U. S. Department of Agriculture
Agricultural Research Service
Soil and Water Conservation Research Branch

ANNUAL REPORT

1955

Drainage and Water Conveyance
Colorado A and M College
Fort Collins, Colorado

By

A. R. Robinson

Civil Engineer (Irrigation)

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INTRODUCTION

During the calendar year 1955 there was a change in personnel in the Fort Collins office of Irrigation and Drainage Investigations, ARS, SWC. Mr. Jack Keller, who was employed as an Irrigation Engineer (agent) in November, 1954, resigned to accept other employment. Mr. Donald J. Sader was on military leave during 1955 and will be returning to duty about November 1, 1956.

Progress has been made during the year on four projects. These are: (1) Seepage Losses from Irrigation Channels, (2) Performance Tests on Well Screens and Gravel Filters, (3) Model Study of a Tile, Interceptor Drain, (4) Portable Measuring Device. In addition, preliminary studies have been made prior to the initiation of a new project for the study sediment ejectors. This project will be a laboratory and field investigation of the vortex tube sand trap.

REPORTS BY INDIVIDUAL EXPERIMENTS

1. Measurement of Seepage Losses from Irrigation Channels, Line Project No. R-3-2-1, Work Plan No. 2.

Location of Experiment: Colorado A and M College, Fort Collins, Colorado and Grand Valley, Colorado.

Personnel Involved: N. A. Evans, Colorado A and M College; M. M. Hastings and A. R. Robinson, Soil and Water Conservation Research Branch, ARS.

Date of Initiation and Expected Duration: Project initiated June 20, 1949. Duration: Recommended that the project be kept active to investigate special seepage measuring problems as they arise.

Objectives: Reference, Annual Report, Calendar Year 1953.

Need for Study: Reference, Annual Report, Calendar Year 1953.

Design of Experiment and Procedures: Reference, Annual Report, Calendar Year 1953.

Experimental Data and Observations: During the year 1955, the final report on the initial phases of the project was revised in accordance with the recommendation of reviewers. This report has been approved for publication as a USDA Technical Bulletin. In addition, a paper on Seepage Measurement was prepared and published by the American Society of Civil Engineers. This paper which is ASCE Separate No. 728 contains a summary of methods of measuring seepage and recommendations for use of these methods.

An additional seepage study was made in the Grand Valley, Colorado for the period of May-October, 1955. The purpose of this study was to determine the seepage rates from two sections of main canal and the relation of this seepage rate to the amount of sediment carried in the water. The study was conducted on a portion of the Main Line of the Grand Valley Irrigation Company canal where high seepage losses had been determined the previous year as well as a section on the Government High Line Canal. The section on the Main Line Canal was 7,300 feet long and that on the High Line was 23,000 feet in length.

The procedure used in conducting the studies consisted of inflow-outflow measurements made with current meters and frequent sediment concentration measurements made with a hand sediment sampler. The current meter measurements were made from catwalks at either end of the sections after periods of near constant stage as indicated by water stage recorders installed in the canals. For the Government High Line, both the inflow and outflow measurements were made in concrete flumes of the same size. The inflow measurements for the Grande Valley Main Line were in a uniform earth section with the outflow being measured in a concrete flume. It was necessary to measure all turnouts in the sections as well as waste inflow. Three sets of discharge measurements were made in each section during the season.

The sediment concentrations were determined at intervals of approximately one week and were made by using a hand sediment sampler of the type developed by the U. S. Geological Survey. Duplicate samples were taken by the integration method at each observation. The concentrations were determined as a percentage of the total weight of the sample.

The results of the inflow-outflow measurements for the Grand Valley Main Line are shown in Table 1. This table shows the total inflow and outflow for the section. All the measurements made during the 1955 season show gains in the section with the largest gain measured in October. Also shown is one measurement made in August, 1954 in which a loss was determined.

Table 2 gives the results of seepage measurements made on the Government High Line Canal. Shown are the losses in cubic feet per second and in cubic feet per square foot per day over the wetted perimeter. For each determination a seepage loss was measured, with the highest loss found for the July 14 measurement. These losses ranged from 0.53 feet per square foot per day on October 24 to 1.11 feet per day on July 14. Results of previous seepage studies made by the U. S.

Bureau of Reclamation are also shown on Table 2. These studies were conducted over a reach that included the section measured in the present study. It should be noted that the magnitude of the losses in the previous study are comparable with those in the 1955 study.

Figure 1 gives the results of the determinations of sediment concentrations for the Government High Line Canal for the period. The concentrations varied from practically 0.00 per cent to a maximum of 5.34 per cent or 53,400 parts per million which would be considered a relatively heavy sediment load. The highest concentrations were measured on May 13 and again on August 26.

Also shown in Fig. 1 are the results of the three seepage measurements made during the season on the Government High Line Canal. It is noted that the lower rates were measured after periods during which the sediment concentrations had been high which indicates that the sediment was probably factor in reducing the seepage from the canal. The results of the inflow-outflow measurements on the Grand Valley Main Line are not shown on Fig. 1 since gains were determined for each measurement during the 1955 season.

Comments and Interpretations: The results of the measurements on the Grand Valley Canal are inconclusive since gains were measured for each seepage determination. Because of the limitations in the method, i.e. accuracy of from 2-5 per cent for current meter measurements in sections such as were used for the Grand Valley canal, no definite conclusions should be drawn. However, these measurements do show that the lowest gain was determined during the July period. This compares to the highest loss recorded for the Government High Line which also occurred during the same period. During August, 1954, a seepage loss of 0.63 feet per day was recorded for this section. As previously stated there was probably a loss in this section but ground water inflow and the limitations of the method of measurement were such that gains were measured.

The reasons why seepage losses were found in 1954 whereas gains were determined for the same section in 1955 are not known. The sediment discharge records for the Colorado River are not available for the two periods. There is a possibility that the sediment concentrations were heavier in 1955 which should result in lower seepage losses in the canals because of a greater amount of sedimenting.

The measurements on the Government High Line Canal give results which are more conclusive. These inflow-outflow measurements were made in uniform concrete sections and good measuring decices were available for measuring turnouts so that more confidence could be placed on the measurements. From Figure 1, it is seen that the seepage rates were lower after periods where the water was carrying a high sediment concentration. However, more frequent seepage and sediment measurements would be desirable for a more precise determination of the effects.

In order to determine more accurately the canal seepage and the reaches over which excessive seepage is occurring, more precise methods of measurement must be employed. Ponding tests give the most precise determinations but it is usually impossible to use this method because the canal must be taken out of operation for the period of the test.

Summary: Seepage measurements were made on two sections of main canals in the Grand Valley of Colorado. In addition, determinations of sediment concentration were made at frequent intervals during the season. In one case seepage losses of fairly high magnitude were determined but for the other test section gains were measured. These gains were attributed to limitation of the method of measuring and ground water inflow into the canals. For that section from which losses were measured the highest loss was determined near the middle of the irrigation season.

The relationship of sediment concentration to the seepage rate was not definitely defined. Generally, the seepage rate was lower after periods when the water was carrying a high sediment concentration.

Table 1

Seepage Measurements

Grand Valley Main Line Canal Sec. 3

Sta 106+66 - 180+08 L = 7342 Ft. (Earth Section)

(North of G Road and Rico Flume to Concrete Flume near Appleton)

	Inflow				Outflow						
Date	GH Ft	Inflow 106+66 cfs	Wasteway Inflow cfs	Total Inflow cfs	GH Ft	Outflow Sta 180+08 cfs	Turnouts cfs	Total Outflow cfs	Section Gain cfs	Section Loss cfs	Seepage $Ft^3/Ft^2/day$ (Ft/day)
5/11/55	0.707	109.23	0.48	109.71	0.702	102.12	7.77	109.89	0.28	-	-
7/12/55	1.02	123.03	0.85	123.88	1.02	115.33	8.65	123.98	0.10	-	-
10/20/55	0.782	109.55	0.05	109.60	0.730	104.86	6.60	111.66	2.06	-	-
8/2/54	-	125.69	0.10	125.79	-	114.87	9.23	124.10		1.69	0.65

Table 2

Seepage Measurements
Government High Line Canal - 1955
Stations - 1273+00 - 1553+00 - 28000 Ft.

Date 1955	Inflow: GH Ft	Inflow: Inflow Sta 1273 cfs	Inflow: Wasteways inflow cfs	Inflow: Total Inflow cfs	Outflow: GH Ft	Outflow: Outflow Sta 1553 cfs	Outflow: Turnouts cfs	Outflow: Total Outflow cfs	Section Loss cfs	Section Loss* Ft^3/Ft^2/day (Ft/day)	Section Loss % of avg. flow
5/13	5.35	536.74	0.0	538.74	5.20	453.08	76.64	529.72	9.02	0.64	1.82%
7/14	5.50	554.24	0.0	554.24	5.15	449.74	88.84	538.58	15.66	1.11	3.12%
10/24	4.84	413.93	0.0	413.93	4.88	376.28	30.69	406.97	6.96	0.55	1.76%

* USBR Publication - Use of Water on Federal Irrigation Projects, 1952 gives comparable data from previous studies - Sta. 940 - 1965

Year	Loss (Ft/day)
1927	0.98
1928	1.04
1929	0.99
1930	0.87
1931	1.55
1932	2.36
1953	1.07

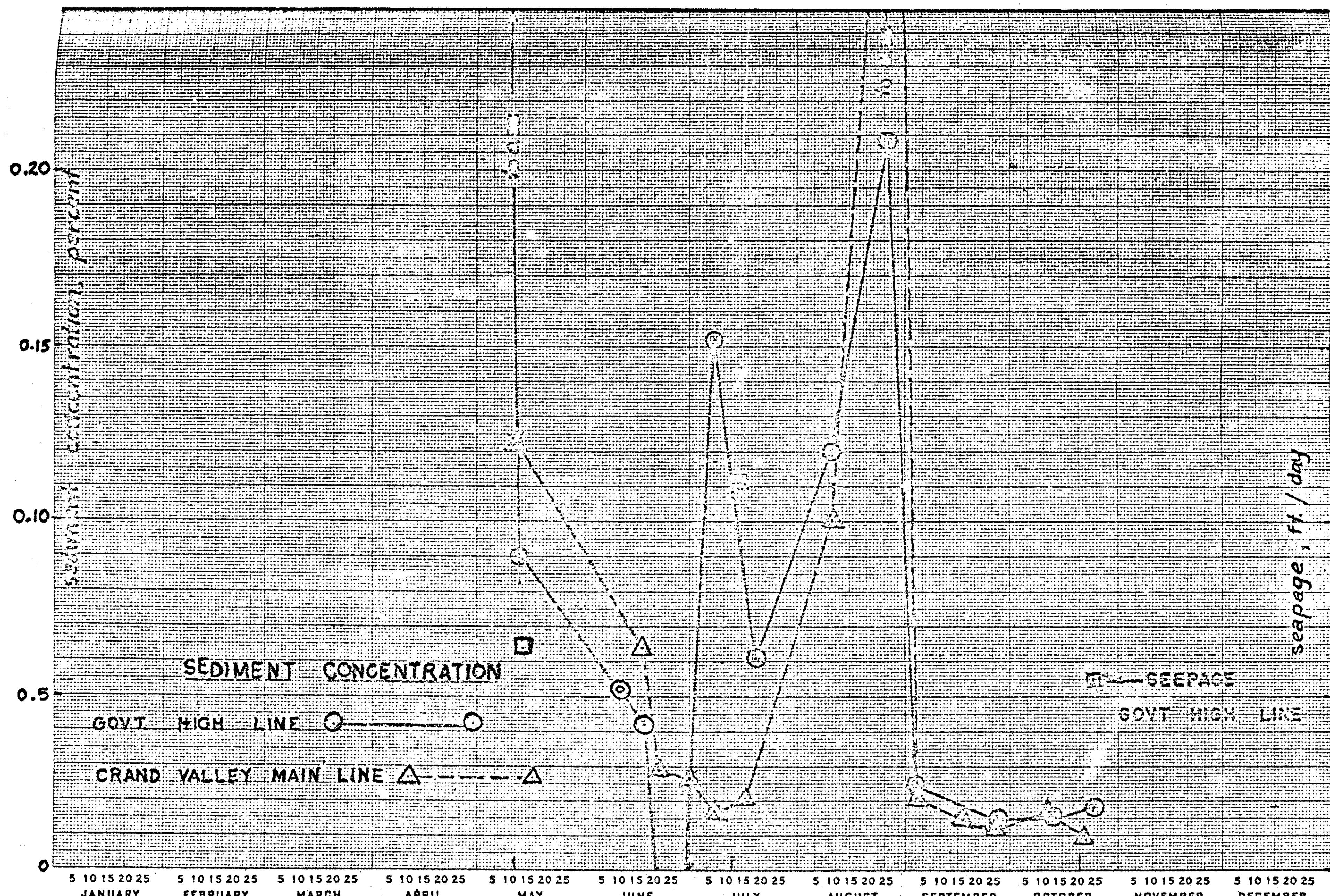

FIG. I — RESULTS OF SEDIMENT AND SEEPAGE STUDY, 1955

2. Model Study of a Tile, Interceptor Drain, Line Project No. SWC-d-7-9-4, Code No. Colo.-FC-1, State Project No. 125.

Location of Experiment: Colorado A and M College, Fort Collins, Colorado.

Personnel Involved: A. R. Robinson and Jack Keller, Soil and Water Conservation Research Branch, ARS; and Dr. D. F. Peterson, Colorado A and M College.

Date of Initiation and Expected Duration: Project initiated September, 1954. Expected duration as originally planned, one year; recommended that the project continue indefinitely until report is completed.

Objectives: Reference, annual report, Fiscal Year 1954.

Need for Study: Reference, annual report, Fiscal Year 1954.

Design of Experiment and Procedure to be Followed: Reference, annual report, Fiscal Year 1954.

Experimental Data and Observations: Two distinct aspects of interceptor drains were studied in the investigation. One aspect can be termed the flow analyses and includes the discharge that the drain will intercept, the increase in total discharge after the installation of the drain and the flow that will bypass the drain. Another portion of the study was the determination of the shape of the drawdown curve resulting from the installation of an interceptor drain. This is termed the "shape analysis" in the discussion to follow.

In this study dimensional analysis was used to relate the pertinent parameters for the various conditions of shape and flow to be studied. The laboratory experiment was designed to collect data useful for establishing the functional relationships between these parameters. A second objective of the experiments was to obtain data for comparison with the findings of other investigators.

Flow Analyses: There were three assumptions that were made in the analyses of the data on flow: (1) that the capillary flow was negligible, (2) that the drain was completely effective, i.e. there was no bridging of flow over the drain (3) for the model tests an open ditch interceptor system was simulated in the model by shutting off all the tile drains and using the tail box to intercept the flow.

The dimensional analyses of the flow system resulted in the functional relationship

$$\frac{q_d}{q_o} = \phi\left(\frac{sL}{H + sL}, \frac{h}{H}\right) \quad . \qquad (1)$$

where

q_d = the flow in the drain after installation

q_o = flow in the system before drain installation

s = slope of impermeable boundary

L = length from drain to source of supply i.e. an irrigation canal

H = depth from impermeable bed to ground water surface in the system before drainage

h = depth from impermeable bed to drain after drain installation.

This relatiohship is shown plotted in Figure 3. Shown on this plot are the data computed from the experimental observations. This plot is used for determining the flow in a drain after the drain installation and applies to any system where the length from the source of seepage supply to the point where the drain is installed is known. A further limitation is imposed in that the source of seepage supply must be one where the head at the source is not decreased by the installation of a drain. This would be the case where an interceptor drain was installed near an irrigation canal.

The special case where the slope of the impermeable layer was zero was also studied. The dimensional analyses of this case yields the functional relationship

$$\frac{q_d}{kH} = \phi \left(\frac{h}{H} , \frac{L}{H} \right) \quad . \qquad (2)$$

Where k is the hydraulic conductivity and the other terms as have been previously defined. The experimental data for this relationship are shown plotted in Figure 4. Shown on this plot are lines of constant values of L/H . The value of h/H ranges from zero to one depending on the position of the drain tile above the barrier layer.

It should be pointed out at this point that these plots can be used for all values of hydraulic conductivity. This is true since the plots are in dimensionless form. In the parameter q_d/q_o , (Fig. 3), k is contained in both q_o and q_d so that k can be canceled. The relationship shown in Figures 2 and 3 are valid for all ranges of hydraulic conductivity.

Examples of the use of these plots will be given in the section on comments and interpretations.

Shape Analyses: For the shape analyses the assumptions were made that the drains were completely effective and that the tail box acted as an interceptor drain. Previous investigations have shown that the capillary fringe does not affect the shape of the drawdown curve when the material being used is coarse material. Tests on the model showed that the drains were completely effective. From Figure 3, it is shown that the tailbox acted as an interceptor drain in the same manner as the tile lines. This is on the basis of alignment of points irrespective of whether the tiles or tailbox is being used.

For the shape analyses the functional relationship was found to be

$$\phi\left(\frac{y + Sx}{H + SL}, \frac{x}{L}, \frac{H}{H + SL}, \frac{h}{H + SL}\right) = 0 \qquad (3)$$

where x and y are the coordinates of any point on the drawdown curve and the other terms as have been previously defined. The plot of this relationship is shown in Figure 5. This plot applies only to the case where the interceptor drain is placed on the barrier layer. It should be pointed out that this plot can be used for all conditions found in the field where the depth of water bearing formation is known as well as the slope of the barrier layer.

Essentially, Figure 6 shows the relative specific energy curves for the conditions before the drain is installed and after installation. In other words this gives the relative position of the water table for both cases.

In Figure 5, the 10 per cent and 50 per cent drawdown curves are shown. These are curves connecting the points on the original water surface such that the installation of a drain causes the water surface to be lowered by the given per cent i.e. 10% drawdown equals 0.1 H . A better explanation of these curves can be given by using an example. Given an impermeable bed with a slope of 1 per cent overlain by an aquifer 10 feet thick. The drain is to be installed on the impermeable bed and 500 feet from the source of recharge. How far from the drain will the water table be lowered 1 foot. When $H = 10$ feet, a 1 foot drawdown equals 10 per cent. The parameter $H/(H + sL)$ equals 0.66. From Figure 4 the 10 per cent drawdown curves intersects the line representing $H/(H + sL) = 0.66$ at $x/L = 0.66$. Thus $x = 0.66 \times L = 0.66 \times 500 = 330$ feet.

In Figure 5, only the 10 and 50 per cent drawdown curves are shown. Any number of per cent drawdown curves could be placed on this chart. Figures similar to this could be made for other values of $h/H + sL$ i.e. for cases when the drain is placed above the impermeable barrier.

Another analyses was made which is of interest. This considers the shape of the free water surface downstream from the interceptor drain installation. Figure 6 is a representative plot of the results of the study. If the drain is completely effective the shape represented in Figure 6 was found to apply. The ground water surface was found to be parallel to the barrier layer downslope from the tile except in the near vicinity of the drain. At this point there was a slight increase in piezometric head.

Comparison of Data: An equation for the solution the shape of the uphill drawdown curve resulting from an interceptor drain has been presented previously by Donnan. This equation is

$$x = \frac{H \log_e (H-h)/(H-y) - (y-h)}{s} \qquad (4)$$

It was found in this study that this equation was only valid for the case where the drain was intercepting flow from some source which was at a great distance from the drain. In the terms of this study this is at a distance such that the term $sL/(1 + sL)$ had a value greater than 0.85. For the cases where the source was at a finite distance from the drain it was necessary to make an adjustment in equation 1. This adjustment has the effect of increasing the value of H to some value greater than the original depth of flow through the system. This value can be determined by assigning values to the terms in equation 4 so that y equals the depth of ground water flow before drainage and x the distance from the drain to the source and solving for a new value of H. This value is termed H' and the relationship is shown in Figure 7.

Figure 7 show s that equation 4 yields an incorrect plot of the drawdown curve when H is taken as the depth of waterbearing formation. However, when the adjustment is made in H to yield H' the correlation between observed data and computed data is very good.

When the slope of the impermeable boundary is zero, equation 1 is not usable and the formula developed by Dupuit is used. This formula is

$$\frac{H^2 - y^2}{H^2 - h^2} = \frac{L - x}{L} \tag{5}$$

This formula was checked by using observed data obtained with a horizontal barrier layer in the model. Shown on Figure 8 are the correlations for observed data and calculated data for the conditions of $h = 0$, 16, and 32 inches. In view of the correlations of this equation with the experimental data, equation 5 is usable for solving those problems concerning the shape of the drawdown curve after the installation of an interceptor drain above a horizontal barrier layer.

Comments and Interpretations: The information gained from this study makes possible the solving of problems concerning interceptor drains which was previously impossible. The best way to illustrate the results is in solving a hypothetical problem. For this purpose consider Figure 9.

There is a field situation where it is desired to install an interceptor drain to alleviate a high water table condition. (See Fig. 9) From field measurements it is determined that the ground water is 2 feet below the surface and the saturated stratum overlaying a barrier layer is 4 feet thick. The slope of the barrier is 0.01 and the hydraulic conductivity is 4 inches per hour. The edge of the field is 100 feet from the canal which is the source of seepage flow. It is desired to lower the ground water to at least 4 feet below the ground surface at the edge of the field. The tile line will be placed on the impermeable layer. The problem is to determine how far down the slope shall the tile interceptor drain be installed and what will be the flow in the drain.

Solution:

$H = 4'$ $s = .010$ $k = 4.3''/hr. = 10^{-4} ft/sec$

$y = 2'$ $L = 100' + X'$

$h = 0$ % Drawdown = $2/4 = .5 = 50\%$

A. Location of Drain

(1) Assume $x = 30'$, then $L = 130$ feet and $\dfrac{H}{H + sL} = .758.$

From Figure 5 for this value of $H/(H + sL)$ and for 50 per cent drawdown the value for x/L is 0.192

$x/L = 0.192 \qquad x = 130 \times 0.192 = 25$ ft

(2) Since the first assumption is not correct the next assumption will be that $x = 25'$ so that $L = 125'$ and $H/(H + sL)$ is .762. Again from Figure 4

$x/L = .194 \qquad x = .194 \times 125 = 25'$

The tile drain should then be placed at a depth of 6 feet at a distance downslope from the canal of 125 feet.

B. Flow in Drains

$h/H = 0 \qquad sL/(H + sL) = .24$ from Figure 3, $q_d/q_o = 2.2$.

$q_o = Hks = (4)(10^{-4})(.01) = 4 \times 10^{-6}$ ft^3/sec/foot width

$q_d = 2.2\ (4 \times 10^{-6}) = 8.8 \times 10^{-6}$ ft^3/sec/linear foot of drain.

From this problem it can be seen that the seepage from the canal over the length where the drain was installed nearby was increased by 20 per cent.

Summary: The objectives of this study were to: (1) study the shape of the drawdown curve resulting from the installation of an interceptor type drain, (2) determine the resulting discharge into drains after installation, (3) to check existing theoretical equations used for determining the shape of the water-table drawdown curves. The problem was investigated using a large laboratory model (Figure 2) of an idealized, uniformly sloping, homogeneous, water-saturated sand. Variables included the slope s, the thickness of the water bearing formation H, the depth of the tile above the impermeable stratum h, and the distance from the drain to the upstream source L.

Graphical methods have been developed for determining the shape of the drawdown curve for cases where the slope is zero or greater than zero. It was found from the flow analyses that in the case where a drain is intercepting seepage from a source where a constant head can be maintained such as a canal or lake that the seepage rate may be increased many fold. Installing the drain very near the source may increase the seepage by several hundred per cent.

It was found that the theoretical formula for determining the drawdown curve for systems where the slope was greater than zero,

$$x = \frac{2.3\ H\ \text{Log}(H-h)/(H-y) - (y-h)}{S}$$

was applicable for systems where the source of seepage was at a great distance from the drain. However, in the case where the source was at a finite distance from the drain, an adjustment was necessary in the value of H. This adjustment, in effect was an increase in the value of H to some value greater than the thickness of saturated stratum.

The investigation also showed that the equation for determining the shape of the drawdown curve for the case where the barrier layer was horizontal was correct. This equation is

$$\frac{H^2 - y^2}{H^2 \quad h^2} = \frac{L - X}{L}$$

Plots of observed experimental data checked very close to the graph of this equation.

The experiment also answered a question which has arisen regarding the shape of the free water surface downstream from the tile installation. It was found that there was a slight recovery in piezometric head in the near vicinity downslope from the drain. After this recovery the ground water surface remained essentially parallel to the barrier layer.

These tests were conducted in material having a capillary rise of less than two inches. The effects of capillarity on the shape and flow analyses were not determined.

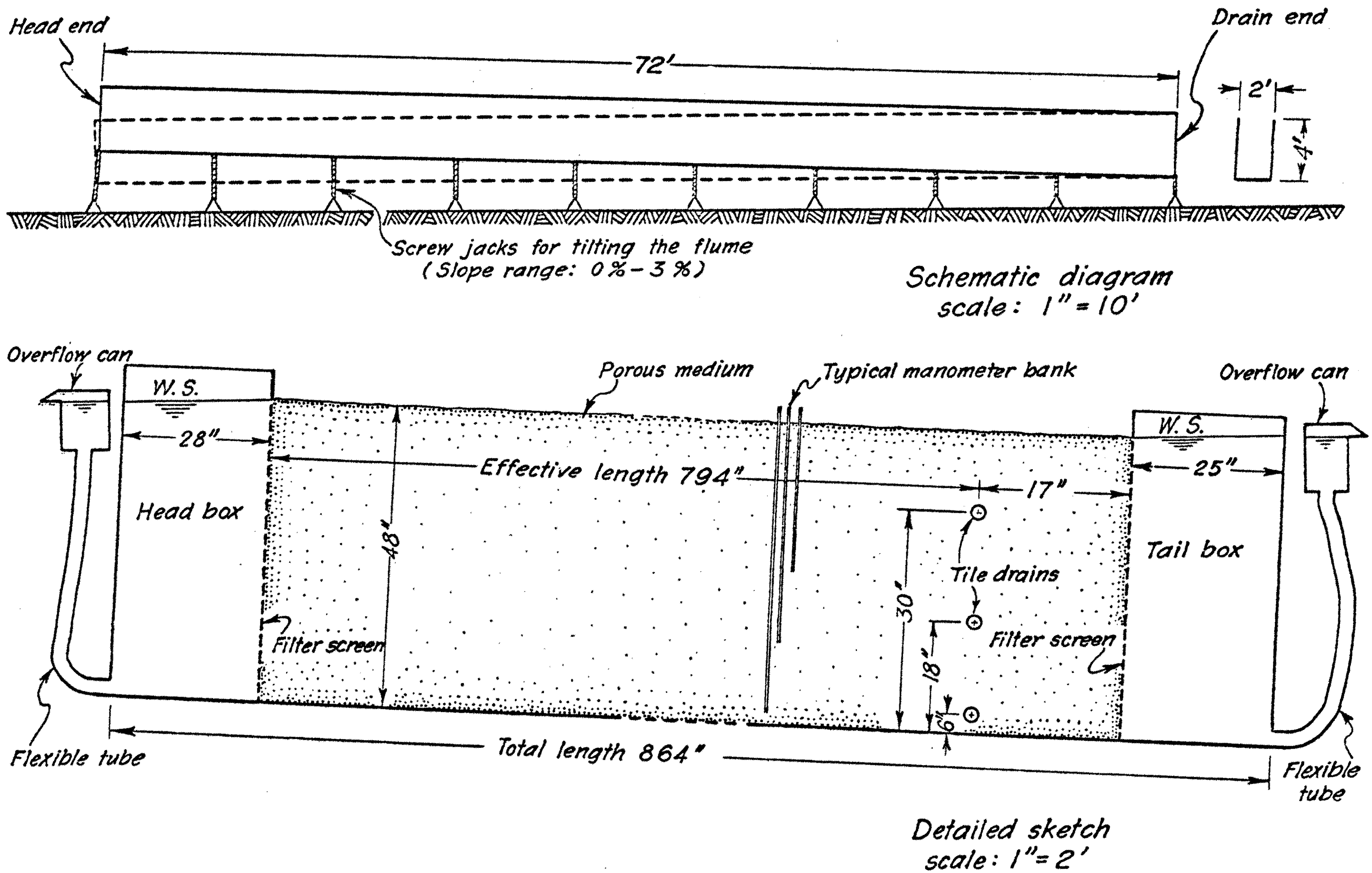

Fig. 2 Lay-out of tilting flume.

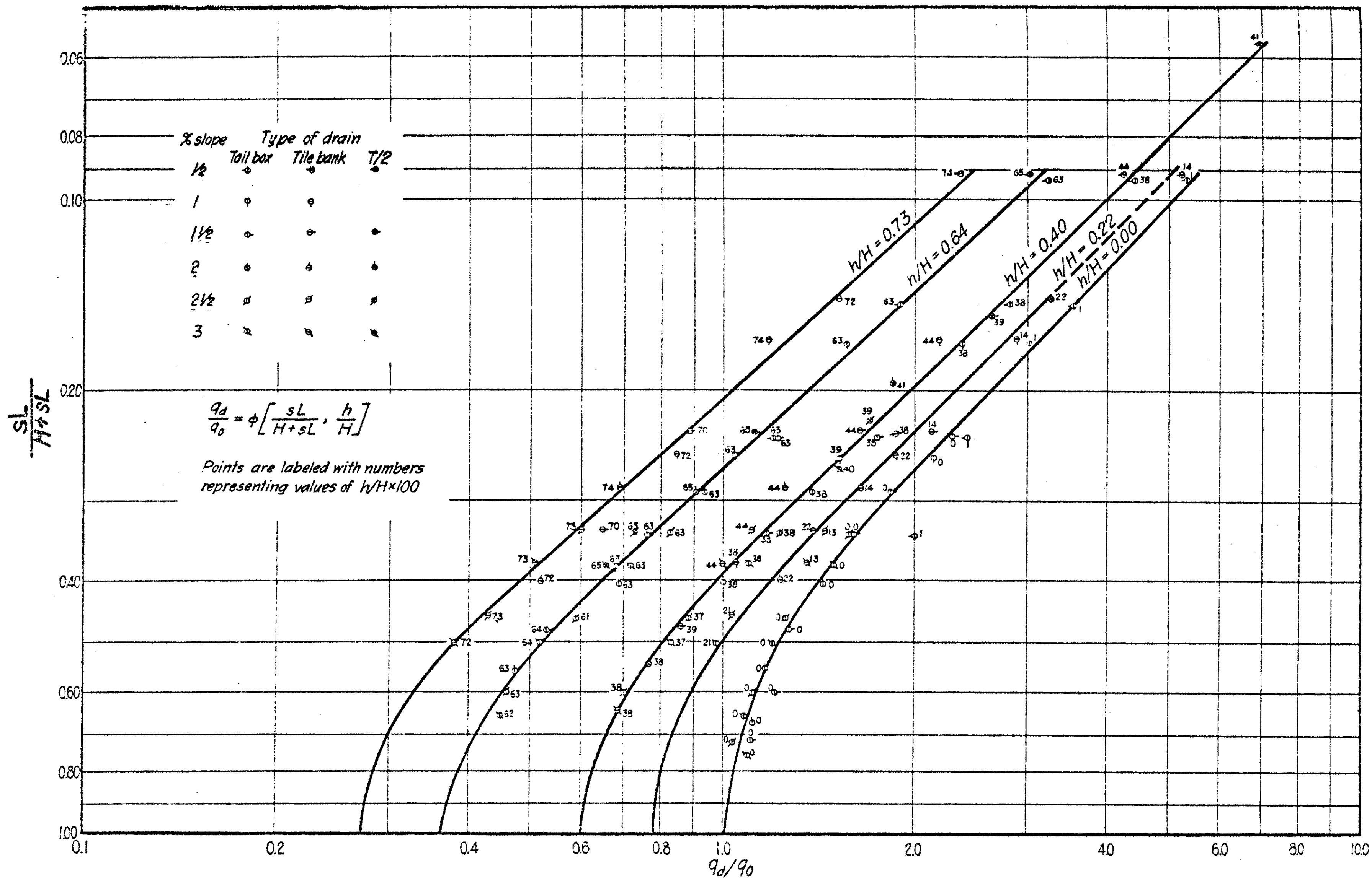

Fig. 3 Discharge of interceptor drains for slopes greater than zero.

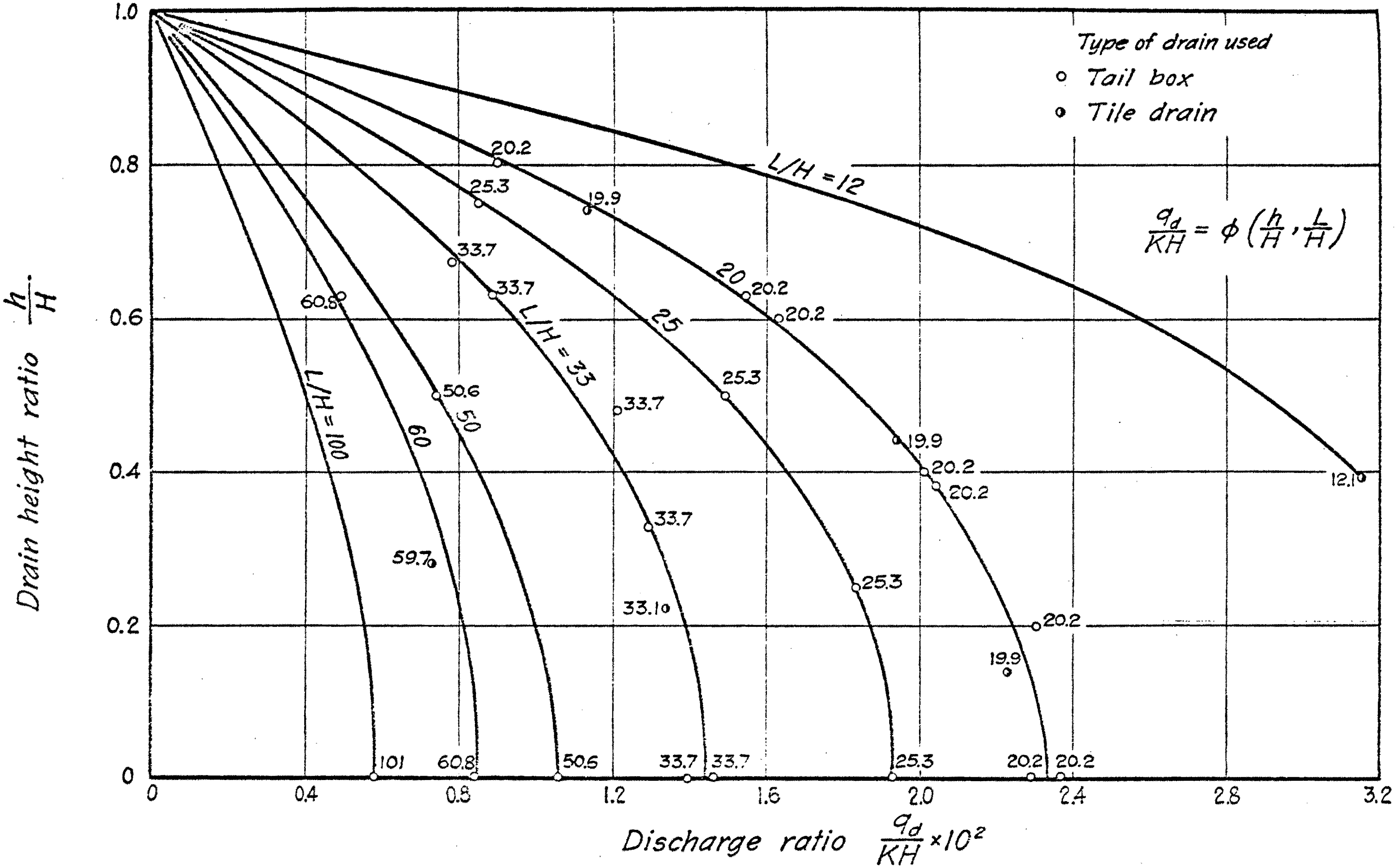

Fig. 4 Discharge as a function of drain height for slope equal to zero.

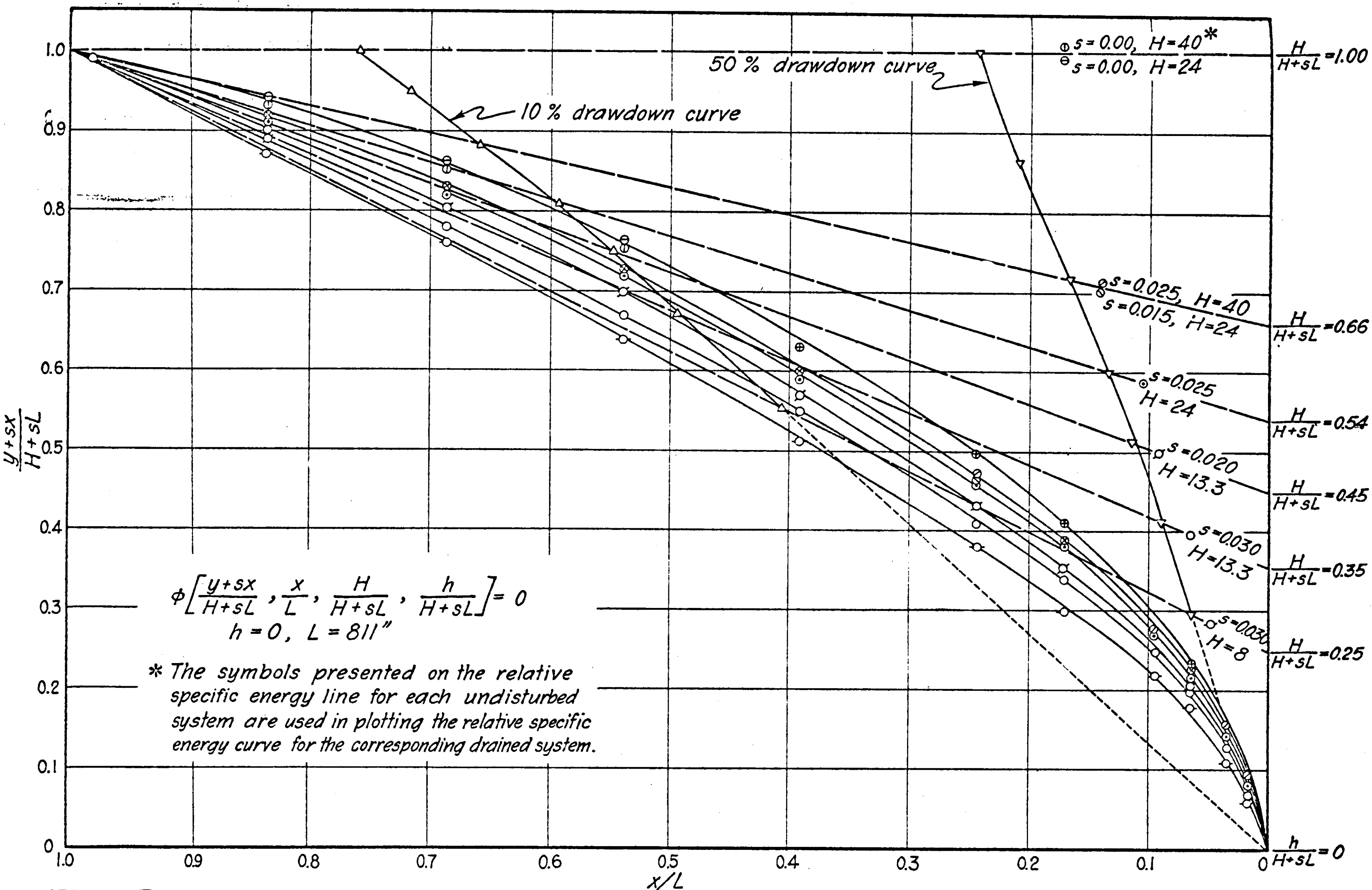

Fig. 5 Observed drawdown curves for interceptor drains when h equals to zero.

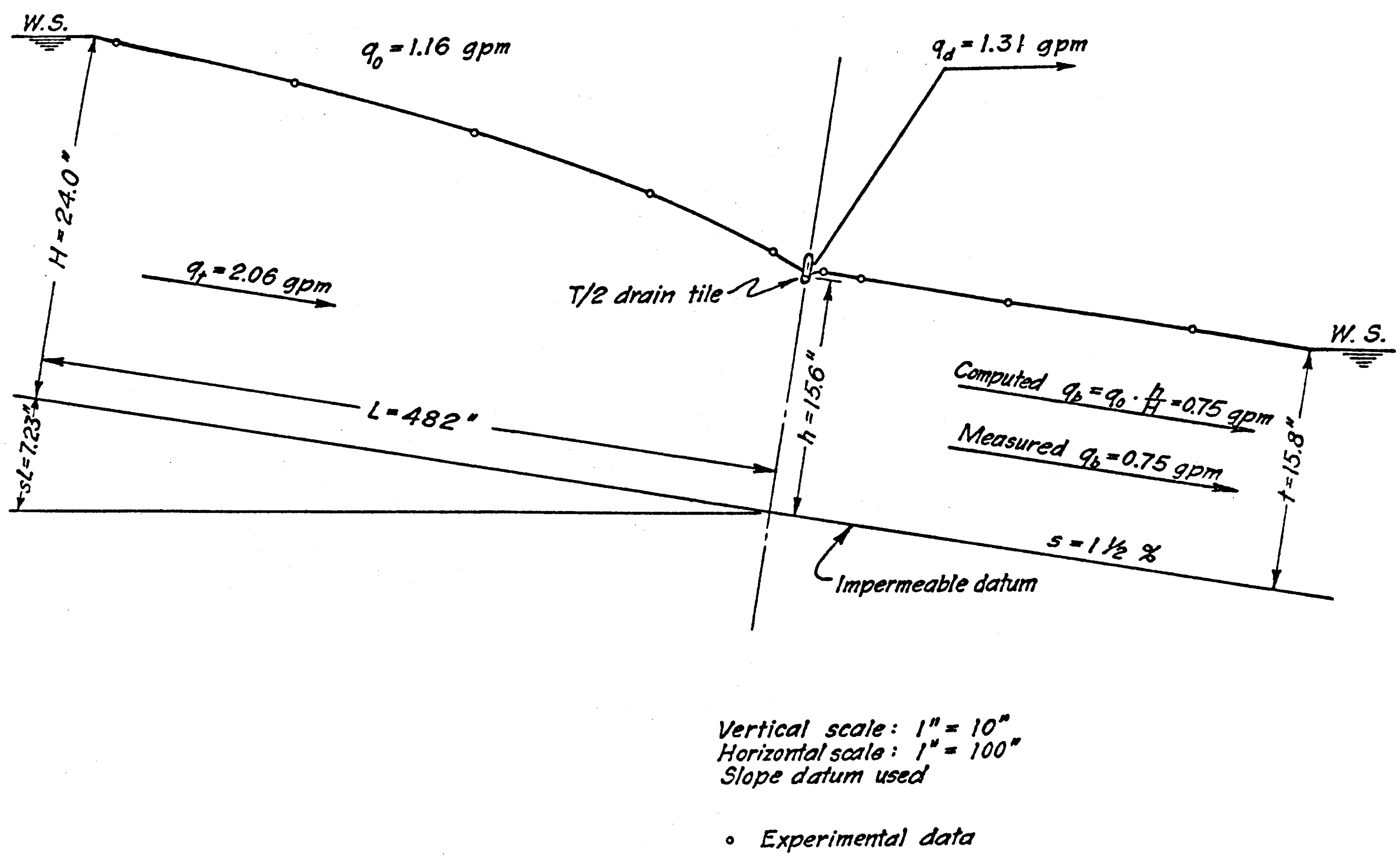

Fig. 6 Example of an interceptor drain drawdown curve.

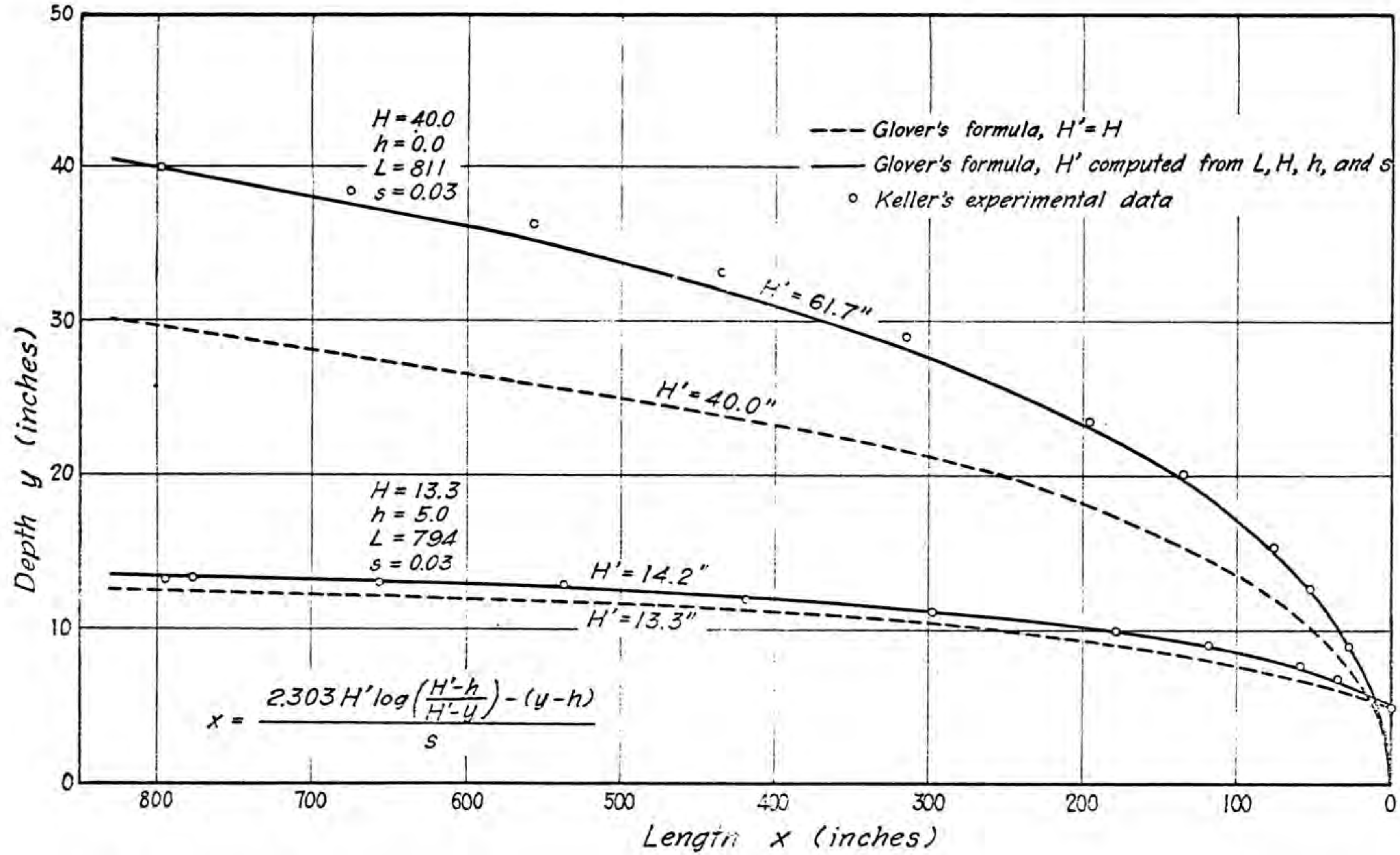

Fig. 7 Comparison of observed drawdown curves with Glover's formula for slopes greater than zero.

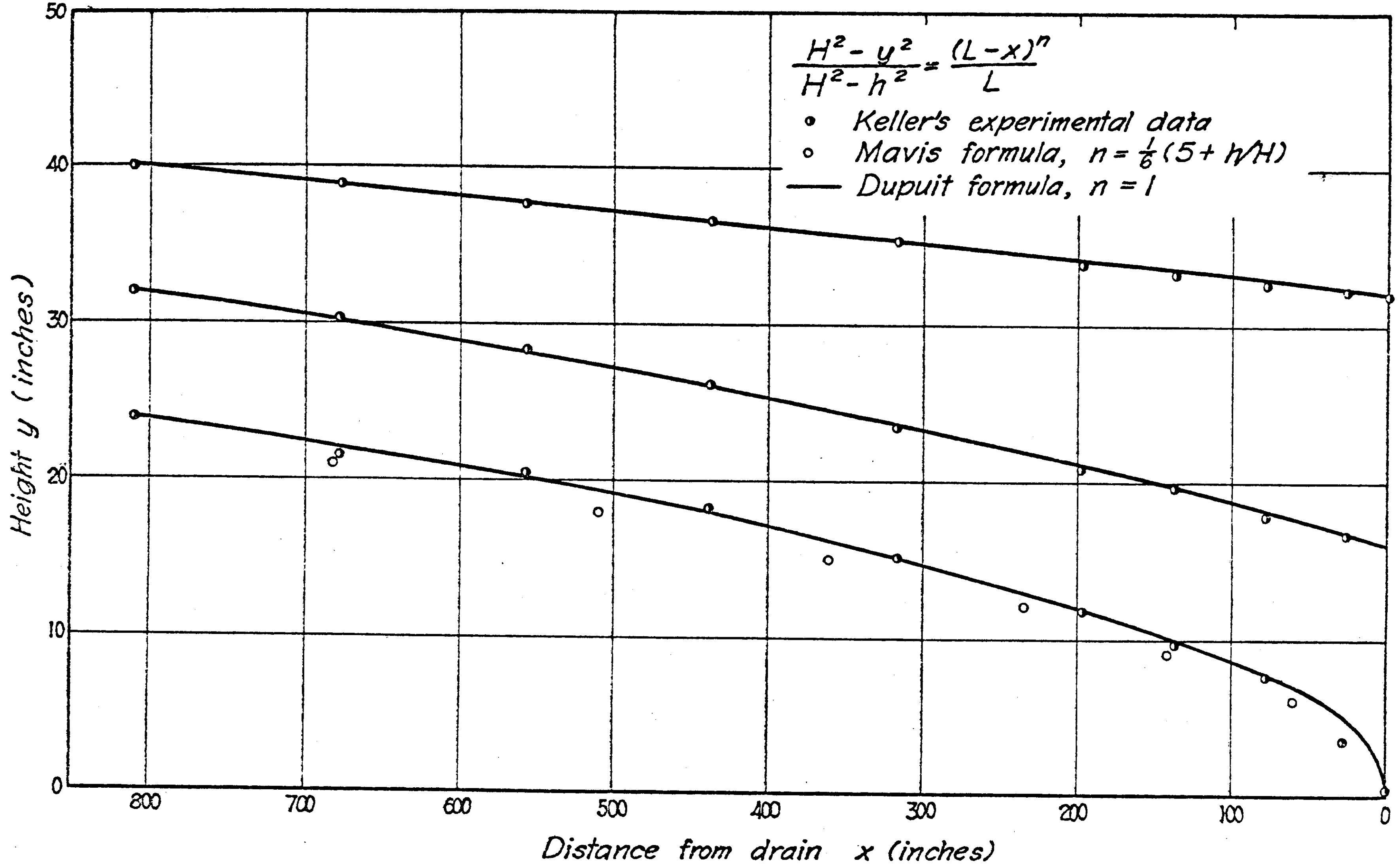

Fig. 8 Comparison of observed and calculated drawdown curves for slope equal to zero.

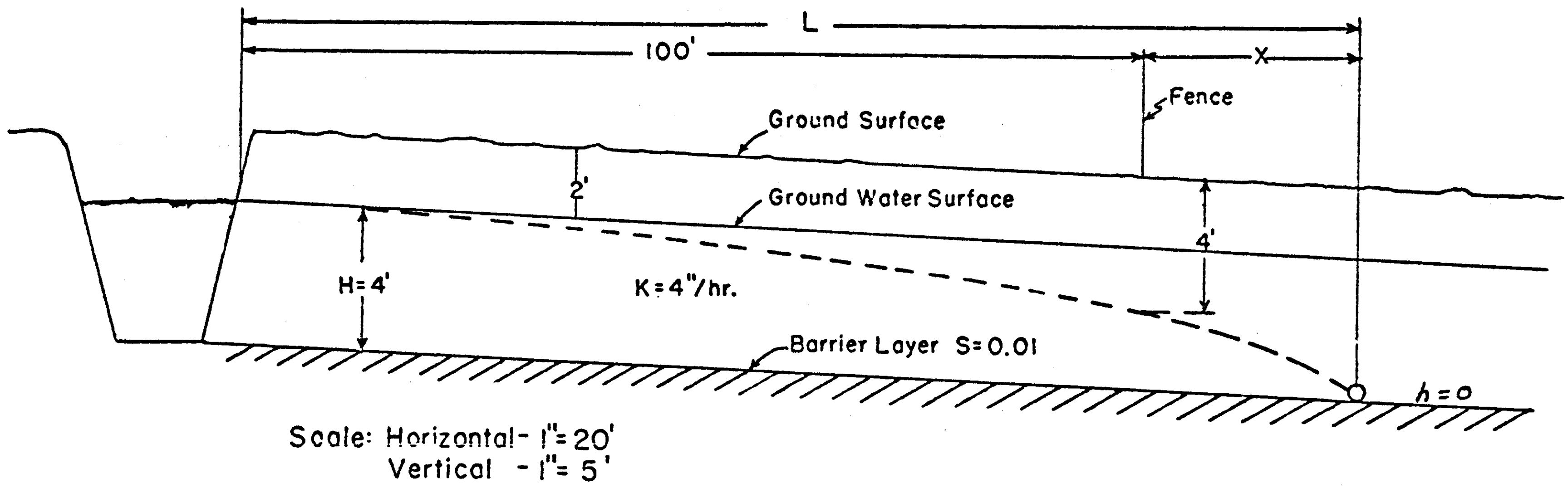

Fig. 9 Field Problem in locating Interceptor Drain

3. Portable Measuring Device for Small Slipform Concrete Canals. Line Project SWC-1-7-9-4; Code No: Colo.-FC-2

Location of Experiment: Colorado A and M College, Fort Collins, Colorado.

Personnel Involved: A. R. Robinson, Soil and Water Conservation Research Branch, ARS.

Date of Initiation and Expected Duration: Project initiated July 1, 1955. Duration, one year.

Objective: The objective of this study is to develop a portable measuring device for making quick and reasonably accurate discharge measurements in small farm ditches. The ditches for which the device is needed are concrete lined and have uniform, trapezoidal cross sections. These ditches are lined by the slipform method and have either a 1 foot bottom with 1:1 side slopes or a 2 foot bottom with 1 1/4:1 side slopes.

Need for Study: In a number of western states there are many miles of small farm canals and laterals of the slipform type. The installation of this type of lining is increasing each year. There is need for a method of determining discharges in these canals for use by technicians of the SCS and ARS in making water use studies.

Design of Experiment: The purpose of this study was to develop or calibrate an instrument for discharge measurement in these canals. There were several possible methods which were to be investigated. These methods include:

(1) A modified pitot tube device.

(2) An electrical device employing a thermistor grid.

(3) Current meter measurements.

(4) A modified Parshall flume which could be placed in the lined section.

(5) A floating diaphram device.

(6) The deflection vane meter.

These devices were to be investigated and if found suitable were to be tested in the laboratory and also used in the field.

Experimental Data and Observations: During the period it was possible to investigate methods (1), (2), (5) and (6) from the standpoint of existing literature and current studies being conducted at other locations. Method (1) was found to be unsuitable since in many cases the velocities are so low that the velocity head would not be in the order of magnitude so that it could be read on a water manometer. Method (2) would require a great amount of instrumentation so that it would not be adapted to field use. Method (5) could not be used since the canal would need to be free of debris and sediment. It was found that many of the canals were carrying a considerable amount of silt with the result that there were deposits of sediment and sand on the canal bottoms. This would interfere with the operation of a floating diaphram. Method (6) was thought to be suitable and was investigated as to its adaptability for making measurements in this type of canal. The device was developed for a permanent measuring installation in a rectangular section. Because of delicate indicating mechanisms and the construction of the pendulum it was found that the instrument should not be moved from place to place for use as a portable measuring device. Tests were made to calibrate the device in the trapezoidal cross-section by personnel of a consulting engineering firm in Denver, Colorado. These tests have not been successful and the device cannot be recommended at this time for making measurements in sections other than rectangular in shape. However, the device is being improved and redesigned for the trapezoidal shape so that it may be available in the future.

A limited number of current meter measurements were made in slipform ditches. These measurements indicate that reasonably accurate determinations of discharge can be made quickly with current meters. The method consists of making only two determinations of velocity at the .2 and .8 depth at the center of the cross section. An average of those two velocities adjusted by a coefficient will yield an average velocity for the channel. This velocity times the areas of flow will then determine the discharge.

Representative measurements giving the relationship are shown in Table 1. The coefficient to be applied to the center velocity was determined to be 0.88. It is recommended that this value be used until further tests can be made to substantiate the value.

Comments and Interpretations: The method which has been proposed for making discharge measurements in slipform canals which are constructed with a 1 foot bottom should have an accuracy within ± 5 per cent. Care must be exercised in selection of the measuring stations in order for the measurements to be within this li it. The canal should be in straight alignment for a distance of 200 feet or more upstream from the measuring station. If there is sediment deposited in the bottom, the channel should be cleaned for several feet both above and below the measuring section.

<u>Summary</u>: Several proposed methods were investigated for making discharge measurements in slipform canals constructed with a 1 foot bottom and 1:1 side slopes. Most of the methods were found to be unsuitable for making this type of measurement.

A method utilizing current meters was investigated and found to be adaptable. The procedure consists of determining the velocity at .2 and .8 of the depth of the centerline of the canal. An average of these two velocities gives the average velocity at the mid point of the canal. The velocity at the centerline times a coefficient of 0.89 multiplied by the cross sectional area yields the discharge.

From a limited number of tests it was determined that this method will yeild discharge measurements which have an accuracy of ± 5 per cent. Further investigations of the method will be made during the coming season.

4. Performance Tests of Well Screens and Gravel Filters, Line Project No. R-3-2-1, Work Plan No. 2

Location of Experiment: Colorado A and M College, Fort Collins, Colorado.

Personnel Involved: Carl Rohwer, Senior Irrigation Engineer (retired); A. D. Halderman, Graduate Student; N. A. Evans, Assistant Professor, Civil Engineering, Colorado A and M College; and A. R. Robinson, Civil Engineer (Irrigation) ARS.

Date of Initiation and Expected Duration: Project initiated: May 15, 1947. Expected duration: indefinite.

Objectives: Reference, Annual Report, Calendar Year 1953.

Need for Study: Reference, Annual Report Calendar Year 1953.

Design of Experiment and Procedure to be Followed: Reference, Annual Report 1953.

Experimental Data and Observations: Several investigators in the past have established criteria for the gravel packing of water wells. These criteria were generally established using uniform materials and the criteria specify that the pack-aquifer ratio should fall between the limits of 5 and 8. The pack-aquifer ratio is defined as the ratio of the 50 per cent size of the gravel pack to the 50 per cent size of the aquifer material. The study that was conducted during 1954 indicated that the P-A ratio criteria for uniform materials should be adjusted by some function of the uniformity of both the aquifer and filter material in order to adapt those criteria to non-uniform materials.

The present tests were performed using stable pack-aquifer ratios in which the uniformity coefficient of both the pack and aquifer materials were systematically varied. Pack-aquifer ratios used were 4.0, 5.6 and 8. Filter uniformity coefficients were 1.1 and 2.0. Aquifer uniformity coefficients ($\frac{D_{60}}{D_{10}}$) were 1.1, 1.4, 1.7, 2.0, 3.0 and 4.0. Observations were made on the amount of sand moved into the filter for the different combinations of material. Observations were made on the effect on the head loss at the interface and through the filter and aquifer as well as the effect of pore velocity and Reynolds number on the amount of sand moved. A constant discharge through the apparatus was used. This discharge corresponded to a well discharge which was near the maximum which could be expected. All tests were performed on a model using a plastic cylinder. Figure 10 shows the equipment.

Comments and Interpretations: The function of a gravel pack is to control the movement of sand from the aquifer into or through the gravel. The effect of this sand movement could result in either an increase in head loss through the filter beyond an allowable limit or, for the extreme case, could result in the pumping of sand by the pump until failure of the entire system might occur. From this study the movement of sand into the filter was used as the criterion for filter performance. The tests were run for 30 minutes after it had been found that this time was sufficient to determine the effect of the filter.

Figure 11 shows the results of using aquifers of different uniformity coefficients on the movement of sand. It is noted that there is no definite correlation between the uniformity of the aquifer and the amount of sand moved in the range of stable pack-aquifer ratios. In general, more sand moved into the uniform filter ($Cu_f = 1.1$) than into the graded filter (Cu_f = 2.0). It is also noted that the upper limit of pack-aquifer ratio of 8 is bordering on the limit of stability since the movement of sand is increasing at a high rate at this point.

There are two main factors which govern the amount of sand moved into an aquifer. One is the effect of smaller particles bridging across larger particles. This effect is greater in materials which have a large distribution of particle sizes i.e. the uniformity coefficient is greater. This bridging results in less sand being moved for materials as the uniformity coefficient increases. An opposite effect results in the percentage of fine materials which bears an unstable P-A ratio to the filter. As the uniformity coefficient of the material is increased, this percentage is generally increased so that more sand movement is encouraged. These two effects are compensating in some instances which could explain the lack of increased sand movement with an increase in non-uniformity of the material.

<u>Summary:</u> Previous tests performed on uniform materials have established criteria for allowable P-A ratios for the stability of a pack-aquifer system. The present tests were made to determine if these allowable ratios were also correct for non-uniform materials. An analysis of data may be summarized as follows:

1. The amount of sand moved appeared to ~~be~~ have as little relationship to the uniformity coefficient of the aquifer.
2. Less sand moved into a graded filter than into a uniform filter.
3. The amount of sand moved was not a function of the hydraulic gradient, pore velocity or Reynolds number within the range of value uses.

4. At a P-A ratio of 8.0, the head loss at the interface decreased during the test. For other P-A ratios, the loss remained constant.

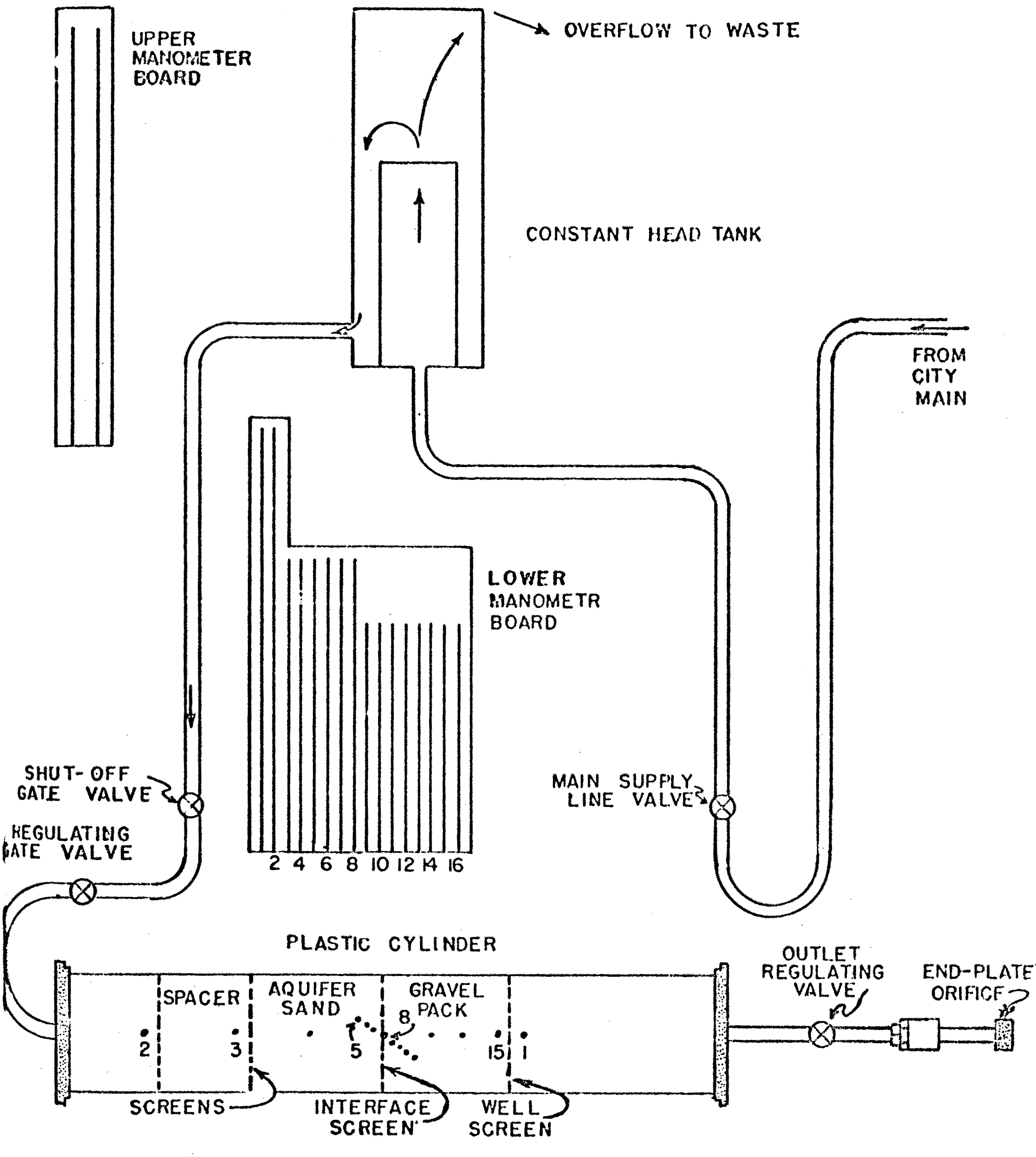

Fig. 10 Equipment for Study of Gravel Filters

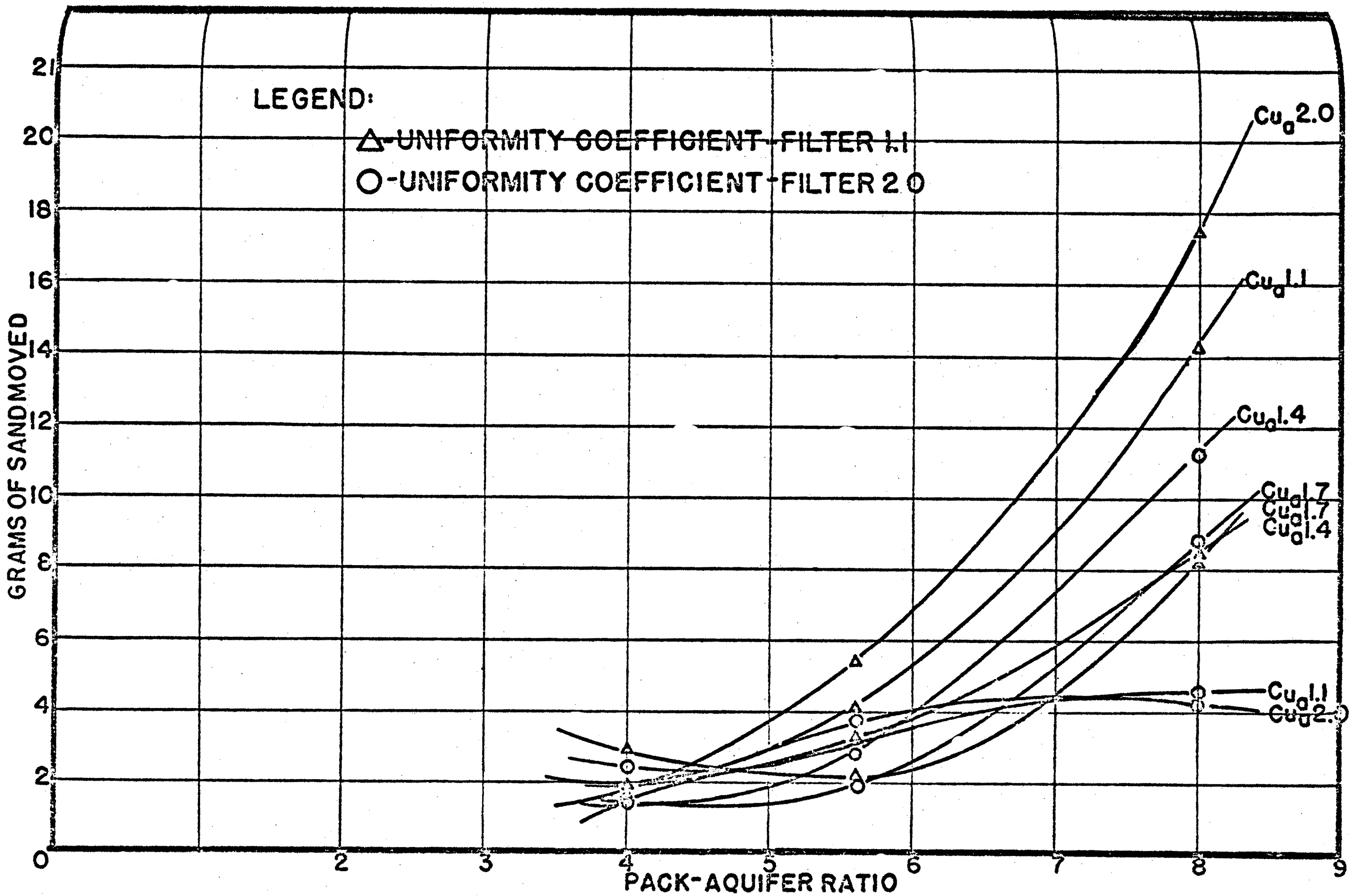

Fig. 11 Relationship of Pack-Aquifer Ratio to the amount of Sand Moved for Different Uniformities.

LIST OF PUBLICATIONS AND RESEARCH REPORTS

(1) "Model Study of Interceptor Drains", by Jack Keller, Graduate Thesis, Colorado A and M College, Fort Collins, Colorado. October 1955.

(2) "Gravel Filters for Water Wells" by A. D. Halderman, Graduate Thesis, Colorado A and M College, Fort Collins, Colorado, August 1955.

(3) "Measurement of Seepage" by A. R. Robinson and Carl Rohwer, Final Report (Manuscript being processed for publication as a USDA Technical Bulletin.)

(4) "Measurement of Canal Seepage" by A. R. Robinson and Carl Rohwer, ASCE Separate No. 728, June, 1955.

RECOMMENDATIONS

During past investigations in the study of seepage it was found that there was an appreciable variation in seepage rates over a 24-hour period. These fluctuations were of considerable magnitude in some cases and the seepage seemed to vary inversely with temperature. Several factors were investigated to determine the cause of these fluctuations but none were found which would explain the phenomena. Since this occurence had not been recognized previously, it should be investigated further to determine the factors involved.

The study on the effect of uniformity of the gravel pack and aquifer material on the allowable P-A ratios did not fully determine the relationship. The experiment was of a limited nature and did not cover the complete range of field conditions. For this reason, the tests should be continued using a greater range of flow conditions and a larger variation of filter materials.

The study of measuring devices should be continued with the purpose of developing a simple device which could be used by the farmer as well as technicians. Calibrations of small Parshall flumes are needed and should be made with the purpose of standardizing the equipment.

It is recommended that the final report on the interceptor drain be prepared. There should probably be two reports, one for publication in a technical journal and the other for use by field technicians.