DIFFICULTIES IN SOME FIELD METHODS OF MEASURING HYDRAULIC CONDUCTIVITY

by

R. William Nelson

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In recent years considerable emphasis has been placed upon rational approaches to drainage design. Imperative to any rational approach is the development and utilization of measurement methods to determine the magnitude of acting variables. Hydraulic conductivity being one of the more important factors has received considerable attention. However, with the exception of pumping tests, only during the past decade have in situ measurements based upon sound potential theory been proposed.

The inherent variability of soils along with the wide range of hydraulic conductivities found poses serious problems in defining adequate measurement. Various investigators have reported values of hydraulic conductivity from less than 0.001 in. per hour to over 1000 in. per hour. This represents a relative range of one to a million. Essentially any method of measurement is positioning transmitting ability of a soil on this scale. It may be asked, how close must one position a particular soil on a scale of one to a million? Obviously as close as

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1 Contribution from the U. S. Department of Agriculture, Agricultural Research Service.

2 Drainage Engineer, Western Soil and Water Management Section, Soil and Water Conservation Research Branch, Agricultural Research Service, U. S. Department of Agriculture.

possible. However, usually as the precision increases, so does the expenditure of time and money. Whether or not this additional expenditure can be justified depends upon the manner in which the measurement is to be used. If five or six measurements on a small segment of the area is the basis of design for an entire area, then more precision is required and more time can be justified in getting greater accuracy. On the other hand, if many measurements are averaged or used only in a qualitative sense, then much less precision and accordingly less time in getting values can be justified.

The writer's interest is in more precision in measurement since work is currently under way which indicates that possibly a detailed knowledge of hydraulic conductivity on a small area can be rationally extended to the larger problem area without assuming soil uniformity. Accordingly, the purpose of this study was to examine the reproducibility of four field methods of measuring hydraulic conductivity, making particular note of possible causes of measurement inadequacies. As indicated previously, the accuracy required depends upon the use to which the measurement is to be put. Accordingly, the decision of method adequacy must be left completely to the judgment of the individual investigator to be used as seen fit.

Procedure

Hydraulic conductivity measurements were made by four methods in the laminated lacusterine sub-soils which characteristically underlay the agricultural area between Marsing and Homedale, Idaho. The sub-soil is a silt loam, platty structure with some cracks between the plates. A less pervious strata was found at a depth of from 10.5 to 12 ft apparently of the same texture but of massive structure. During the observations, the depth to water table was recorded continuously and remained relatively constant at 5.5 to 6 ft.

Treatments made up of four measurement methods of hydraulic conductivity were laid out in a randomized block design to allow nine replications. The measurement locations were spaced at 10 ft intervals making the over-all dimensions of the experimental site 50 x 50 ft. Originally the four methods used included the 1 in. and 4 in. piezometer, auger hole, and 2-hole methods. Because of difficulties in obtaining reliable data with the 1 in. piezometer[1] and the auger hole[2] methods, only the 4 in. piezometer and 2-hole methods can be discussed with an adequate degree of reliance.

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1 The rate of inflow to a piezometer cavity increases approximately linearily with its diameter d, yet the storage volume in the pipe depends upon the area (proportional to d^2) as well as the rate of rise. Accordingly in highly pervious soils, rates of water level change can be diminished by increasing the piezometer diameter. The rate of rise in the 1 in. piezometer was too rapid for good measurement accuracy where as the 4 in. did allow good measurement which is consistent with the above reasoning.

2 Water flowed into the auger hole so rapidly that either continuous pumping was required thereby obviously invalidating the assumptions of a level water table; or if the hole was simply bailed the rate of recovering was so rapid that increments of head change per unit time were difficult to measure accurately. Complete data is available based upon the second case. However, it is not believed reliable enough to allow direct comparison.

Piezometer Method (4 in.)

A length of 4 in. aluminum irrigation pipe was alternately augered and driven to the desired depth below the water table. Cavities of about 3.62 in. in diameter and from 4 to 6 in. long were augered below the bottom of the aluminum pipe. After pumping the water from the piezometer several times to help clean the soil pores, water was raised inside the pipe thereby causing flow from the cavity into the surrounding soil.[1] Two stop watches and an electrical depth gage were used to measure the rate of fall of the free water surface in the pipe.

The rate of free fall was converted to standard units of hydraulic conductivity through the equation for this case as derived by Kirkham (4), namely:

$$K = \frac{\pi r^2 \ln \frac{H_1}{H_2}}{E(t_2 - t_1)} \qquad (1)$$

Let $H_o = H_1$ when $t_1 = 0$, then Eq 1 can be written as

$$K = \frac{\pi r^2}{E} \frac{\left(-\ln \frac{H_2}{H_o}\right)}{t_2}$$

or in general form

$$K = \frac{-\pi r^2}{E} \frac{\ln \frac{H}{H_o}}{t} \qquad (2)$$

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[1] Curviture like that shown in figures 1 was found by R. R. Hore (Personal Communication, 1956) for the case of water flowing from the soil into the piezometer cavity.

where K = hydraulic conductivity of soil (L/T)

$2r$ = inside diameter of piezometer (L)

H = effective head in piezometer at any time t (L)

H_o = effective head in piezometer when $t = 0$ (L)

t = elapsed time since $H = H_o$ (T)

ln = natural logarithm

E = shape or geometric factor (originally this was called the A-function, however, Kirkham has suggested changing it to E-function since the tendency to call it an area function is misleading.) (L)

Subscripts 1 and 2 specify related values of head and time.

Figure 1 is a plot of the logarithm of head ratios against time for the observations for this method. A plot utilizing the head ratio has the advantage of allowing the direct comparison of consecutive observation even if the initial heads are at slightly different levels. On semi-log paper this plot of head ratios against time will be a straight line, of which the slope times a constant is hydraulic conductivity, if the conditions assumed in deriving the measurement equation are met. Some of the curves in Fig. 1 are not linear and with successive observations at a given location the curve migrates. Since the observations were made in the field, adequate control could not be exercised to separate with certainty various factors. Thus, only clues are available as to which causes of error are acting. The following are possible causes of disagreement between the experimental results and the theory:

<u>Possible Causes of Non-linearity</u>:

1. Curvature could be caused by the tendency for a water mound to develop near the piezometer, thereby causing deviations from the assumed condition of a level water table. Kirkham and Van Bavel (5) have shown analytically using reasonable values that this inconsistency is of only minor importance in the auger hole method and it would be expected to be even less important in the piezometer method. However, Reeve and Kirkham (7) pointed out that vertical channels in the soil may accentuate this problem. Kadir (3) attributed most of the curvature to this mounding effect.
2. Closely related to the previous item is the possibility of energy being dissipated due to unsaturated flow above the water table. During one observation small dust clouds appeared at the soil surface indicating that air was being displaced around the piezometer.
3. Leakage along the conduit wall may occur.
4. Darcy's law may not adequately describe the flow which could be turbulent through the fissures in this soil.

5. Curvature may be caused by deposition of sediment suspended in the water which tends to clog pores and channels as the water moves from the cavity into the soil.[1]

6. Air bubbles may exist below the water table which would change volume depending upon the head applied.

Possible Causes of Curve Migration

1. Curve migration may be an interaction with the characteristic non-linearity described in the previous topic. This would assume little or no migration would occur if the time-head ratio plot were a straight line.

2. The cavity changes shape as successive tests are made, accordingly the geometry factor or E-function increases so the apparent observed hydraulic conductivity increases.

3. The hydraulic conductivity actually increases since the soil fines are washed into the cavity with successive runs. This would necessarily assume that the decrease in hydraulic conductivity due to the accumulation of fines in the bottom of the cavity would be much less than the increase due to their leaving the acquifer.

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[1] This is not very likely in view of the clogging data presented later in connection with Childs' 2-hole method since such a small volume, less than 1 ft^3, of water flowed during an observation.

4. A thin film on the cavity wall caused by forming the cavity may gradually be removed, thereby increasing the hydraulic conductivity as successive measurements are made. This is rather unlikely since the well was pumped continuously for an extended length of time prior to measurement. Kirkham (4) in theoretical considerations indicates this to cause maximum errors of 5 to 8%.

In this group of possibilities, two types of changes are apparent. Type 1 is an irreversible change, for example, if one side of the soil cavity were to collapse, any effect it had would be permanent. In contrast, Type 2 would be those where variations are reversible. Keeping these two possibilities in mind, consider the successive measurements made at Row 1, Column 3 in Fig. 1, which shows the widest deviations of all of the data taken.

Observations No. 1 through No. 10 for Row 1, Column 3 in Fig. 1 in general falls into two distinct classes, the first group made up of observations 1, 5, 7 and 9 and the second group composed of Nos. 2, 4, 6 and 10 with No. 3 falling midway between the two groups. Differences between the groups correspond in every case with the initial height from which the water level in the pipe was allowed to fall. For group one the initial height was approximately twice that for group two while the initial height for observation No. 3 was about midway between the previous two groups.

The remaining observation Nos. 11, 12, 14 and 15 makes up a group in which the initial head was low where as for observations Nos. 16, 17 and 18, had approximately twice that of the low group. If the latter group of observation Nos. 11 through 18, are plotted on a separate sheet of paper, then placed over observation Nos. 1 through 10, rotating it slightly, it is seen that they represent essentially the same family of curves. This would indicate a distinct change between observation Nos. 10 and 11 and apparently once the change occurred there was no tendency to return to the original condition. Since this was the only irreversible change encountered in all of the observations, there is considerable doubt that Item 5 under "Possible Causes of Non-linearity" and Items 2, 3, and 4 under "Possible Causes of Curve Migration" are the causes of deviations from theory.

Returning to the discussion of Row 1, Column 3, we still need an explanation for the effect of initial head which may be acting independently of the migration.

Careful consideration of the remaining possibilities for disagreement of the experimental data with theory as previously listed, reveals that each item falls into one of two catagories, either the E-function is altered or affected or the hydraulic conductivity is changed.

Although it is impossible to explicitly separate these two catagories in the data at hand, perhaps by further considering these two larger groups, indications can be found concerning which is the greater of the two effects acting.

If Eq 2 is rearranged to the form

$$KE = \frac{-\pi r^2 \ln \frac{H}{H_0}}{t} \qquad (3)$$

It is apparent that if the hydraulic conductivity K and the geometry factor E are true constants as assumed, the product of the two also must be a constant, thereby requiring the right hand member of Eq 3 to be a constant. Plotting head (H) as abscissa and the product of K, and E as determined from Eq 3 as ordinate, the curve should be a straight line parallel to the horizontal axis if K and E are constants. Figure 2 is such a plot and it is seen that the product of hydraulic conductivity and the geometry factor is dependent both upon head and on the initial height from which the observation started. Since this product changes with head, either the geometry factor or hydraulic conductivity must be related to head. Assume for the moment that E is a constant and that all the effect in Fig. 2[1] is due to K varying.

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1 Consideration of a plot like figure 2 will indicate where the theory is most nearly approached; thereby suggesting what part of the curve as in figure 1 should be used to calculate a most representative hydraulic conductivity.

This essentially would require (item 4 under "Possible Causes of Non-linearity) hydraulic conductivity K to change by passing from the turbulent flow range at the initially high head to laminar flow at a lower head later during the observation. If we take an equal increment of head change (equal energy applied) in both turbulent and laminar flow, more flow will occur in the laminar range than in the turbulent. Therefore, if we use Darcy's law to describe flow in the turbulent range, this observed K would be less than the K in the laminar range. Accordingly, in going from a high head in the pipe (turbulent flow) to a low head (laminar flow) the hydraulic conductivity K must increase. Then if E remains constant as assumed, the product KE would increase as the head decreased. Returning to Fig. 2 it is seen that as head decreases the product of KE decreases[1]. Therefore, E in most

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[1] The plot for KE and head such as in Fig. 2 will have a positive slope for each observation shown in Fig. 1, which curves upward or has positive value for the rate of change of slope (i.e. $\frac{d^2 (\ln \frac{H}{H_o})}{dt^2}$ is positive). If in Fig. 1 $\frac{d^2 (\ln \frac{H}{H_o})}{dt^2} = 0$ which required $\frac{d (\ln \frac{H}{H_o})}{dt}$ = constant (see Row 3, Column 1, Fig. 1), then for a plot as in Fig. 2 KE would be a constant and the theory is satisfied completely.

When $\frac{d^2 (\ln \frac{H}{H_o})}{dt^2}$ becomes negative as for Row 1, Column 1 in Fig. 1 a negative slope occurs in a plot like Fig. 2 thereby indicating that K is causing the greater affect on KE. In considering Row 1, Column 1 in this manner, it should be born in mind that experimental technique could have caused the negative change in slope.

cases shown in Fig. 1 has overcome any affect K would have caused in the product of K and E.

This result essentially points to the greater error effect being in E and suggests perhaps a combination of Items 1 and 2 listed under "Possible Causes of Non-linearity" above, namely: the development of a water mound and possibly closely associated is flow in the unsaturated zone above the phreatic surface.

Kirkham (4) has shown that the shape of equipotentials and streamlines (accordingly E) surrounding the piezometer cavity is independent of the head inside the pipe for a level water table. This is the case and by a like method, it can be shown that for any steady state shape of water table, the configurations of equipotentials and streamlines will be independent of head. However, in the detail consideration of the measurement of Row 1, Column 3, and in Fig. 2, it was found that the initial head did affect the slope of the heat ratio-time plot. Further in every case, save one, after it was realized that initial head had an effect[1] it was found that the higher the initial head the flatter the slope of the head ratio-time plot. The difficulty is that a steady state condition does not exist. The water table shape goes through a cycle from a level water table just before flow starts (condition of high head), then as head in the pipe drops, a water mound

[1] While conducting the first three measurements, it was not realized that the initial head affected the observed hydraulic conductivity. On the third measurement the initial head effect was noted so subsequent data included this information.

gradually builds up and sometime later as the head diminishes, the mound begins to recede toward the level water table again. For this non-steady state condition, necessarily the shape of equipotentials and streamlines, therefore E, would change as the mound grew and then receded. Since the mound development is dependent upon the head in the pipe then E also is dependent on head and probably provides an explanation for the greatest effect shown in Fig. 2.

Two-hole Method

The two-hole method was used on the Churchill site with 3 ft between the two auger holes of about 3.62 in. diameters. Both of the holes penetrated to the less pervious layer at a depth of 10.5 to 12 ft. A small pump driven by a gas engine pumped water out of one hole and into the second thereby creating a difference of water levels in the two auger holes of from 0.25 to 0.7 ft. Head differences of this amount were large enough to allow reasonable precision in measurement and yet not enough to cause undue flow above the original equilibrium water level. No liners of gravel or screen were used on this study. These observations were reduced to unit values of hydraulic conductivity by utilizing the equations presented by Childs (1), namely:

$$K = \frac{QF}{\Delta h l} \qquad (4)$$

where

$$F = \frac{\cosh^{-1} \frac{d}{2r}}{\pi}$$

K = hydraulic conductivity (L/T)

Q = volume of water being circulated per unit time (L^3/T)

Δh = difference in head in two auger holes (L)

l = length of auger holes below the equilibrium water surface (L)

d = center to center distance between auger holes (L)

$2r$ = diameter of auger holes (L)

F = geometry or shape factor (dimensionless)

On preliminary work before the main observations were made, it was found impossible to get a constant flow between the pair of holes simply by using a valve on the discharge side of the pump. Accordingly, a five gallon open top bucket with float valve and a short hose in the bucket bottom served as a constant level reservoir. The discharge could be changed by raising or lowering the reservoir level with respect to the outlet end of the discharge hose. Pore clogging difficulties resulted if the water was pumped directly into the receiving auger hole. This was overcome by directing the water down a 2 in. pipe to below the water surface then outward through several small holes.

Water was circulated for approximately three times as long as was usually required for a steady state condition to be approached. The resulting curve shown in Fig. 3 falls into three general zones, Zone 1 represents the transient condition in approaching the steady state condition found in Zone 2. In Zone 3 the conductance (discharge per unit difference in head) decreases practically linearly with the cumulative quantity of water circulated. It is believed the decrease was caused by the accumulation of suspended material in the soil pores.

Such pore clogging restricts the number of observations to one or two at a location. Subsequent to the taking of this data, Kirkham suggested what might be called a 4-hole method which would overcome the difficulty found in pore clogging (6). By placing two holes between the initial pair the gradient can be observed independent of the clogging and sluffing occurring in the outer holes. As will be discussed later, Kirkham's idea may have much more merit than just to alleviate the sediment problem.

F-function Inadequacies

The F-function[1] used by Childs is based upon Smythe's (8) derivation for the capacitance between two circular conductors in the segment isolated when two parallel planes of infinite extent cut the conductors at right angles[2].

It is further assumed that the space isolated has uniform dielectric properties and is of infinite areal extent. If those conditions are met and a charge, q, is placed on one conductor,

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1 A differentiation has been made between the shape factor E used for the piezometer method and F, the geometry factor or function for the 2-hole method. This would seem desirable since E contains a length parameter where as F is dimensionless. The analogous expression for E in the 2-hole method would be ℓ/F.

2 Isolated as used here requires the exclusion of all other charge sources from the enclosed space except the charge found upon the two conductors.

the equilibrium charge on the second conductor will be - q. Taking these conditions to the analagous physical quantities in the soil water system, the following conditions must be met respectively:

1. The water table and impervious layer must be horizontal (i.e. perpendicular to the holes) and the only hydraulic gradient present must be that between the two holes.

2. The hydraulic conductivity of the zone bounded by the water table and impervious layer must be uniform (requires homogeneous material).

Essentially then it must be assumed, in order to mathematically determine the shape factor F, that the water table is a plane surface which cannot explicitly be the case by the very nature of the procedure of raising the water level in one well while lowering it in the other to induce flow. Further unsaturated flow above the water table is not considered. Considering Items 1 and 2 above, it is evident that if the increase in water level in one hole does not equal the decrease in the second, then the assumption of uniform material is not met. Figure 4 is a bar graph of the relative magnitude of head increase to decrease for the observations made.[1] It is apparent that in only one case did they approach

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1 This type of difficulty was perhaps the cause of the discrepancy mentioned near the end of Childs (2) paper.

equality, therefore, these deviations could be attributed to non-uniformities in hydraulic conductivities.[1] The possibility of pore clogging causing some of the cases of higher rise than fall should not be overlooked. Yet it is also noted that in several cases the fall exceeded the rise considerably, a case which would not be expected to be caused by pore clogging.

Since the difference in the absolute value of the rise and fall at the two holes is a result of heterogeneity of hydraulic conductivity in the sampled soil mass, this difference is a measure of soil variability. Considering further the data at hand, there are deviations at only two points (the 2-holes). Reasonably then, the volume sampled can be broken only into two different values of hydraulic conductivity; where as the possibility exists for an infinite number of combinations of portions of the entire sample volume and relative hydraulic conductivities which could give the composite result as witnessed by differences in water levels between holes. The maximum hydraulic conductivity based on two different values in the system would result when the volume sampled was assumed to be made up of an impermeable part and a permeable part.[2] As would be expected, values for this limiting case tended to over-emphasize differences.

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1 It should be noted that essentially the same assumptions of soil uniformity are used in deriving the equations for the piezometer method discussed earlier. Although the observation sequence of the piezometer method does not allow direct observations of this source of error, the possibility of this introducing discrepancies should not be overlooked.

2 E-function for this case has been determined for two combinations of hole sizes and spacings.

The hydraulic conductivity found in the field with two observation points lies between the lower limit of assuming a uniform soil and the upper limit found when the sampled volume is divided into a pervious part and the remainder is considered impervious. This assumes that pore clogging has not been the cause of differences between rise and fall in the two auger holes. Kirkham's suggestion to utilize observations of water levels in holes placed between the pair which water is being circulated between, would seem to overcome the pore clogging problem. Additional observation holes could be used to indicate soil variability equally as well as the original pair.

Measurement of Soil Anisotropy by Differences

Combination of the 2-hole method and piezometer method to obtain estimates of anisotropy with respect to hydraulic conductivity in the vertical and horizontal planes was proposed by Childs (1). Observations of this soil seem to indicate so much natural variability that moving over even very short distances would involve the hazardous assumption of uniformity. Accordingly, attributing the differences between a piezometer observation and a two well determination even when spaced closely is certainly open to question. If one wanted to use this approach it is believed measurement with the piezometer method should be made first, then the casing removed, and the cavity extended to be used for one of the two wells in the 2-hole method. Only then could it be justified and even in this case perhaps the feasibility should be questioned.

Summary - Weaknesses and Advantages

Piezometer Method

Weaknesses:

1. Cavities cannot be observed so predicting E-function from cavity shapes involves assumption concerning their true shape.
2. The falling head measurement in some soils causes a non-steady state flow problem for which adequate geometry factors (E-function) are not available.
3. Smaller volumes of soil sampled than is often desired.
4. No indications can be had of variability in the soil mass sampled.

Advantages:

1. A minimum of equipment is required for determinations.
2. Only moderate expenditure of time required.

Two-hole Method

Weaknesses:

1. Auger hole shapes cannot be observed so predicting F-functions from their size involves assumptions concerning their true shapes and degree of pore clogging.

2. F-functions presently available are not able to consider perturbation of the water level around holes not unsaturated flow above the saturated surface.

3. F-function presently assumes impervious layer at bottom of holes or rather extensive assumptions are required to account for partial penetration.

4. Larger soil masses sampled would be desirable.

5. Elaborate equipment and more time is required per measurement.

Advantages (although not originally proposed, capable of attainment)

1. Pore clogging difficulty can be overcome by utilizing Kirkham's proposal.

2. This type of approach allows an estimate of soil variability providing additional effort is used in obtaining head measurements.

3. Since an estimate of soil variability is obtainable much larger samples could be expected to be rationally examined.

Conclusions

Soils are certainly to be expected to be extremely variable insofar as hydraulic conductivity is concerned. Experience indicates high expected variation in terms of per cent of mean value from measurement methods. However, the manner in which the measurement is to be used, dictates the accuracy required and therefore the amount of time and effort which can be justified in obtaining precision.

With these ideas in mind, the observations and difficulties found in utilizing the piezometer and 2-hole method in a structured soil have been discussed. Detailed consideration was felt necessary since the possibilities of several factors acting is always found in field work. The primary difficulty in the piezometer method seems to be in the E-function not being a true constant due to the non-steady condition associated with water table mounding around the piezometer. Closely related is the multiplicity of clogging and sluffing factors which may introduce error. The two-hole method suffers similarly with function difficulties, pore clogging and other discrepancies in auger hole shape. Kirkham's suggestion for observing head exterior to the wells between which water is being circulated probably would overcome pore clogging. Since being a steady state condition, this method allows observation of head out in the sampled soil mass, therefore, indications and possibly functions can be found to yield expressions to consider the effect of soil variability on geometry factors. If variability functions are obtained, much larger volume of soil can be examined rationally without hazardous assumptions of soil uniformity.

From the weaknesses and strong points of these two methods, the writer envisions what might be called a double trench or parallel pit method for measuring hydraulic conductivity. Water could be circulated at a steady rate between two parallel pits penetrating below the water table. Cavity or trench shapes could be determined as accurately as desired by common surveying procedures. Non-uniformities in the much larger soil mass could be considered by utilizing piezometers as desired to determine deviations from expected head at the piezometer location. Pore clogging can be overcome by utilizing the piezometers or observation wells used for head measurements. Although the proposed method would probably be unsatisfactory in materials which tend to slough, it should work well in cobble and stony soils. Standard excavation equipment could be used to construct necessary trenchs thereby reducing the cost of installation. It would (may) require a greater expenditure in time and effort, yet in this writer's opinion, it would fill the need to characterize, in some detail, larger soil masses.

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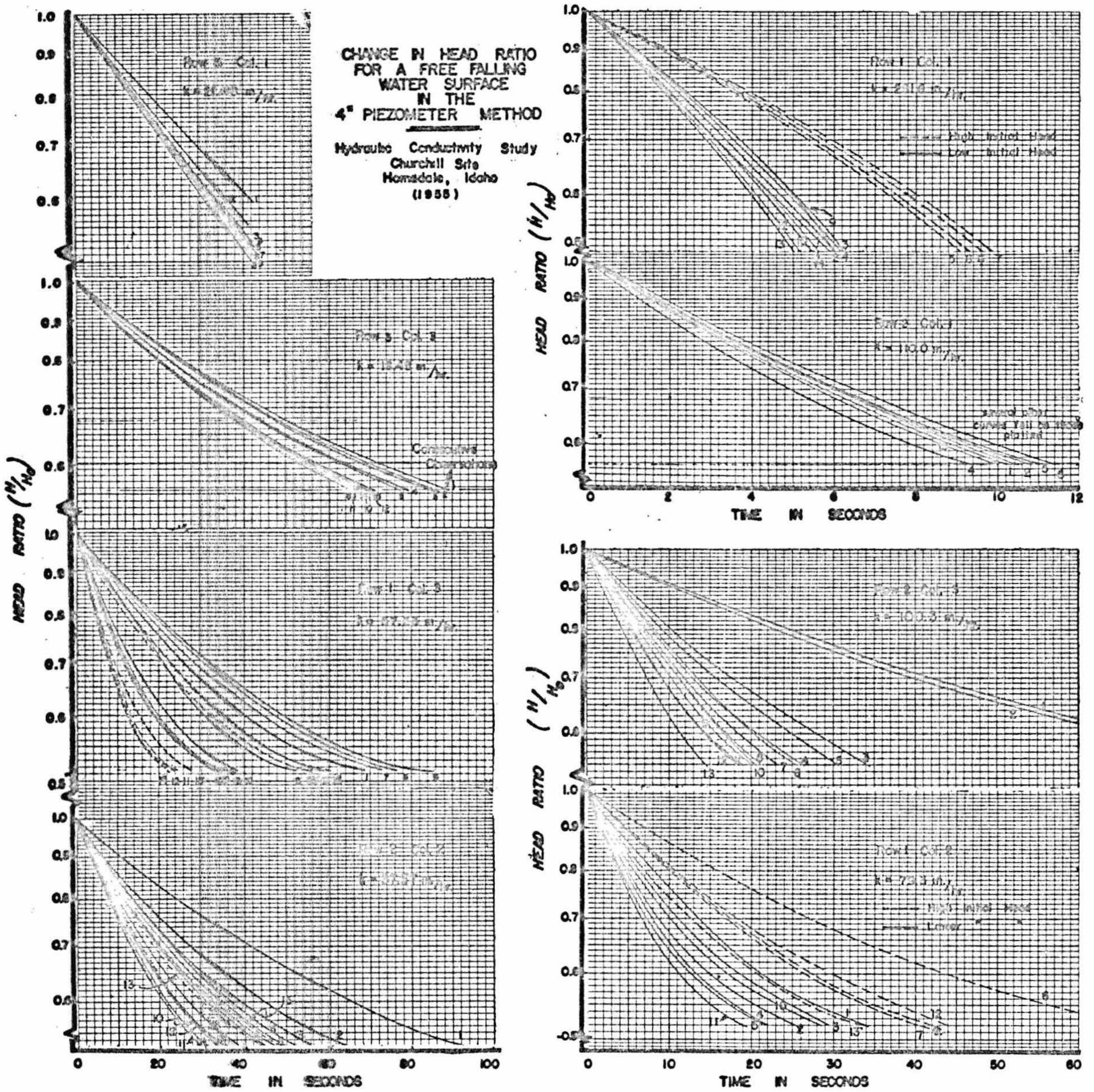
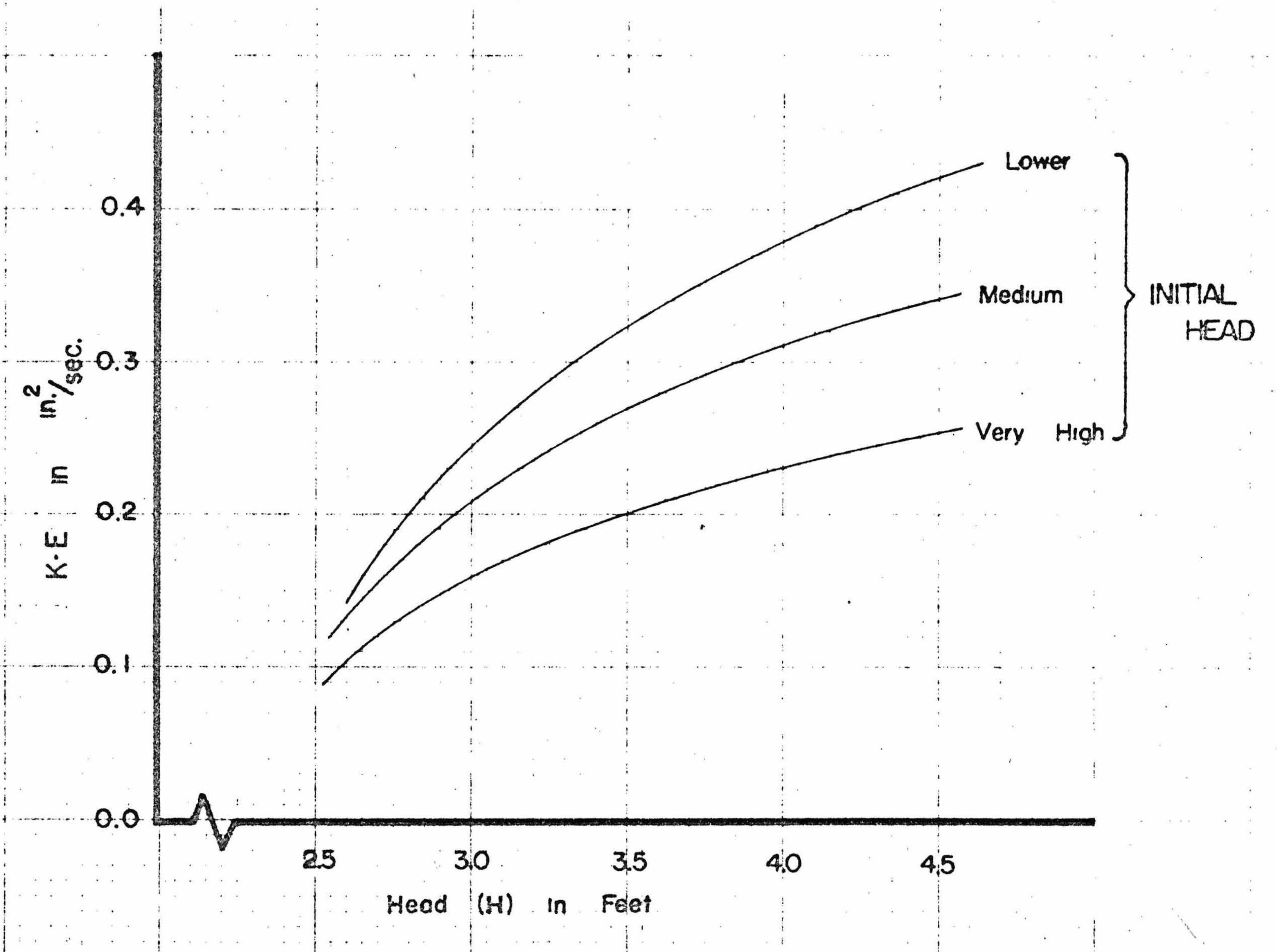
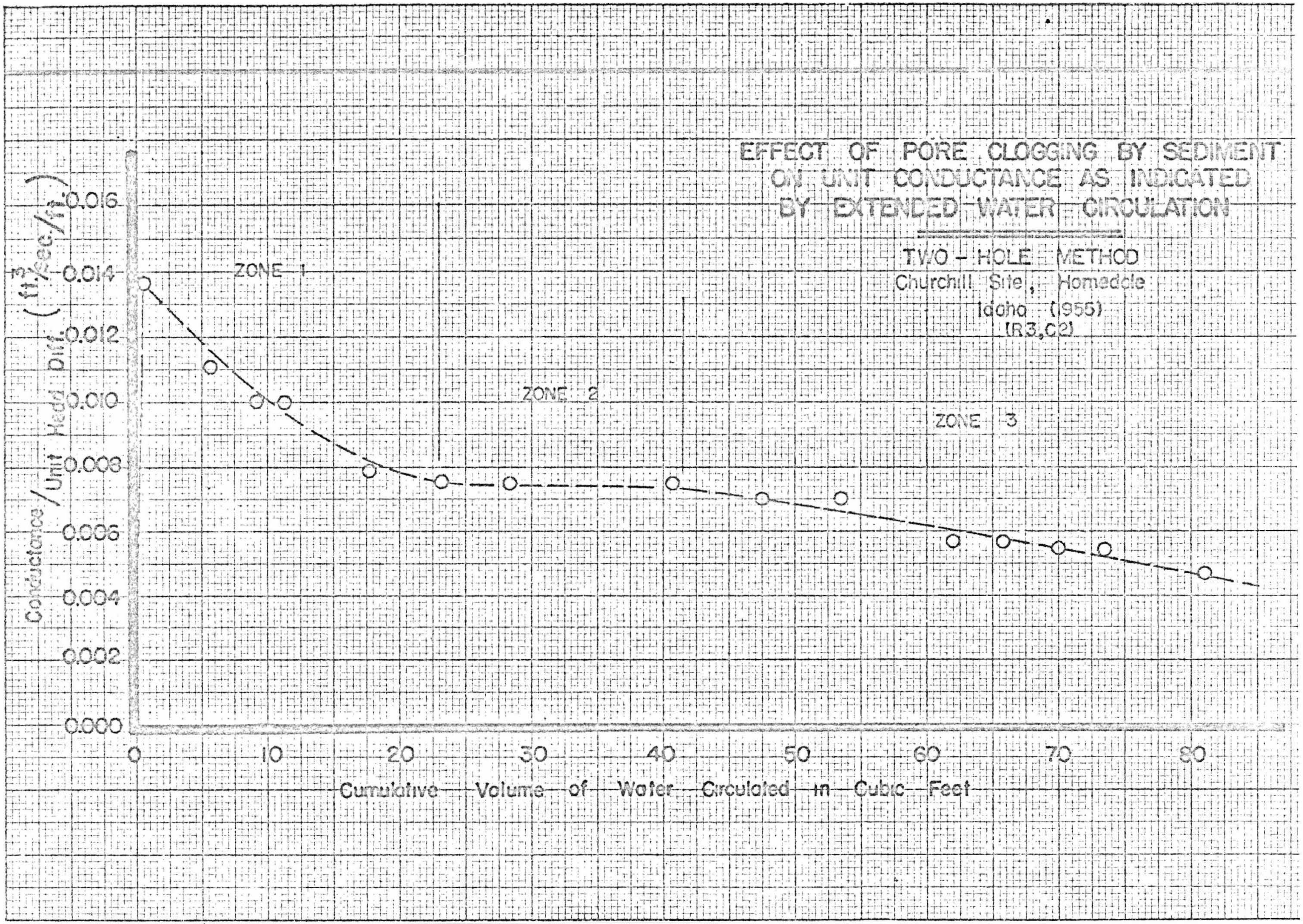


CHANGE IN HEAD RATIO FOR A FREE FALLING WATER SURFACE IN THE 4" PIEZOMETER METHOD
Hydraulic Conductivity Study
Churchill Site
Homedale, Idaho
(1955)
HEAD RATIO (H/H_0)
TIME IN SECONDS
Row 1 Col. 1
High Initial Head
Low Initial Head
Row 3 Col. 3
Row 1 Col. 3
Row 2 Col. 3
Row 1 Col. 2
Consecutive Observations

Fig. 1

Fig. 2 Effect of head on the product of hydraulic conductivity K and the geometry factor E (piezometer method).





EFFECT OF PORE CLOGGING BY SEDIMENT
ON UNIT CONDUCTANCE AS INDICATED
BY EXTENDED WATER CIRCULATION
TWO - HOLE METHOD
Churchill Site, Homedale
Idaho (1955)
(R3,C2)
ZONE 1
ZONE 2
ZONE 3
Conductance/Unit Head Diff. (ft.³/sec./ft.)
0.016
0.014
0.012
0.010
0.008
0.006
0.004
0.002
0.000
0
10
20
30
40
50
60
70
80
Cumulative Volume of Water Circulated in Cubic Feet

Fig. 3

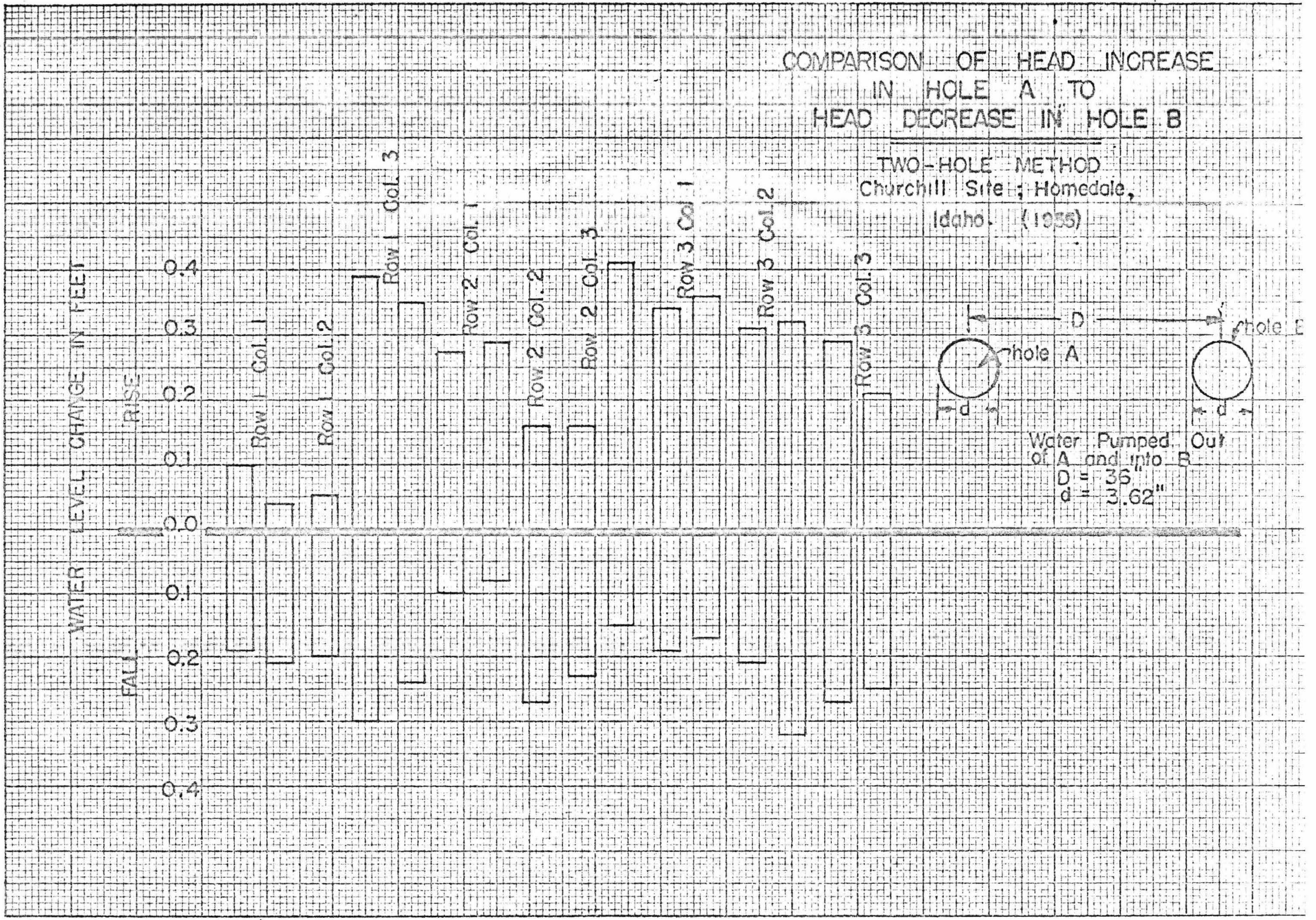

COMPARISON OF HEAD INCREASE IN HOLE A TO HEAD DECREASE IN HOLE B
TWO-HOLE METHOD
Churchill Site ; Homedale, Idaho. (1956)
WATER LEVEL CHANGE IN FEET
RISE
FALL
0.4
0.3
0.2
0.1
0.0
0.1
0.2
0.3
0.4
Row 1 Col. 1
Row 1 Col. 2
Row 1 Col. 3
Row 2 Col. 1
Row 2 Col. 2
Row 2 Col. 3
Row 3 Col. 1
Row 3 Col. 2
Row 3 Col. 3
D
hole A
hole B
d
Water Pumped Out of A and into B
D = 36"
d = 3.62"

Fig. 4