1965

Field permeability determination of partially saturated fine grained soil

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FIELD PERMEABILITY DETERMINATION OF PARTIALLY SATURATED FINE GRAINED SOIL

BY

EDWARD G. erard RAPP, 1937

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THESIS

submitted to the faculty of the

UNIVERSITY OF MISSOURI AT ROLLA

in partial fulfillment of the requirements for the

Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

Rolla, Missouri

1965

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ABSTRACT

The determination of water permeability through a soil is necessary to estimate seepage losses from small reservoirs. At the present time, the methods available to conduct this determination in partially saturated fine grained soils are expensive and in most cases not commensurate with the cost or importance of the individual structure. This investigation was conducted to evaluate the use of a proposed, potentially inexpensive field procedure in this situation.

A modified borehole apparatus was utilized in field tests to determine the permeability of a partially saturated soil deposit in place. Undisturbed samples from the same location were tested in the laboratory by several recommended procedures. It was found that the range of permeability values obtained in the field correlated closely to the range of values obtained by detailed laboratory testing.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation for the utmost cooperation afforded this investigation by the Missouri Geological Survey, the Missouri Conservation Commission, the Missouri Highway Department, and the United States Forest Service. The writer is particularly indebted to Mr. James H. Williams and Mr. James Martin of the Missouri Geological Survey for their aid and assistance throughout this investigation.

The author further appreciates the efforts made by the staffs of both the Civil Engineering Department and the Ceramics Department of the University of Missouri at Rolla for their assistance in the construction of test apparatus. Finally the author sincerely appreciates the constructive criticism given by Dr. Thomas S. Fry and Professor John B. Heagler, Jr. during the preparation of this thesis.

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I. INTRODUCTION

An engineering evaluation of permeability is necessary for the design of every structure where seepage of water or seepage forces could have an adverse effect on the structure. The principle engineering problems involving permeability include seepage from reservoirs and canals, dewatering of excavations, drainage of roads and airfields and stability of embankment slopes. In the State of Missouri, approximately 2 million dollars are spent annually for the investigation and construction of small reservoirs having impoundment areas in the order of 10 to 100 acres.(1) Further, approximately one-third of the small reservoirs constructed on the predominately residual soils of the southern one-half of the state do not reach their design pool elevation due to unanticipated seepage losses. In many cases, these reservoirs fail to reach a usable pool elevation.

These failures reflect primarily the fact that the laboratory and field methods currently available and generally accepted for evaluating permeability, require expense which in many cases is not commensurate with the size and importance of these individual structures. The U. S. Bureau of Reclamation recommends the use of average values of permeability based on soil classification for investigations where the cost of present permeability testing procedures cannot be justified. The use of average values for estimating seepage losses from small reservoirs does not appear to be appropriate if economical testing procedures can be devised. This investigation was conducted to evaluate a proposed, potentially inexpensive field method for determining permeability in a fine grained, partially saturated soil.

II. REVIEW OF PERMEABILITY

A. Permeability and Darcy's Law

In soils engineering, permeability is the measure of water conductivity through a soil media. Darcy's Law establishes a relationship between the velocity of saturated, laminar flow and the gravity potential causing the flow. This relationship is:

$v = ki$ where: v = superficial velocity

i = hydraulic gradient

k = coefficient of permeability

Since permeability is not only a function of the soil, but also the unit weight and viscosity of the water, it is necessary to establish unit conditions of control so that the coefficient of permeability reflects the properties of the soil alone. Since both the density and viscosity of water are a function of temperature, a control temperature of $20^{\circ}C$ has been generally accepted. Thus the coefficient of permeability (k_{20}) reflects empirically a property of the soil alone.

It has been found experimentally by Richards(4) and others, that a form of Darcy's Law is applicable to laminar flow through unsaturated soil and can be expressed as:

$v = k_u i$ where: v = superficial velocity

i = hydraulic gradient

k_u = unsaturated coefficient of permeability

In this equation, k_u is not constant but a function of the degree of saturation as indicated in Figure 1.(5) The stages of moisture

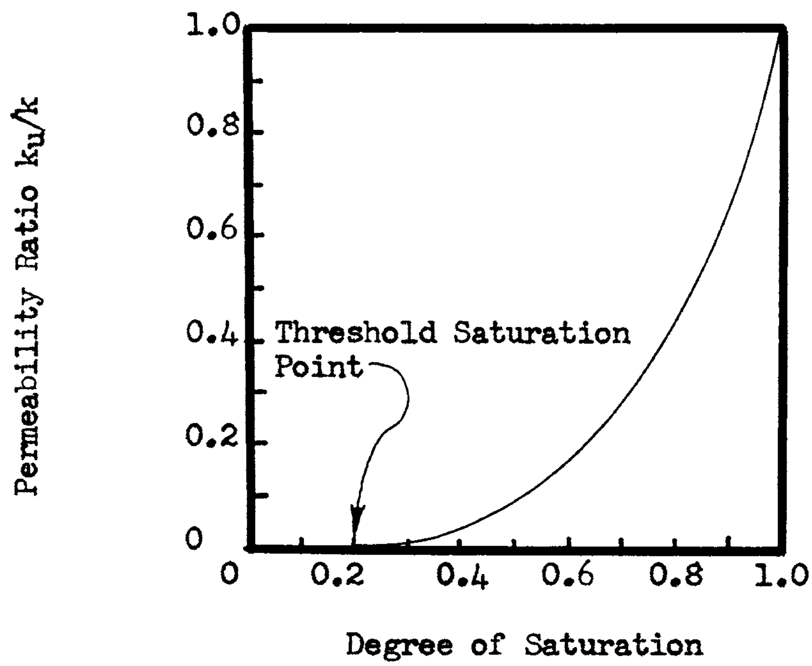


FIGURE 1. RELATIONSHIP BETWEEN DEGREE OF SATURATION AND PERMEABILITY RATIO after Irmay[6]

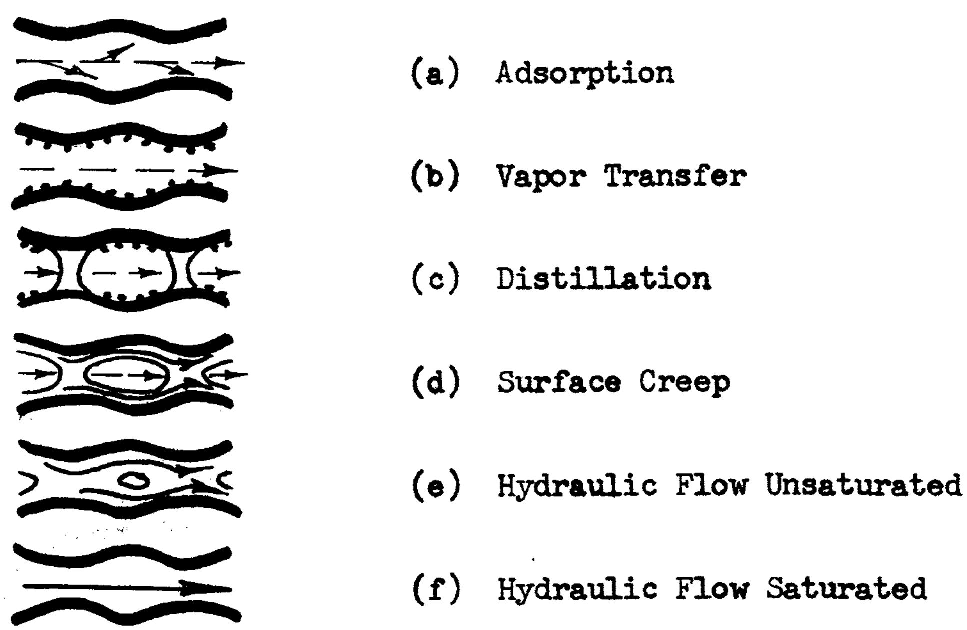


FIGURE 2. WATER MOVEMENT IN THE VARIOUS STAGES IN A POROUS MATERIAL after Rose[6]

transmission through soil from the zero saturated point to the saturated condition have been identified by Rose[6] and are illustrated in Figure 2.

Water transmission through an unsaturated soil is effected by gravity, electrical and chemical forces; thermal gradients; vapor pressure; and/or capillary influences.[7] In the evaluation of seepage flow from small reservoirs, gravity and capillary potentials are of primary concern. In saturated soils where there are no air-water interfaces, capillary forces do not exist and seepage takes place primarily due to gravity.

The evaluation of permeability involves the determination of superficial velocity under a gravity potential. In partially saturated soils, the problem is far more complex since the capillary forces play an important role. The evaluation of the total energy potential causing flow under these conditions is not fully understood. Hence, in order to evaluate the coefficient of permeability in an unsaturated soil, it is necessary for the soil to become saturated and for the flow rate to be constant in the area which is influenced by the test.

B. Permeability in Natural Soil Deposits

In natural soil deposits, the coefficient of permeability, reported by many investigators, ranges from 10 cm/sec in gravel to 10^{-9} cm/sec in clay. Qualitatively, soils with permeability values greater than 10^{-4} cm/sec are generally termed as pervious; those with values in the range of 10^{-4} to 10^{-6} cm/sec are semipervious; and those less than 10^{-6} cm/sec are impervious. It has been recognized by many investigators that the coefficient of permeability of soil may vary greatly with minor changes in texture, density, mineralogical composition, structure, and water content. As a result, the engineering indices, void ratio, plasticity and degree saturation, which reflect the condition of the pores for conducting water, are the principle variables of soil permeability. Since these properties vary from place to place within a deposit, rather large variations in the coefficient of permeability can also be expected.

Particle orientation, due to the mode of deposition of the soil and soil structure, gives rise to permeabilities varying in different directions. For instance, in alluvial soils, permeabilities in the horizontal direction may be many times greater than in the vertical direction. Along the same line, fracturing and fissuring in many fine grained soils can give rise to very large permeabilities through these deposits.

Due to the possible variations from place to place within a soil deposit, determination of a single average value of permeability for the deposit is extremely difficult. Therefore, even though the permeability of an individual sample may be determined very accurately, the value of permeability representative of a particular deposit is generally considered as an order of magnitude rather than a specific value. For most practical engineering problems, it is desirable to establish the range of permeability values within the deposit. The value representative of the deposit as a whole is generally limited in accuracy to the nearest power of 10.[8]

C. Evaluation of Permeability

In the laboratory, permeability can be evaluated utilizing Darcy's Law, consolidation theory, capillary theory or empirical equations based on various index properties of the soil. In the field, permeability is evaluated in place, primarily by use of Darcy's Law. Laboratory tests procedures and their range of applicability are outlined in detail in laboratory manuals and in most soil mechanics texts,(9)(10) and will not be discussed here. However, the inherent problems common to all laboratory evaluations of permeability in natural soil deposits are worthy of mentioning. Representative undisturbed samples must be obtained from the field in a manner to preserve the structure and properties of the soil. In many soils this is extremely difficult to accomplish. The problems in this area have been discussed extensively by Hvorslev.(11) Secondly, the conditions of the test must approximate the conditions that will exist in the field. This may be difficult to accomplish in the laboratory. Finally, the number of samples tested, no matter how accurate the individual results, must be sufficient to establish the permeability range of the deposit. This generally requires extensive testing, the cost of which cannot be justified in many instances. The wide variations within a deposit indicate that a field evaluation of permeability through large volumes of soil is desirable provided that the cost of such testing can be justified.

Numerous field methods for determining permeability have been proposed. Of these, the methods developed and tested by Kirkham and others(12)(13) appear to be most applicable for the investigation of small dam sites. These methods measure the loss or increase in head in a single borehole when water is pumped out of or into the hole at a measured flow rate. From these measurements permeability can be evaluated using an empirical equation based on Darcy's Law modified to include the boundary conditions of the test. The general approach of this method can be used in measuring permeability in partially saturated soils provided that time is allowed for the soil to become saturated by infiltration in the vicinity effected by the test.[14][15] The permeability can be computed when a constant flow rate is established through this wetted zone. The procedure described in U. S. Bureau of Reclamation test designation E-18[15] appears to have the widest applicability for the investigation of small dam sites. The basic concepts of this empirical method are illustrated in Figure 3.

In the equations for constant head or falling head laboratory permeameters, which are derived directly from Darcy's Law, a shape factor L/A appears, where L = the length of the flow path corresponding to the head loss and A = the area through which the flow is taking place. It has been found by Harza[16] and Taylor[17] employing different methods, that the shape factor for saturated flow in the situation depicted in Figure 3, is equal to 1/5.5r, where r is the radius of the tube. Shape factors for other boundary conditions have been determined by numerous investigators and are summarized in a paper by Hvorslev.[18]

Utilizing these shape factors, it is possible to estimate permeability in the area of the cavity once a saturated envelope is established and flow into the cavity is constant. The coefficient of permeability measured by these methods is a mean value of the

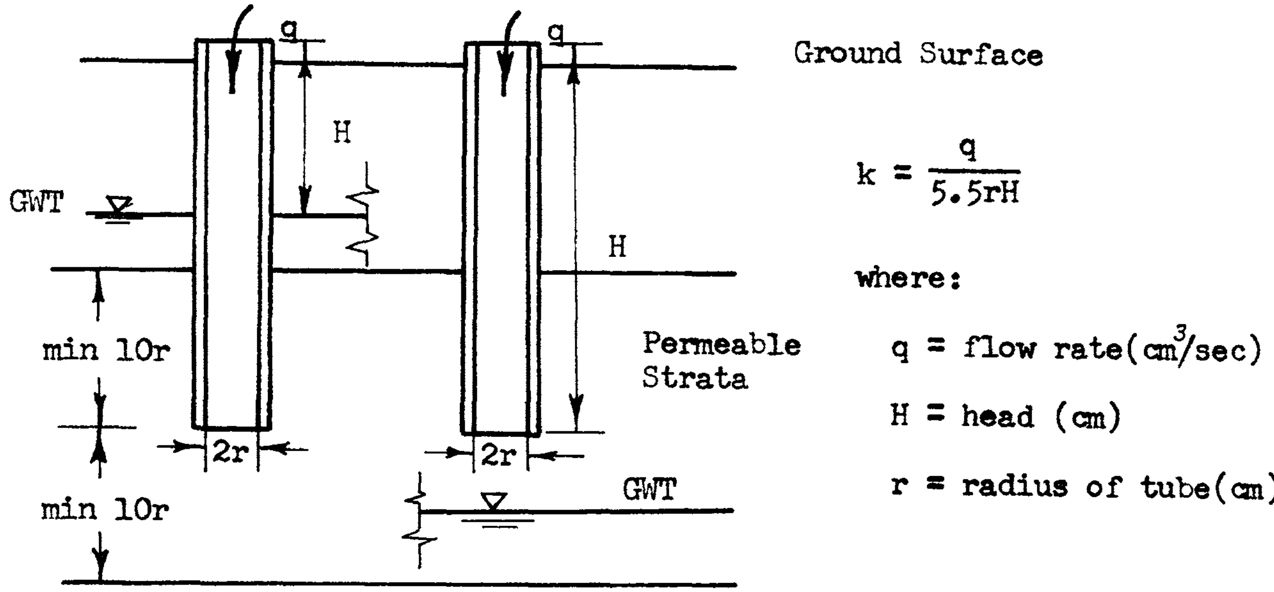


FIGURE 3. OPEN END BOREHOLE TEST
after USBR Test Designation E-18[24]

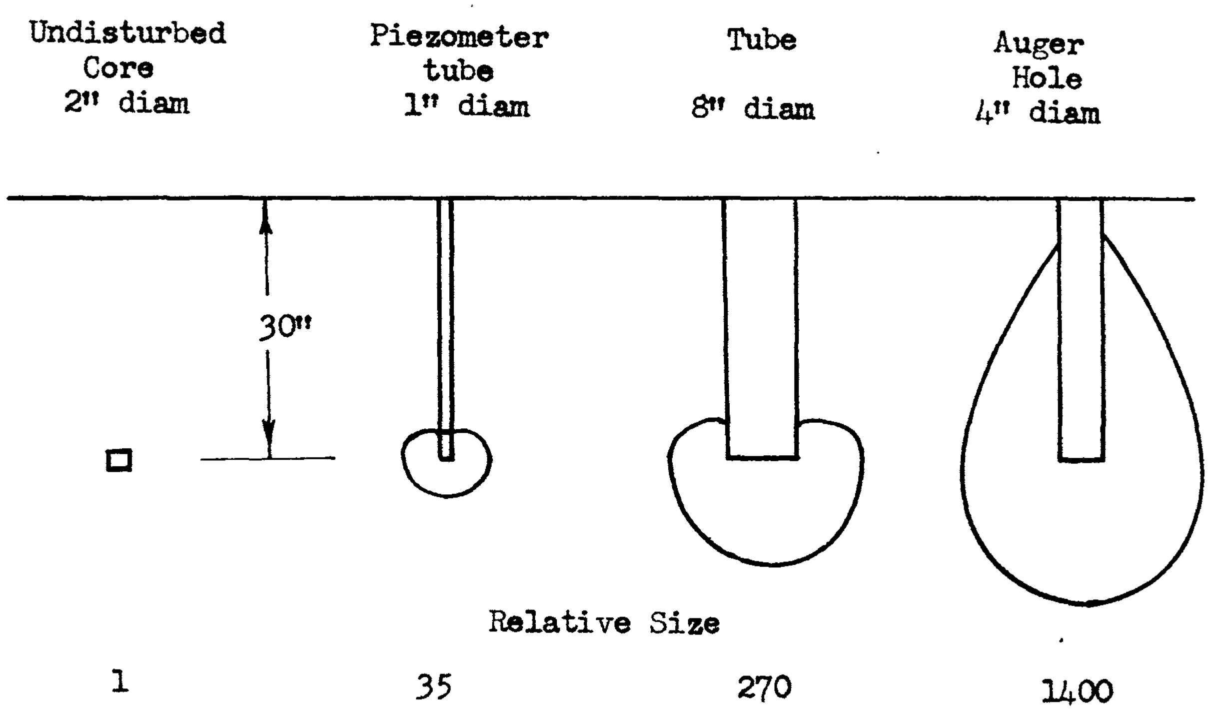
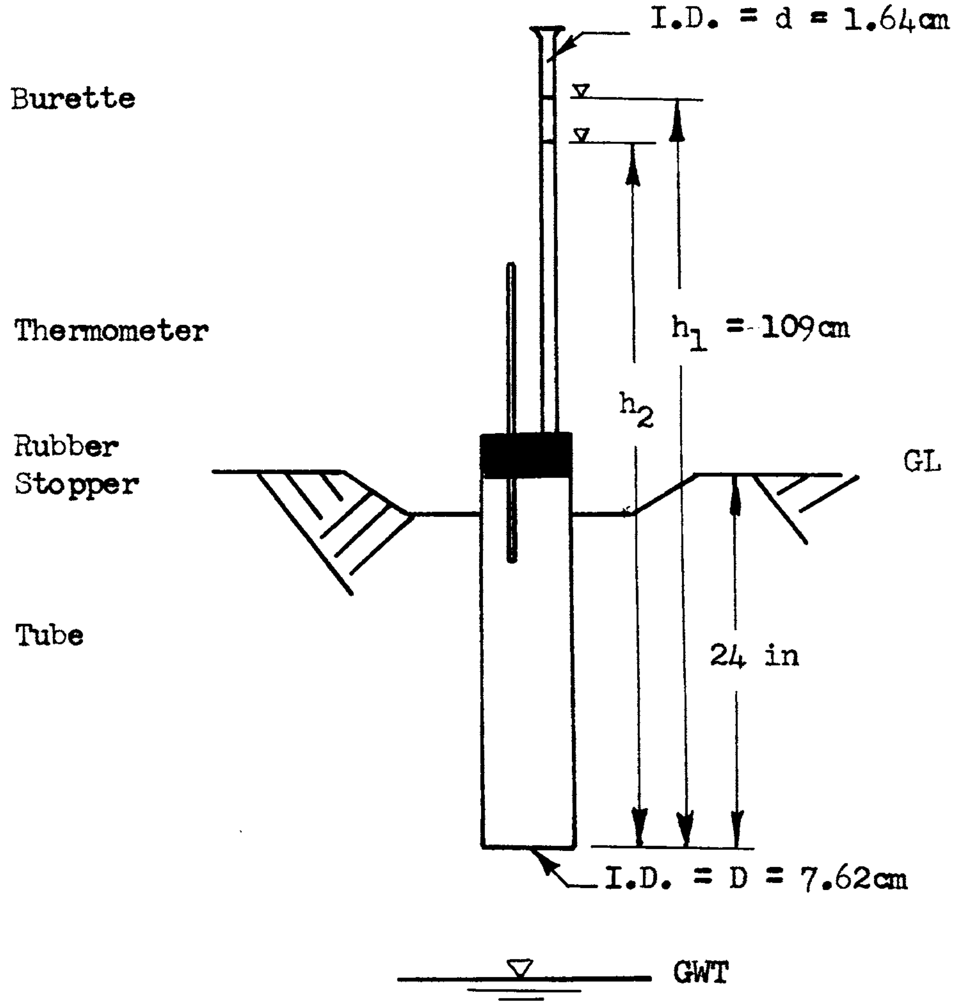


FIGURE 4. RELATIVE SIZE OF SAMPLE
after Reeve and Kirkham[29]

horizontal and vertical permeabilities. However, according to Reeve and Kirkham,(19) horizontal permeability dominates the test results in uncased holes whereas vertical permeabilities dominate the results in cased holes. The relative volumes of soil tested by various single cavity methods have been computed by these two investigators and are shown in Figure 4.

These methods, although crude in nature, give better estimates of the overall permeability than do isolated laboratory tests.(20) However, according to Goulder and Gass,(21) borehole methods are generally limited to the measurement of permeability in the range of 10^{-1} to 10^{-4} cm/sec because of the difficulties in metering flow and the time required to perform the tests. Finally in their opinion refinements in the method in order to extend the range are not justified for normal operations.

Despite this opinion, it was desirable to pursue this approach further in hopes of finding a solution to the original problem. For this reason, an adaption to the apparatus used in U. S. Bureau of Reclamation test designation E-18 was made in order that lower flow rates could be observed. See Figure 5. A field and laboratory investigation was then conducted to evaluate the use of this modified apparatus in fine grained unsaturated soils.



$$k = \frac{d^2}{11D\,\Delta T}\ \ln \frac{h_1}{h_2}$$

where: ΔT = time interval corresponding to the head loss; D, d, h_1 and h_2 are as shown.

FIGURE 5. BOREHOLE APPARATUS ADAPTED FOR FALLING HEAD

III. INVESTIGATION

A. Conduct of the Investigation

In order to evaluate the falling head adaptation to the bore-hole test procedure in a fine grained unsaturated soil, a test site was selected in the field. Utilizing this apparatus, the inplace permeability was evaluated at control locations within a small area. Undisturbed samples for laboratory testing were taken at the same locations. Finally all of the test results were compared and analyzed.

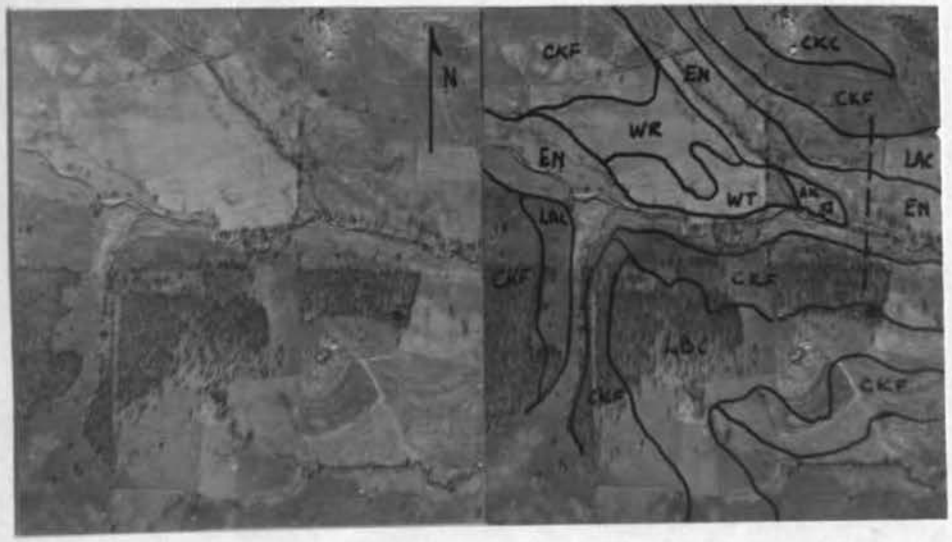
B. Description of the Site

The test site is located in the SE $\frac{1}{4}$, Sec. 21, T38N, R7W. A stereopair of this area appears in Figure 6. The pedological soil boundaries indicated on this stereopair were delineated by Moriarity and Martin.[22]

The test series was performed in the deposit mapped as Atkins, which is a poorly drained silt loam. Boring logs made previously at the site by the Missouri Conservation Commission, indicate that this deposit is an inorganic silt of low plasticity and overlies bedrock at a depth of approximately 10 feet. The bedrock in the area is Jefferson City dolomite.[23] The borings by the Conservation Commission were made in September, 1964. At that time free groundwater was not encountered in any of the borings.

C. Field Procedure

A triangular grid was established so that all the test locations would be equally spaced within the test area. An arbitrary spacing of ten feet was selected to minimize the effects of lateral soil changes within the deposit. The possibility of hydraulic interference between holes during the testing period was also eliminated



LEGEND

CKF	Clarksville Cherty Silt Loam 14-50% slopes	
WR	Wolftever Silt Loam 2-4% slopes	
CKC	Clarksville Cherty Silt Loam 0-13% slopes	
LAC	Landisburg Silt Loam 5-8% slopes	
WT	Westerville Silt Loam	
AK	Atkins Silt Loam	
EN	Ennis Silt Loam	
LBC	Lebanon Silt Loam	
◁	Test Site Location	
--	Centerline, Proposed Dam	

FIGURE 6. STEROPAIR OF THE TEST SITE, Pedogical Soil Boundaries delineated by Moriarity and Martin(22)

by this spacing. The A and B horizons of the soil were removed at each test location. A 3 inch shelby tube was pressed into the soil by a hydraulic ram to a depth of approximately 24 inches below the original ground level to obtain an undisturbed sample for laboratory testing. After removal of the shelby tube, a cylindrical steel tube of the same diameter was placed tightly into the hole, thereby casing the sides of the hole to the bottom.

After the casing was in place, the hole was filled with clean water and fitted with the falling head device shown in Figure 5. Care was exercised to insure that the temperature of the added water was greater than the temperature of the soil, thus precluding the release of gases from the water to the soil. After the initial filling, the tube was not allowed to drain completely.

Periodically, falling head measurements were made over a considerable period of time until consistent readings were obtained. When consistent readings were obtained, it was assumed that the saturated envelope had been established in the area of the test. Permeability was computed based on these consistent readings. Finally at the completion of a field permeability test, a one inch diameter sampling probe was pushed into the saturated soils at the bottom of the hole. The condition and character of these soils were visually examined from the probe core.

D. <u>Laboratory Procedure</u>

The undisturbed samples were transported to the laboratory where the tube was cut and the sample trimmed as shown in Figure 7. The soil trimmings from section 1 and the undisturbed material from section 2 were used to determine the dry unit weight, grain size

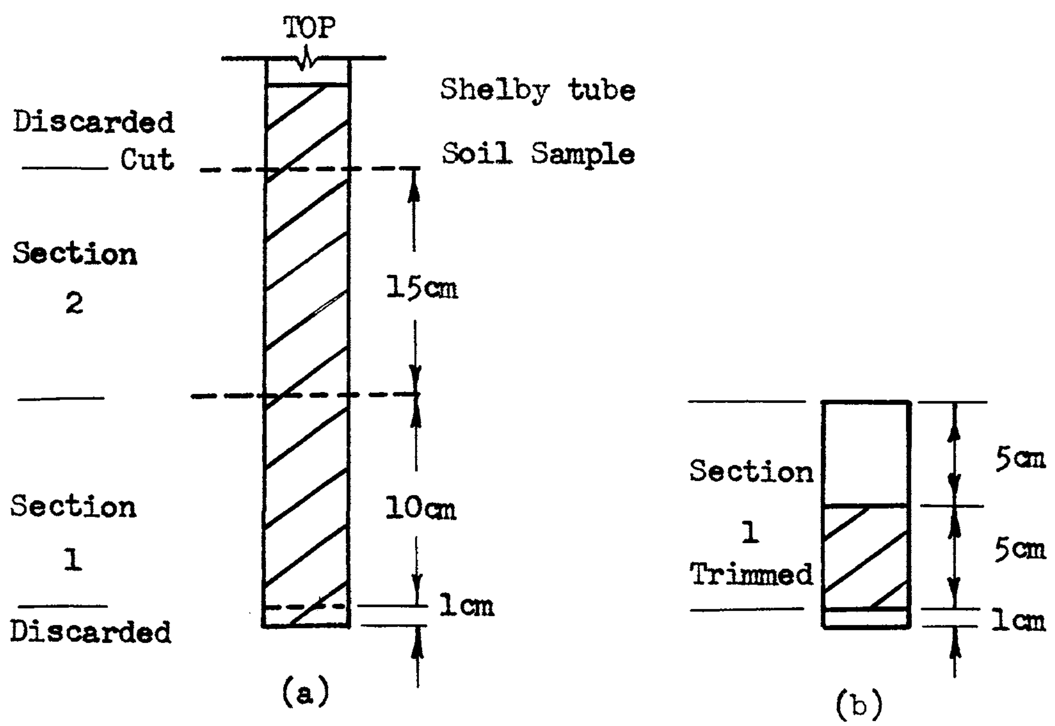
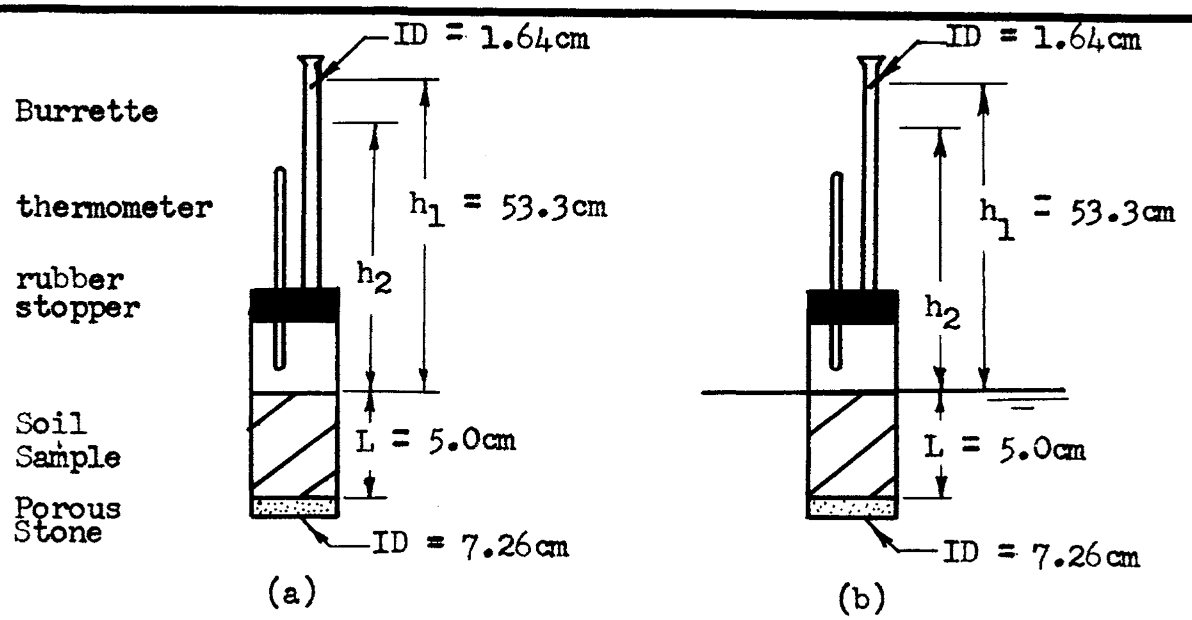


FIGURE 7. PREPARATION OF UNDISTURBED TUBE SAMPLE



INFILTRATION TEST

FALLING HEAD TEST

$$k = \frac{d^2 L}{D^2 \Delta T} \ln \frac{h_1}{h_2}$$

where: ΔT = the time corresponding to the head loss
d, D, L, h_1 & h_2 are as shown

FIGURE 8. INFILTRATION AND FALLING HEAD TEST APPARATUS

distribution, specific gravity and water content in the natural state. The prepared test specimen, section 1, was fitted with the same falling head adaptation as was used in the field. (See Figure 8a). The apparatus was then filled with water and falling head readings were taken in the same manner as in the field. After several days, stable readings were obtained and the permeability was then evaluated from a formula based on Darcy's Law. The sample with the apparatus attached was then placed in water as shown in Figure 8b. After sufficient time had been allowed for the sample to attain stress equilibrium, a falling head permeability test was performed.

Finally, the sample from the test section was extruded and carved into a consolidation ring. A consolidation test was performed and permeability was computed on the basis of the classical theory of consolidation using data from the dial reading time curves.

IV. RESULTS

The results of the field and laboratory testing have been compiled for analysis and are shown graphically in Figures 9 and 10. Results that lend themselves to tabulation are contained in Tables 1 through 6.

Table 1 lists the index characteristics for the samples from each borehole location. The range of index properties for the test site are shown graphically in Figures 9 and 10. Table 2 lists the basic relationships of the soil at each location. Natural water content and degree of saturation were not included in this tabulation since these values varied considerably during the period of testing. The maximum natural water content measured was 20.8 percent and minimum was 4.6 percent. The coefficient of permeability and the corresponding void ratios measured in each of the four testing methods are listed in Tables 3 through 6 and are shown graphically in Figure 11.

TABLE 1. PHYSICAL CHARACTERISTICS

Sample No.	Liquid Limit	Plastic Limit	Plastic Index	%Passing #200	Unified Classification
1-1	23.9	18.0	5.9	88	CL-ML
1-3	33.5	17.2	16.3	93	CL
1-4	29.9	20.4	9.5	91	CL
1-5	30.6	19.2	11.4	92	CL
1-6	31.6	17.8	13.8	93	CL
1-7	31.5	18.7	11.8	94	CL
1-8	23.3	19.4	3.9	88	ML
1-11	24.3	19.7	4.6	88	CL-ML
1-12	21.8	17.5	4.3	89	CL-ML
1-13	21.8	18.8	3.0	88	ML

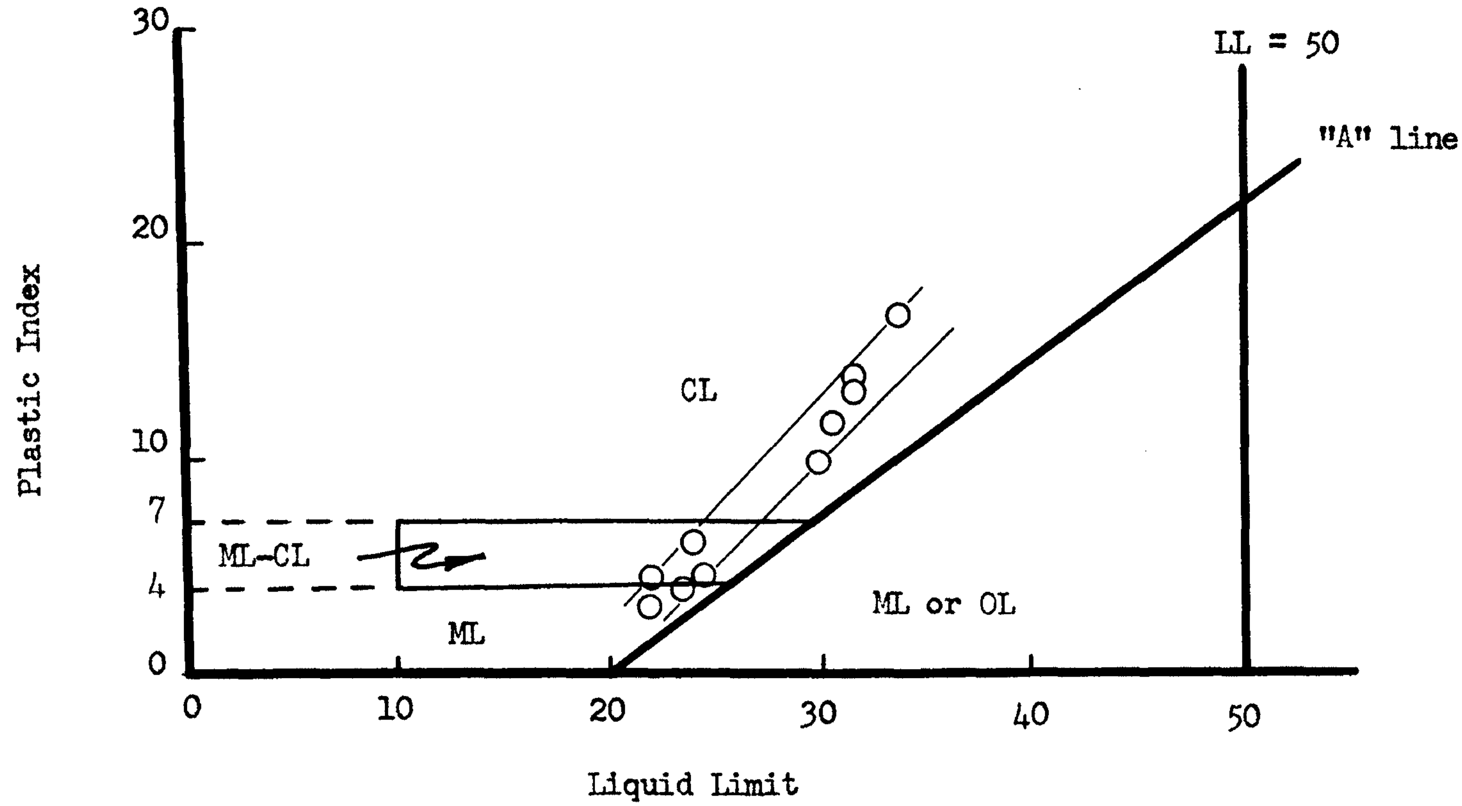


FIGURE 9. RELATIONSHIP BETWEEN LIQUID LIMIT AND PLASTICITY INDEX

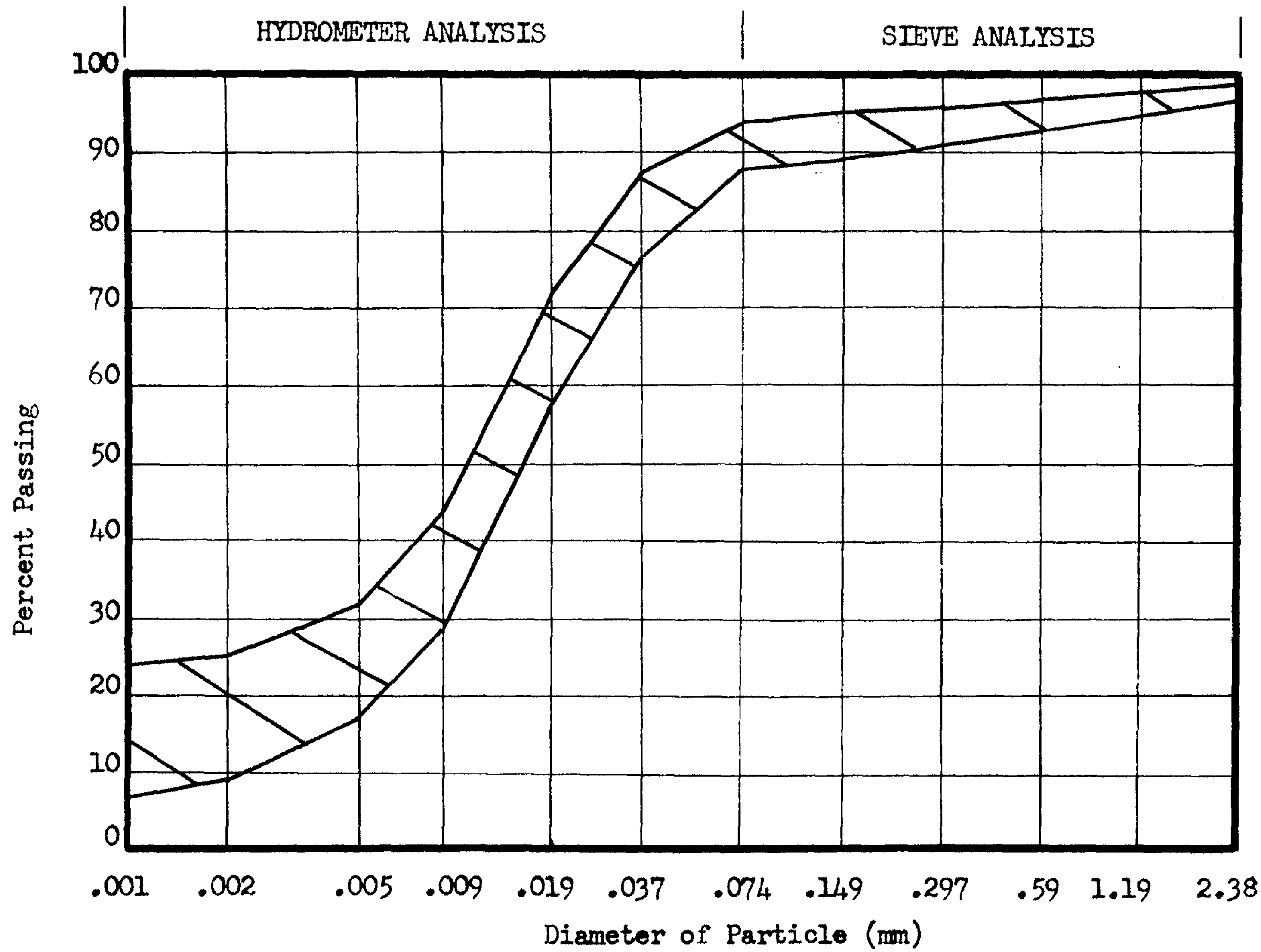


FIGURE 10. RANGE OF GRAIN SIZE DISTRIBUTION

TABLE 2

PROPERTIES OF UNDISTURBED SAMPLES

SAMPLE NO.	DRY UNIT WEIGHT lb/ft^3	SPECIFIC GRAVITY OF SOLIDS	VOID RATIO
1-1	104.5	2.61	.557
1-2	102.0	2.63	.610
1-3	97.0	2.63	.680
1-4	111.5	2.68	.500
1-5	107.0	2.68	.562
1-6	112.5	2.68	.490
1-7	106.0	2.66	.564
1-8	101.0	2.65	.635
1-9	102.5	2.63	.600
1-11	96.5	2.61	.690
1-12	97.5	2.62	.675
1-13	98.0	2.63	.675

TABLE 3

RESULTS FROM FIELD TESTS

SAMPLE NO.	VOID RATIO	PERMEABILITY cm/sec
1-1	.560	3.6×10^{-7}
1-3	.690	1.6×10^{-4}
1-4	.500	8.2×10^{-6}
1-5	.570	1.5×10^{-6}
1-6	.485	1.1×10^{-6}
1-7	.570	* 5.0×10^{-8}
1-8	.635	1.8×10^{-4}
1-11	.690	8.8×10^{-7}
1-12	.675	6.2×10^{-4}
1-13	.675	Seal Broke

*Flow rate too small to be read accurately.

TABLE 4

RESULTS FROM INFILTRATION TESTS
(LABORATORY PROCEDURE)

SAMPLE NO.	VOID RATIO	PERMEABILITY cm/sec
1-1	.593	3.6×10^{-6}
1-3	.680	4.2×10^{-5}
1-4	.719	1.2×10^{-6}
1-5	.700	7.7×10^{-7}
1-6	.841	3.5×10^{-5}
1-7	.747	1.0×10^{-5}
1-8	.636	4.6×10^{-5}
1-11	.649	1.4×10^{-5}
1-12	.677	1.7×10^{-5}
1-13	.676	2.5×10^{-4}

TABLE 5

RESULTS FROM FALLING HEAD TESTS
(LABORATORY PROCEDURE)

SAMPLE NO.	VOID RATIO	PERMEABILITY cm/sec
1-1	.593	$3.0x10^{-6}$
1-3	.680	$4.2x10^{-5}$
1-4	.719	$2.1x10^{-6}$
1-5	.700	$9.7x10^{-7}$
1-6	.841	$2.5x10^{-5}$
1-7	.747	$1.0x10^{-5}$
1-8	.636	$4.3x10^{-5}$
1-11	.649	$1.3x10^{-5}$
1-12	.677	$1.7x10^{-5}$

TABLE 6

RESULTS FROM CONSOLIDATION TESTS
(LABORATORY PROCEDURE)

SAMPLE NO.	AVERAGE VOID RATIO	PERMEABILITY cm/sec
1-1	could not be trimmed into a ring	
1-3	.670	9.4×10^{-6}
	.653	6.5×10^{-6}
	.636	5.3×10^{-6}
	.617	4.8×10^{-6}
	.596	3.6×10^{-6}
1-4	.717	1.2×10^{-5}
	.702	1.7×10^{-5}
	.669	1.4×10^{-5}
	.640	6.6×10^{-6}
1-5	.696	8.9×10^{-6}
	.681	1.6×10^{-5}
	.660	1.7×10^{-5}
	.647	1.0×10^{-5}
	.624	7.2×10^{-6}
1-6	.831	4.9×10^{-5}
	.807	8.6×10^{-5}
	.780	3.9×10^{-5}
	.755	2.6×10^{-5}
	.732	1.8×10^{-5}

(Continued)

TABLE 6 (Concluded)

RESULTS OF CONSOLIDATION TESTS CONTINUED

SAMPLE NO.	AVERAGE VOID RATIO	PERMEABILITY cm/sec
1-7	.740	$5.6x10^{-5}$
	.728	$3.4x10^{-5}$
	.714	$4.9x10^{-5}$
	.698	$2.5x10^{-5}$
	.679	$1.8x10^{-5}$
1-8	could not be trimmed into a ring	
1-11	.646	$5.3x10^{-5}$
	.638	$1.4x10^{-4}$
	.630	$5.5x10^{-5}$
	.624	$7.6x10^{-5}$
	.616	$4.9x10^{-5}$
	.608	$2.5x10^{-5}$
1-12	.671	$5.1x10^{-4}$
	.661	$1.7x10^{-4}$
	.655	$6.7x10^{-5}$
	.652	$1.4x10^{-5}$

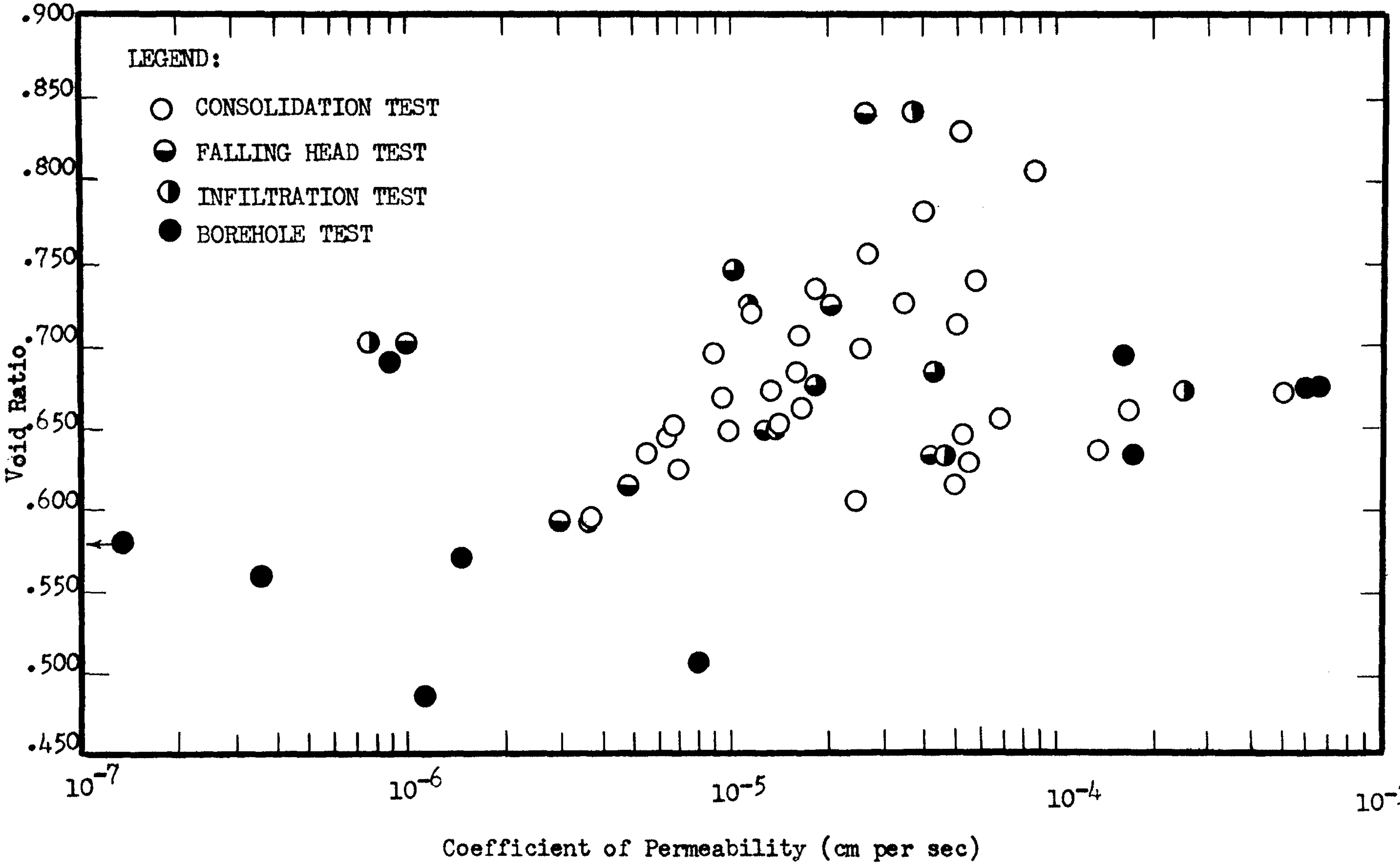


FIGURE 11. RELATIONSHIP BETWEEN VOID RATIO AND PERMEABILITY

V. DISCUSSION

A. Analysis of Results

In order to properly evaluate and compare the results obtained, it is necessary to visualize the significant changes that occurred within the samples, both in the laboratory and in the field as permeation progressed.

Permeation of an unsaturated soil results in the increase in the degree of saturation. This increase in moisture content in addition to the increased hydrostatic pressure of the test, results in complex changes in the stress relationships within the sample. These changes are not completely understood; however, their existence is evidenced by either a swelling or compression of the soil which indicates change in effective stress. These changes in volume within the samples due to permeation were measured in all the laboratory tests and are shown graphically in Figure 12.

The void ratios corresponding to the coefficient of permeability, obtained from laboratory testing, shown in Figure 11, are not the same as the void ratios that existed in the field. In order to compare the laboratory data with the field data, it is necessary to plot void ratio-permeability curves for each sample and from this relationship determine the permeability of the sample that corresponds to the initial void ratio in the field.

According to Lambe,[24] the void ratio-permeability relationship approximates a straight line. Hence, from the void ratio-permeability data obtained from the consolidation test, it is possible to estimate the permeability corresponding to the void ratio in the field. On this basis, the laboratory values can be compared to those obtained

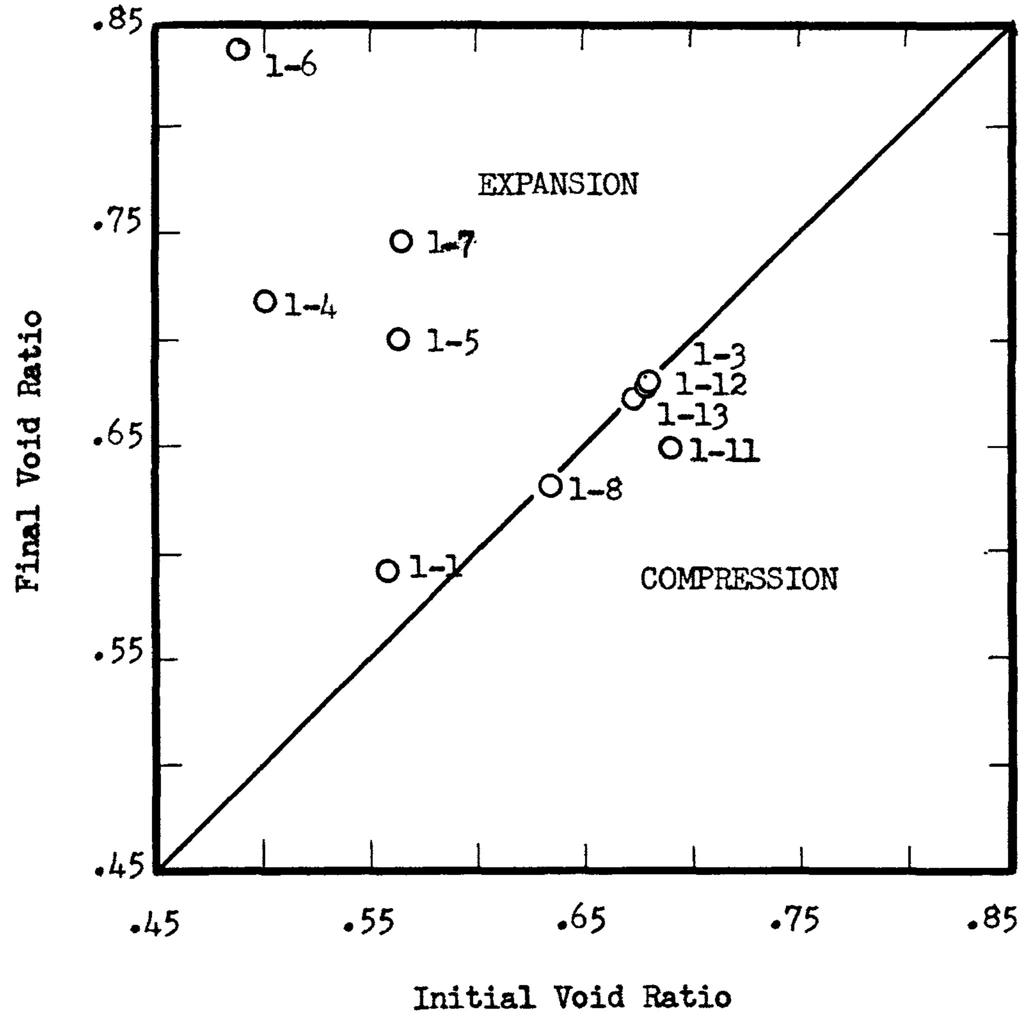


FIGURE 12. VOID RATIO CHANGE DUE TO PERMEATION

in the field. Figure 13 shows the corrected void ratio-permeability relationships for the test site. Figures 14 through 20 show the void ratio permeability relationship for each test sample. Although this procedure appears logical, it may be limited somewhat in accuracy since it assumes that the compression curve obtained in the consolidation test corresponds to the swelling curve due to permeation. Swelling or consolidation due to permeation undoubtedly occurred in the vicinity of the borehole tube. However, this phenomena was not measured due to the physical difficulties involved in obtaining such data. Probe samples taken from the soil at the bottom of the borehole at the conclusion of a field test, showed qualitative evidence of swelling only in the first inch of the material.

Despite the above noted limitations, a comparison of the results for the entire test site was made. (See Table 7) The accuracy of the first digit in the log cycle as shown, may be questionable, however it is useful for comparative purposes. In general the falling head and infiltration values of permeability are smaller than the corresponding consolidation test values. This difference can be due to several factors, however the possibility that the sample contained a certain amount of entrapped air is likely to be the dominant cause of this difference. The range of permeability values obtained from the borehole method compare very well with the laboratory values. Therefore, this field method appears well within the limits of the accuracy desired for estimating permeability in small reservoir investigations.

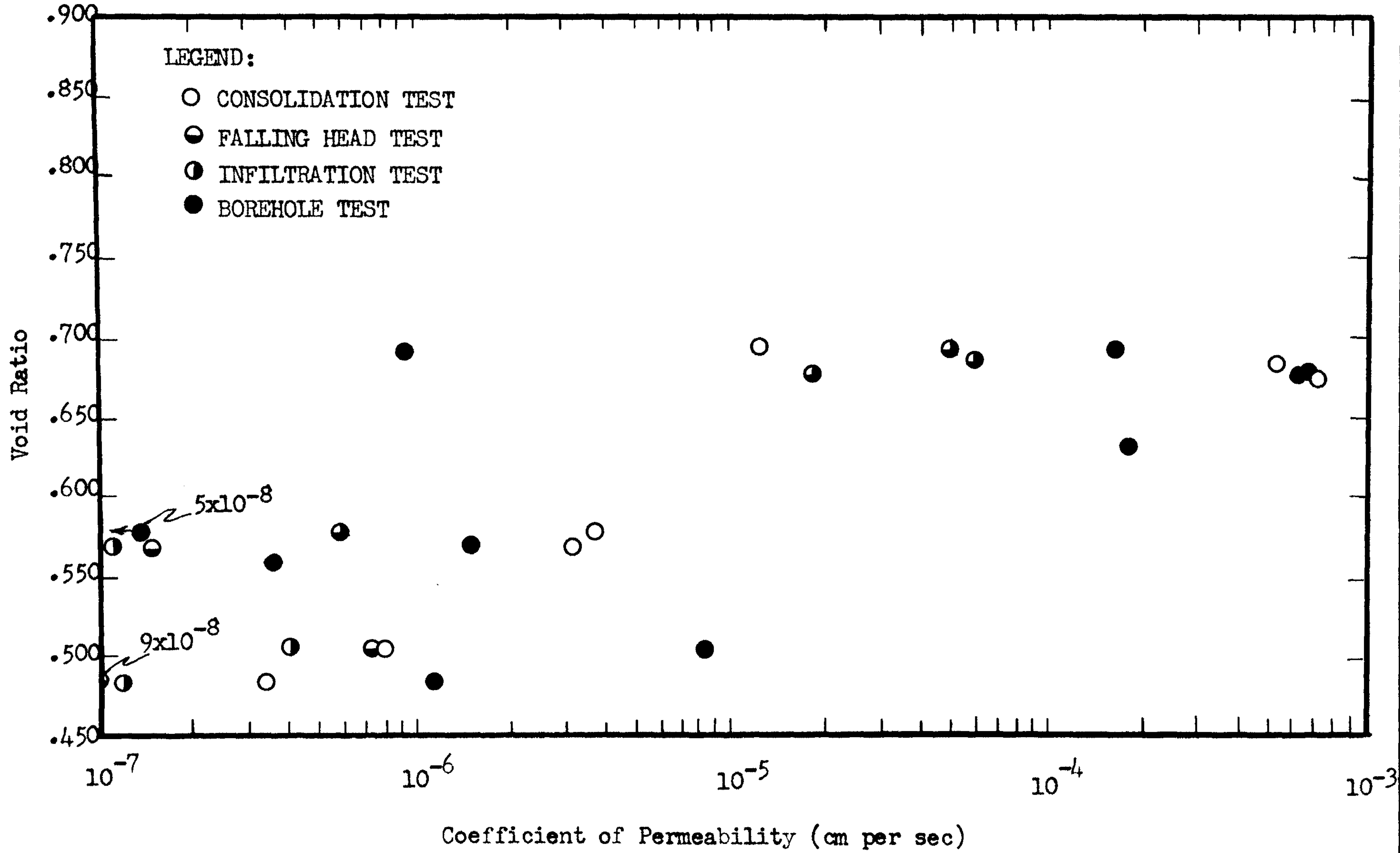


FIGURE 13. RELATIONSHIP BETWEEN PERMEABILITY AND CORRECTED VOID RATIO

TABLE 7

COMPARISON OF RESULTS OF PERMEABILITY DETERMINATION

TEST METHOD	COEFFICIENT OF PERMEABILITY cm/sec
Consolidation-Permeability	$2x10^{-7}$ to $7x10^{-4}$
Falling Head	$9x10^{-8}$ to $6x10^{-5}$
Infiltration	$1x10^{-7}$ to $6x10^{-5}$
Borehole	$5x10^{-8}$ to $7x10^{-4}$

B. Evaluation of the Borehole Method

Although this method is relatively simple, and the close correlation obtained between the field and laboratory values in this investigation indicates its applicability to fine grained unsaturated soils, certain words of caution are necessary here. The advantages and shortcomings of this method as it applies to saturated soils have been discussed in previous papers by Hvorslev and others. (25)(26) In the application of the method to partially saturated soils, these same sources of error not only exist but are likely to be even more significant.

Although the shape factor used in this method is empirical in nature, it is probably not as great a source of error as the difficulties inherent in the practical conduct of the test. Considerable time is required for the development of a wetted zone in the vicinity of the test hole. The problem of determining the minimum time to establish constant flow and the minimum volume of water required to form the bulb was not a part of this investigation. It was noted that the period of time required to obtain stable readings varied from several days in the more pervious soils to several weeks in the less pervious soils. During this period, considerable corrosion of the steel casing was noted. The effects of this corrosion were not evaluated. Leakage of water up the sides of the casing can be a source of considerable error. This was noted in the test hole 1-13 and therefore it was necessary to abandon this test location. In order to preclude this problem, it is necessary to have the casing fit snugly in the hole. In cleaning the hole, a certain amount of disturbance is bound to occur. It is necessary that the field technique be such that this effect is minimal since the greatest friction losses occur very close to the outlet. Further, in filling the hole, extreme care is necessary to prevent turbitity which could mask the results of the test. The existance of entrapped air in the area of the test is very likely. Finally the volume changes, that take place in the area of the test due to permeation and the effect on the resulting permeability value are extremely difficult to evaluate.

C. Applicability of the Borehole Method

This method appears workable in estimating permeability in unsaturated fine grained soils. However, additional investigation is necessary before the applicability of this method of testing is firmly established.

The evaluation of permeability at each test site required approximately six to eight man hours. Only the time required for the installation of the apparatus and final evaluation of permeability required engineering experience and judgement. The mechanics of keeping the hole filled and taking periodic readings does not require technical engineering ability. If the cost of this type of permeability determination can be justified in investigating a small dam site, its uses may be warranted.

VI. CONCLUSIONS AND RECOMMENDATIONS

The primary objective of this investigation was to determine the suitability of an in place method for evaluating permeability in a fine grained unsaturated soil and to evaluate the applicability of this method for the investigation of small dam sites.

This method provides permeability data, for the fine grained partially saturated soils tested, that are within the accuracy generally accepted for the solution of engineering problems.

This method is well suited for measuring the permeability of the soil cover in the investigation of small dam sites, provided qualified engineering personnel are available to provide the necessary technical supervision and to evaluate the results properly.

Systematic investigations should be conducted to detect and evaluate the sources of error in borehole observations. The minimum time to develop experimental flow conditions and the variables involved should be investigated to insure that the test is not discontinued prematurely. The borehole equation could be developed into simple nomograms thereby making calculations less complicated. It was noted that the slope of the void ratio-permeability curves varied considerably with the changing index properties of the soil. An investigation to determine variables in this relationship in terms of index properties could lead to a better understanding of this relationship in engineering terms. Finally it is recommended that further investigations be undertaken to substantiate the tentative conclusions of this investigation. Until such research is completed, this method should be applied only with the best engineering judgement.

APPENDIX A

VOID RATIO-PERMEABILITY RELATIONSHIPS

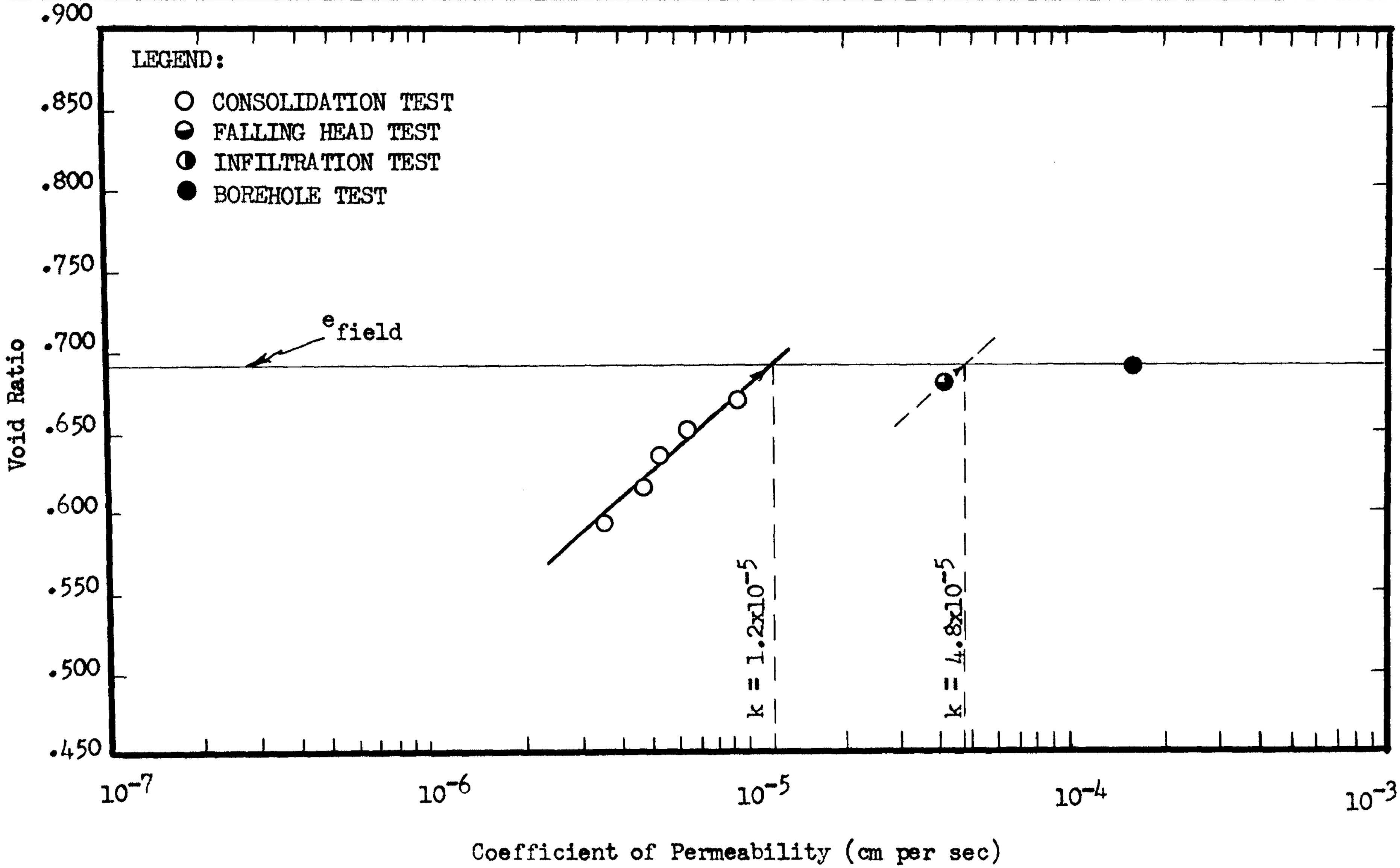


FIGURE 14. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-3

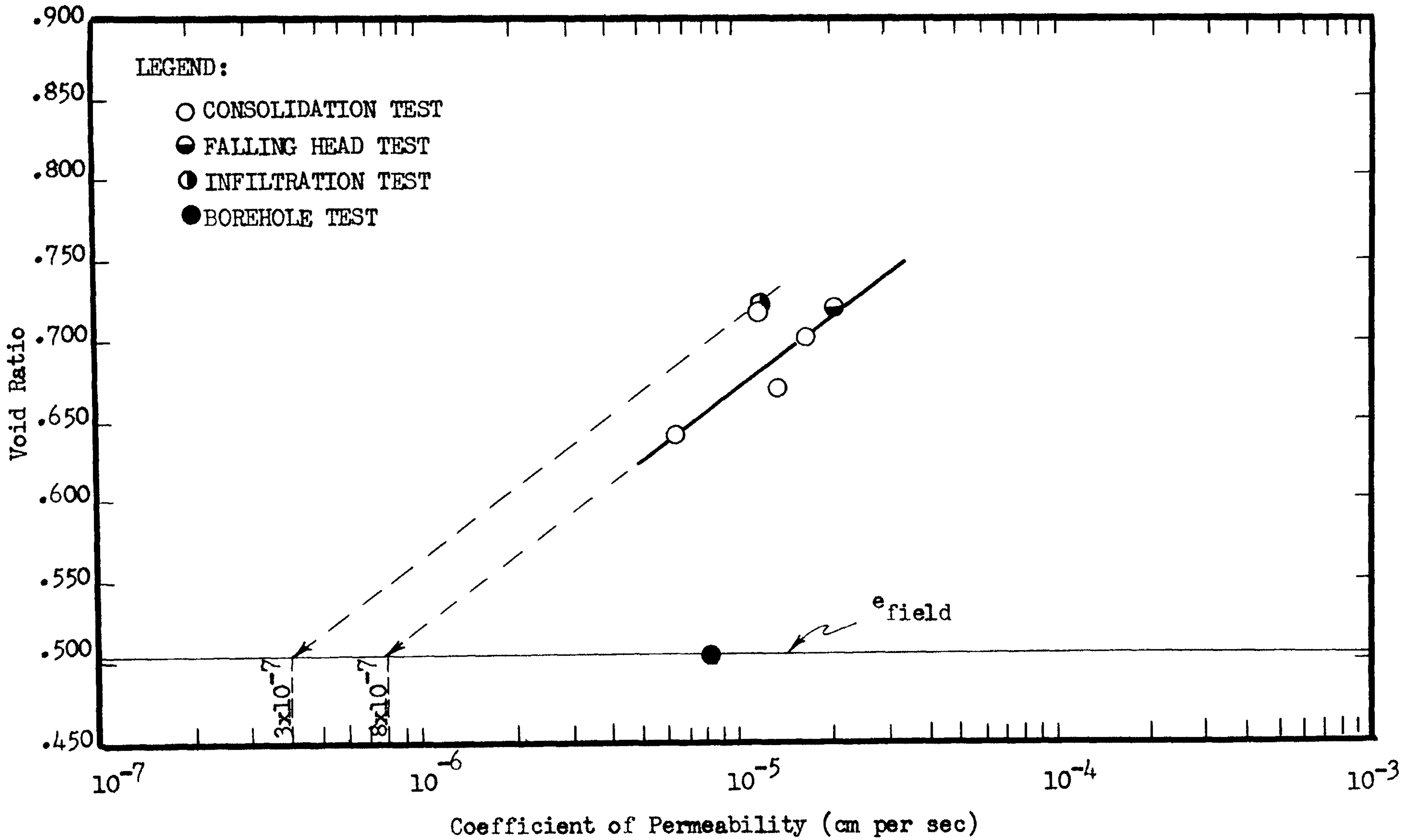


FIGURE 15. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-4

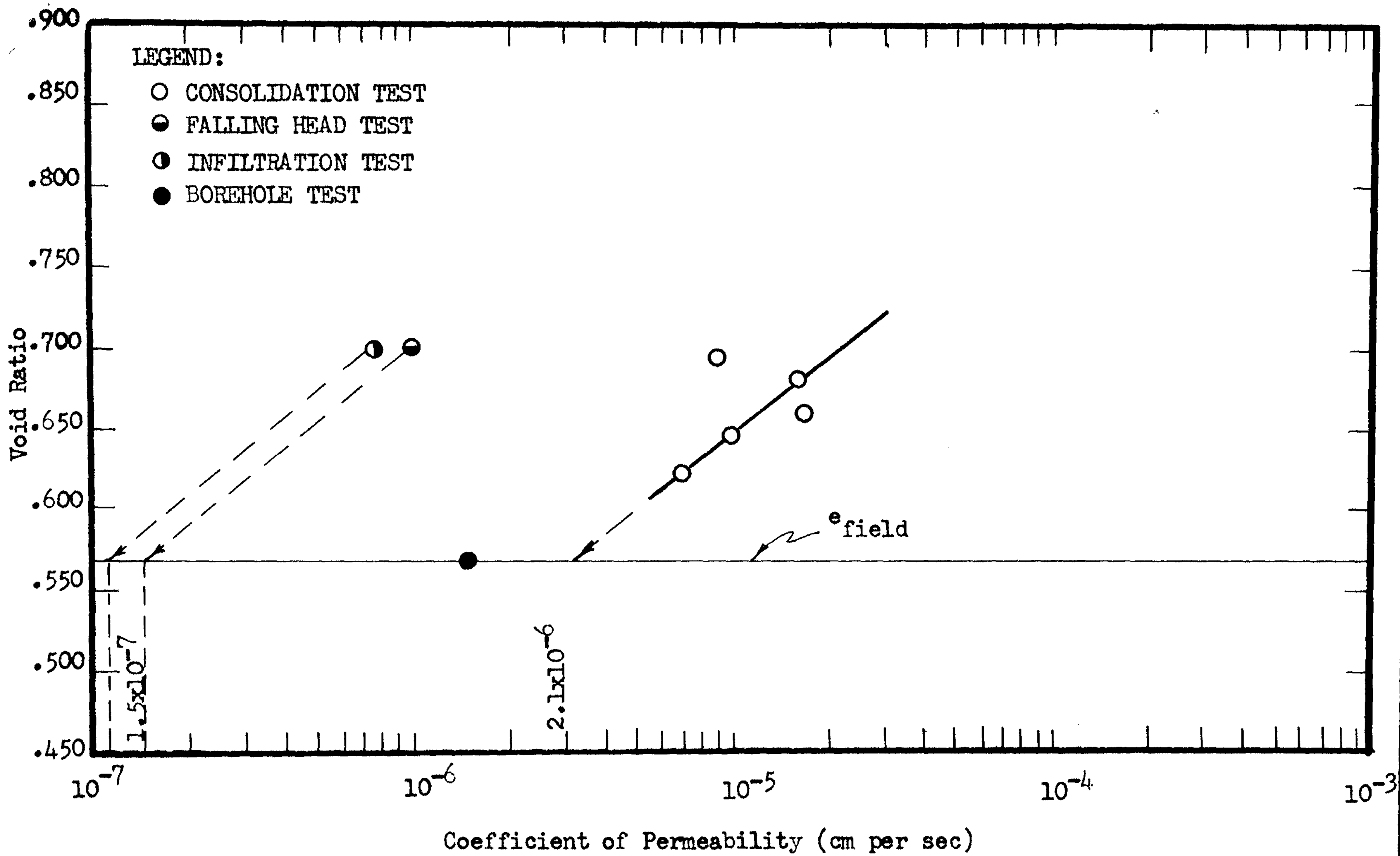


FIGURE 16. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-5

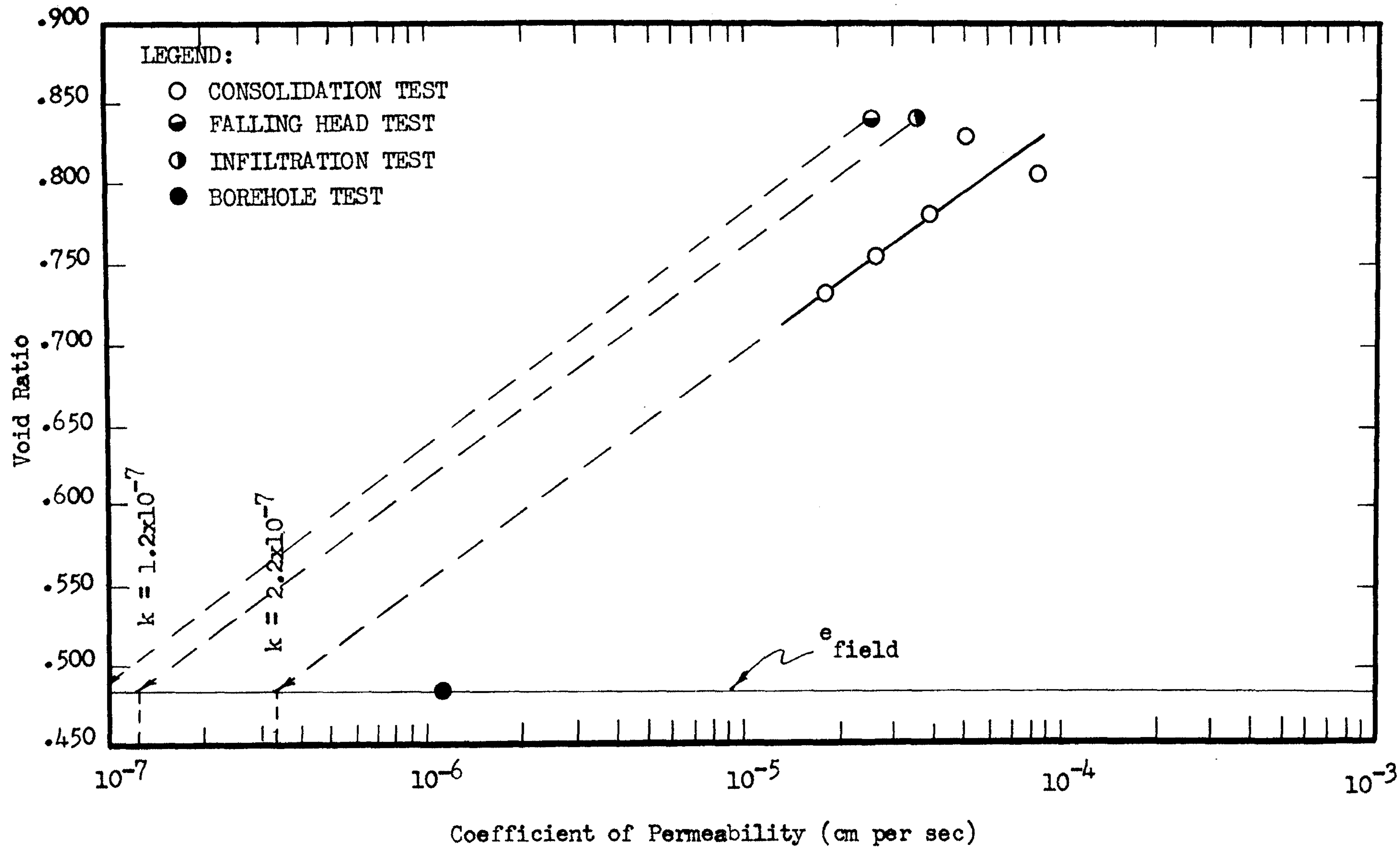


FIGURE 17. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-6

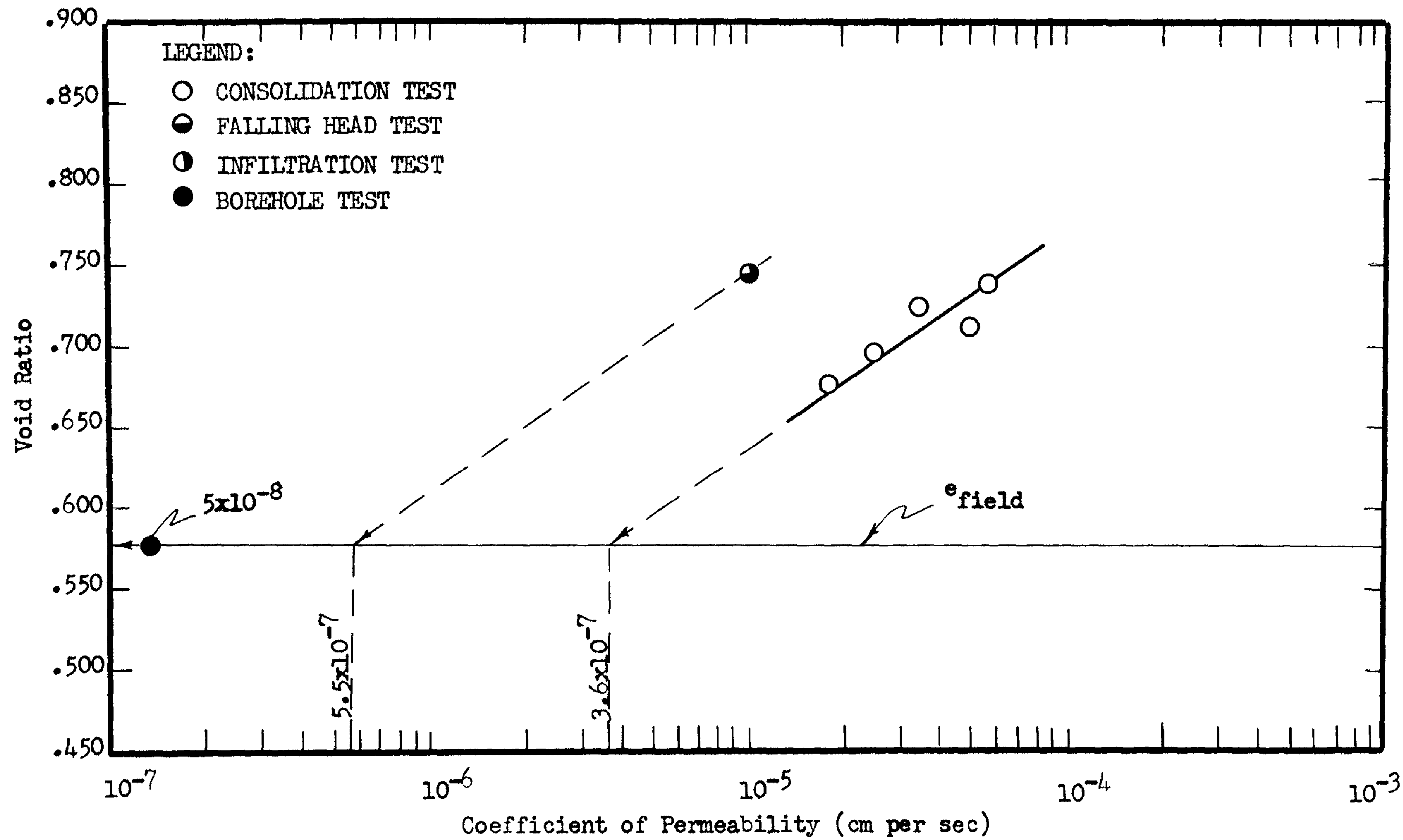


FIGURE 18. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-7

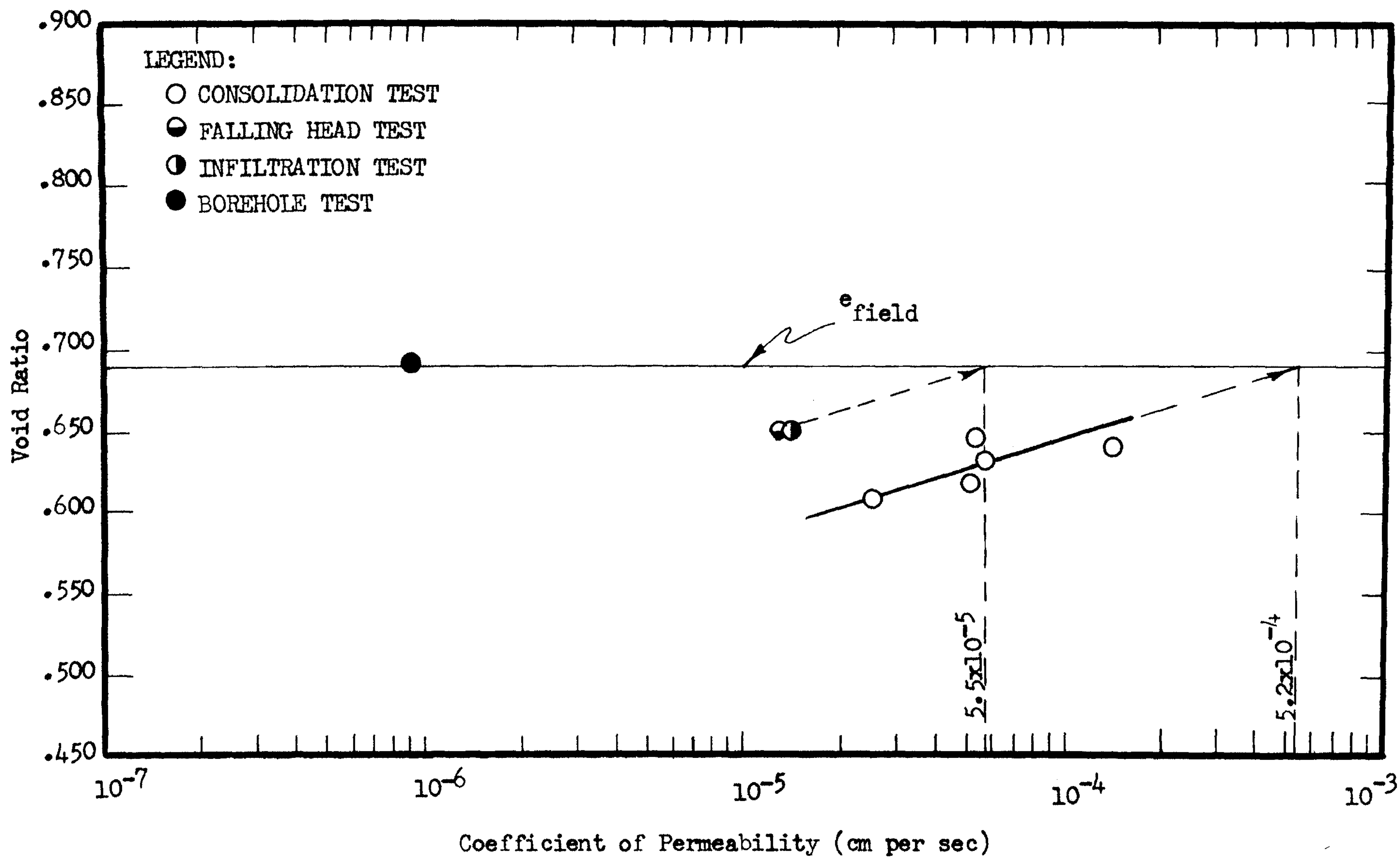


FIGURE 19. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-11

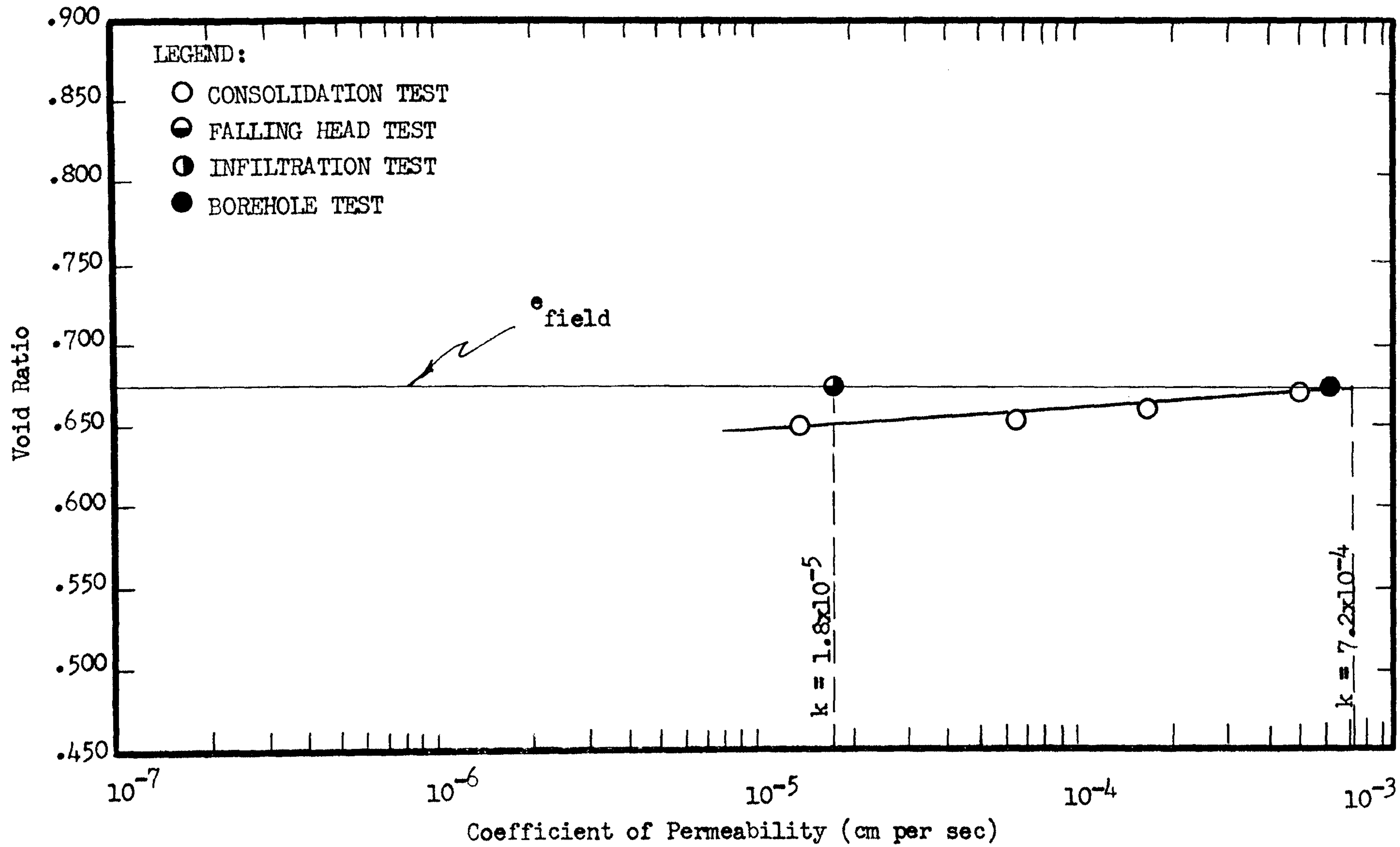


FIGURE 20. VOID RATIO PERMEABILITY RELATIONSHIP, TEST HOLE 1-12

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VITA

Edward Gerard Rapp was born on May 8, 1937, at St. Paul, Minnesota. He received his primary education in the American military dependent school system in Germany and his secondary education in the Littleton, Colorado, public school system.

In June 1960, he was graduated from the Colorado School of Mines with the degree Geological Engineer and was commissioned immediately into the United States Army. He has served in the Corps of Engineers from that date and presently holds the rank of Captain.

He was assigned to the Missouri School of Mines in September 1963 and received a Bachelor of Science Degree in Civil Engineering from that institution in 1964. At that time, he was authorized to continue as a graduate student in Civil Engineering. In 1965, he became a registered Professional Engineer in the State of Missouri.

Captain Rapp is married to the former Trudy Johnston of Colorado Springs, Colorado. They have one son, William Edward and two daughters, Elizabeth Gerard and Jennifer Lynn.