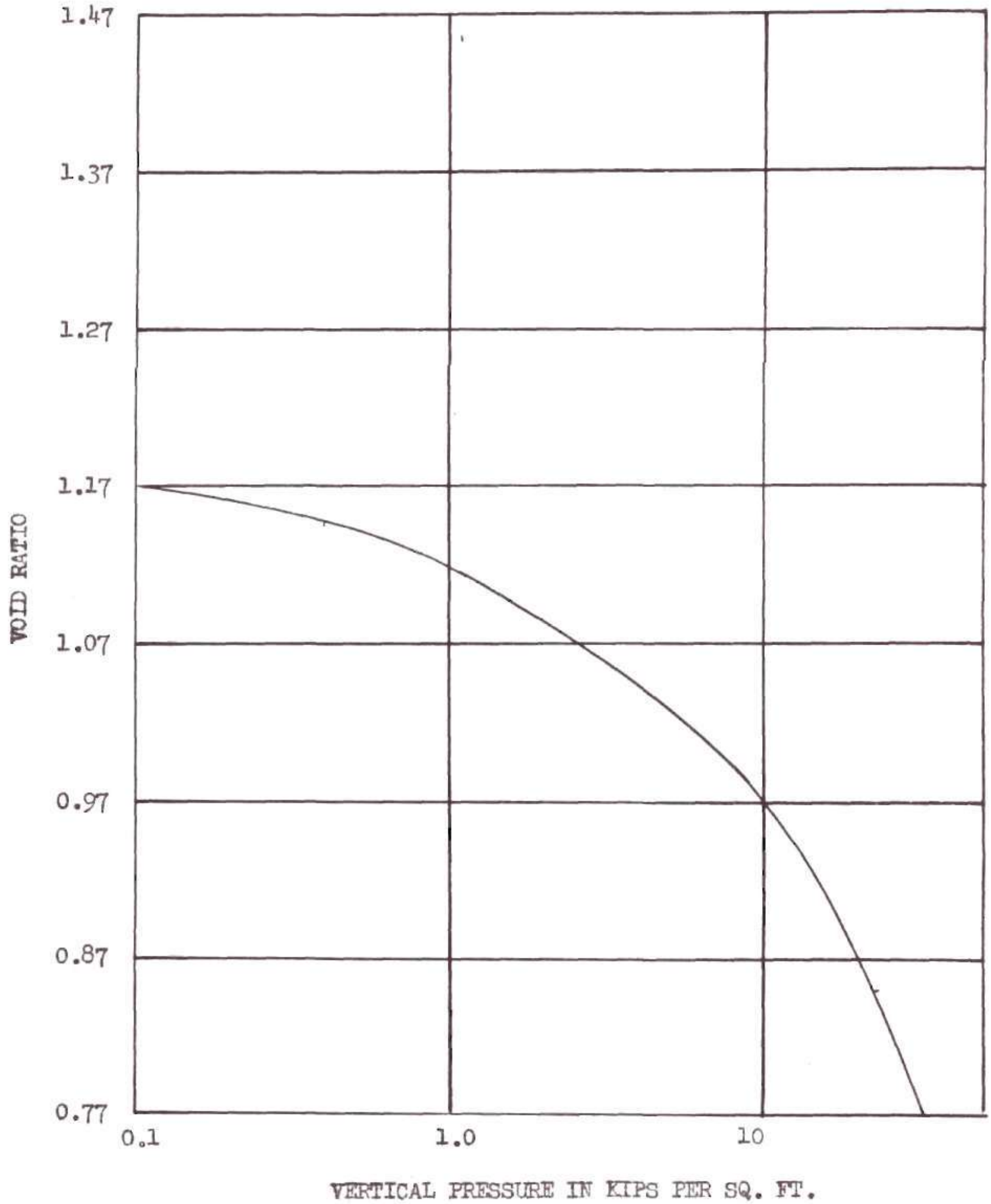


Fig. 22 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 6 FT., BORINGS 3 AND 5

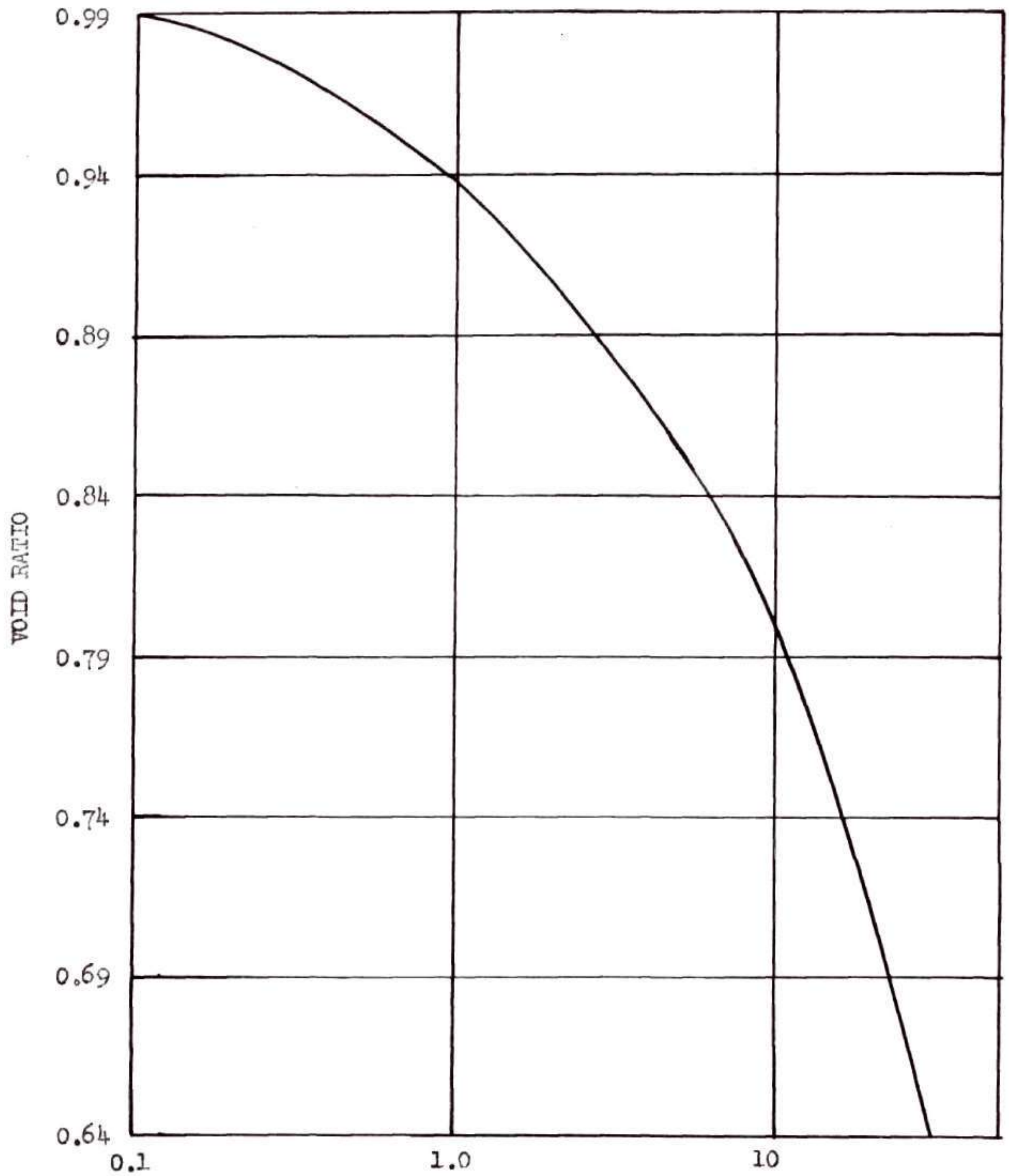


Fig. 23 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 6 FT., BORINGS 2 AND 4

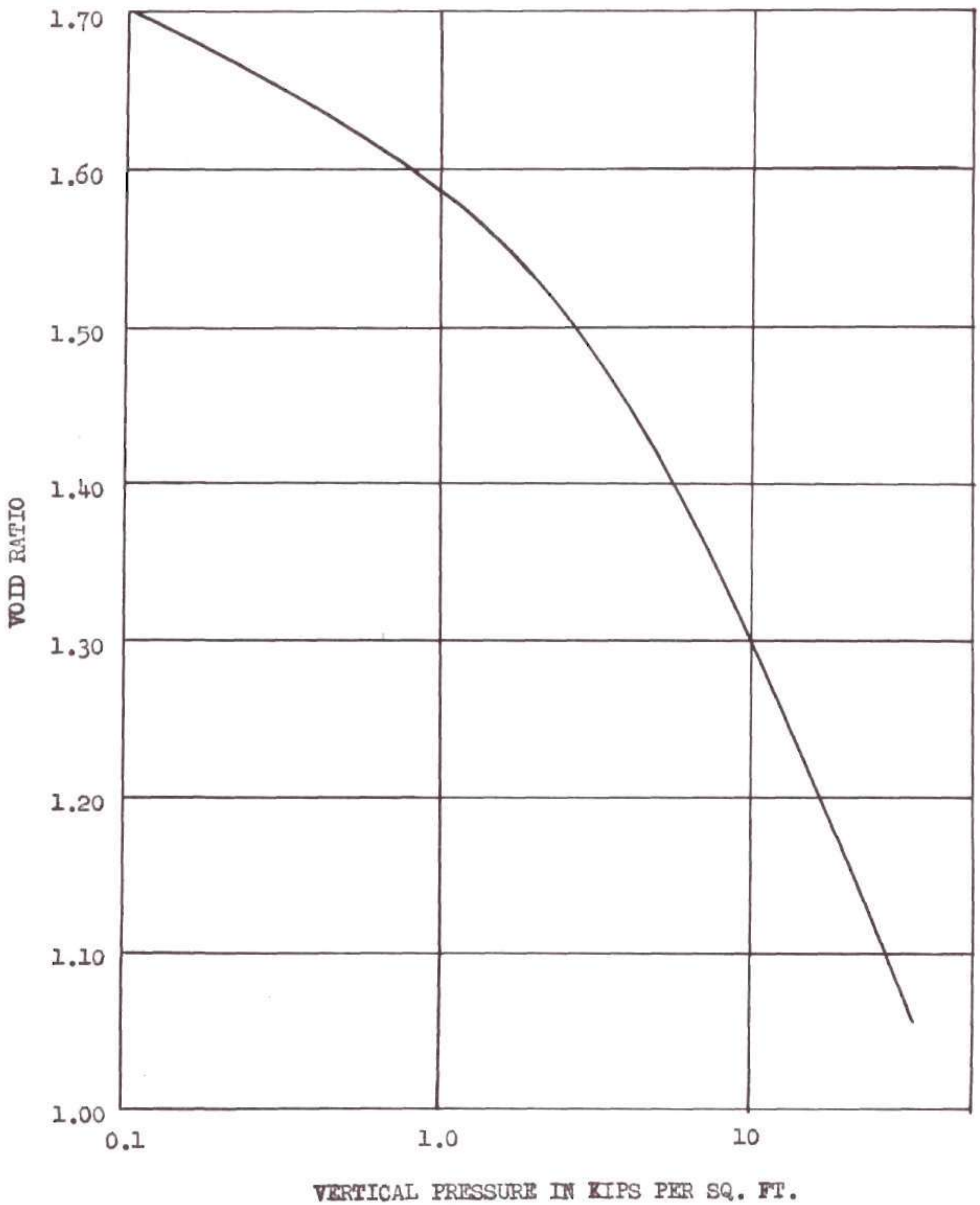


Fig. 24 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 8.5 FT., BORINGS 3 AND 5

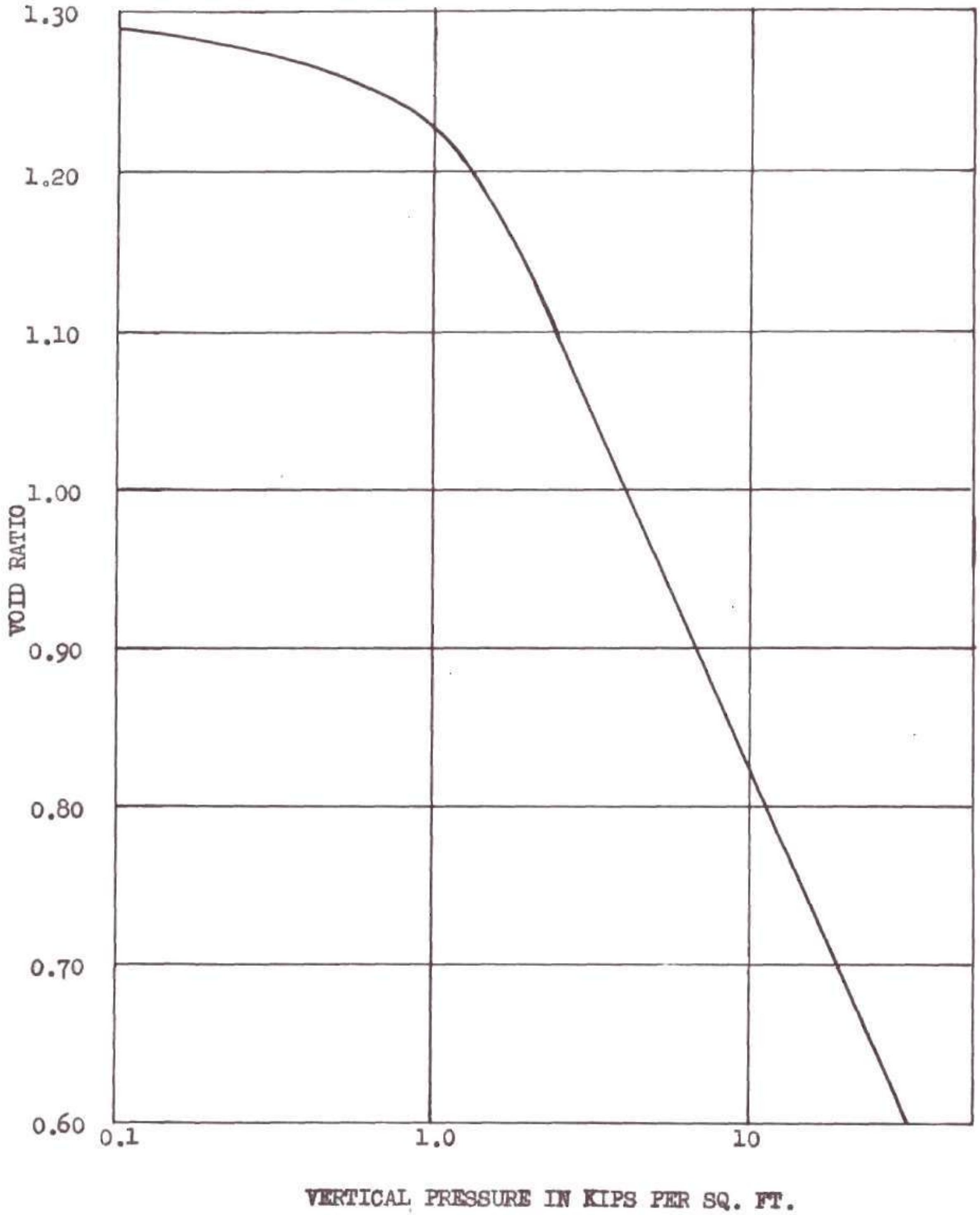


Fig. 25 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 8.5 FT., BORINGS 2 AND 4

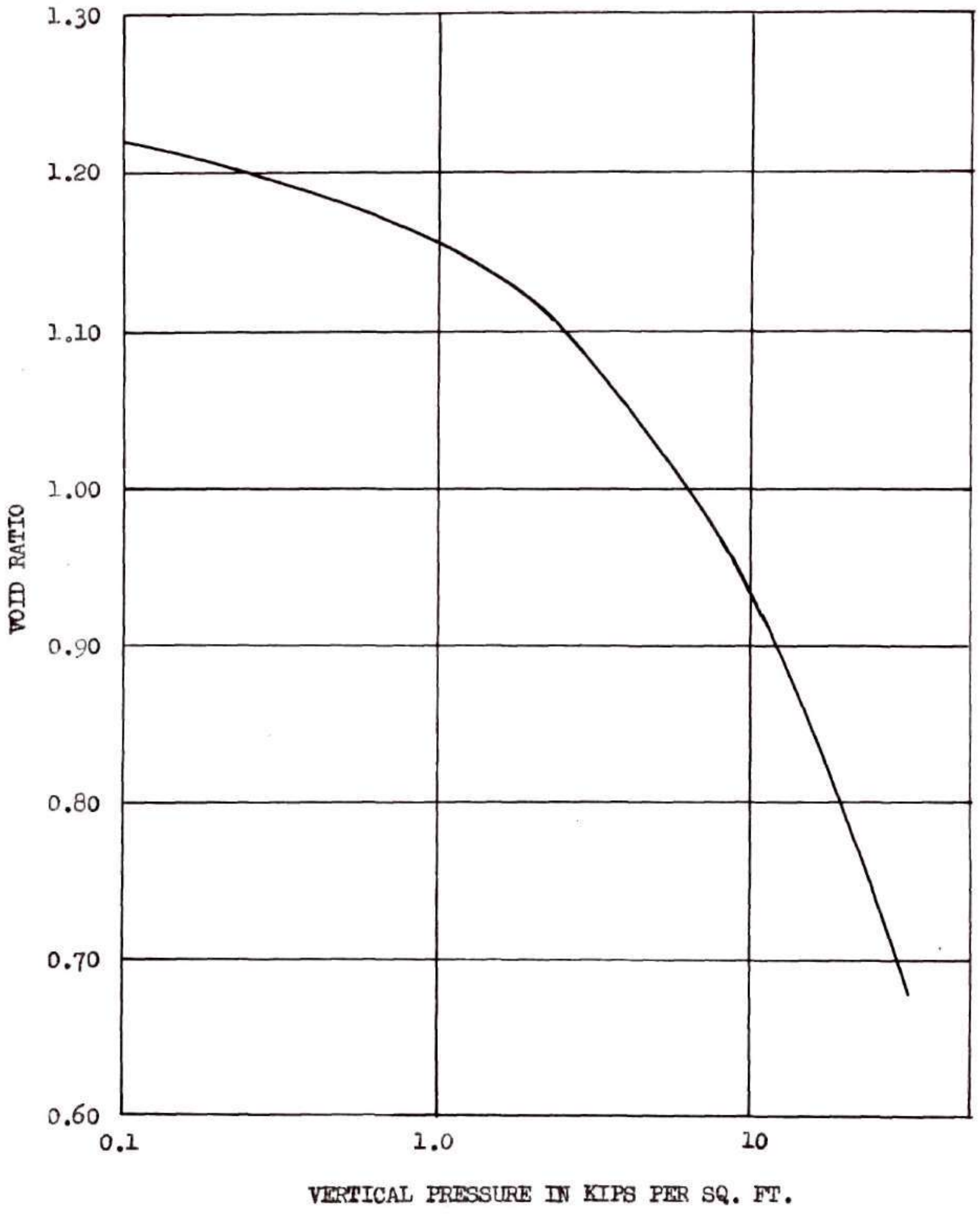


Fig. 26 VERTICAL PRESSURE VS. VOID RATIO, DEPTH, 11 FT.

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