MODEL STUDY OF

INTERCEPTOR DRAINS

By

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A Contribution from the
Western Soil and Water Management Research Branch, ARS
and
Colorado Agricultural Experiment Station, Fort Collins, Colo.

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TABLE OF CONTENTS

LIST OF FIGURES

Chapter I

INTRODUCTION

Drainage problems are frequently encountered in both arid and humid regions. They usually occur where there is a reduction in the carrying capacity of the natural subdrainage system because of impervious layers, cuts, decreased slope, or decreased permeability; or, where the carrying capacity of the natural subdrainage system is insufficient to handle excessive infiltration from rainfall or irrigation, or excessive seepage from lakes, reservoirs, oceans, canals, streams, or rivers.

Two distinct types of drainage problems can be encountered: those problems dealing with land areas where the excessive supply of water is from infiltration within the land area in question, necessitating a grid or herringbone type drainage system; and those problems dealing with land areas where the excessive supply of water originates outside the land area in question, necessitating an interceptor type drainage system. This thesis deals with drains of the interceptor type.

The usual objective of the drainage engineer is to lower the ground water table quickly and to hold it at the proper depth for soil stability and/or agricultural production. A great amount of information is available concerning the depth and spacing of drains when the drainage problem calls for the design of a grid or herringbone drain system. There is very little information available on the design of an interceptor drainage system even though interceptor drain lines are installed almost as frequently as grid systems.

Problem

What is the shape of the water table drawdown curve and what is the discharge resulting from an interceptor drain installed in a large scale model of a uniform sloping bed of homogeneous permeable sand overlying a barrier layer of impermeable material?

Problem analysis.--The following questions must be answered to solve the problem.

1. How does the original slope of the water table affect the drawdown curve and discharge when an interceptor drain is installed?

2. What effect does the elevation of the drain above the impermeable layer have on the drawdown curve and discharge?

3. What effect does the original depth of the water table above the impermeable layer have on the drawdown curve and discharge resulting from the installation of an interceptor drain?

4. What effect does the installation of a drain have on the total discharge of the system as compared to the discharge of the system before drainage?

5. What are the effects of capillarity on the system?

Delimitations.--This study was restricted to values of impermeable barrier slopes ranging between 0.0% and 3.0%. The porous medium used was a graded coarse sand having an effective diameter of 0.1 inch and a size range which varied from 0.025 to 0.25 inch in diameter. Only one medium was tested. These restrictions were necessary owing to the difficulties involved in building and handling the equipment.

Definition of terms.--Interceptor drain as used in this thesis is a drain installed so that it will intercept and remove water flowing laterally from some source up slope.

Drawdown curve is the water surface profile in a drainage system along which the pressure is constant and equal to atmospheric pressure after equilibrium has been established.

Original water table is the water surface profile in an undisturbed system along which the pressure is constant and equal to atmospheric pressure.

Capillary flow is that flow occurring in the partly saturated zone above the drawdown curve.

Chapter II

REVIEW OF LITERATURE

Information regarding the characteristics of interceptor drains is important to the drainage engineer. Donnan (3) in 1953, after a search for technical guidance in the field of interceptor drains, came to the conclusion that few investigators had considered the problem. A similar conclusion was reached as a result of this review of literature.

In 1953, Karaki (7) submitted a thesis concerning seepage flow from a canal to a shallow water table. A general flow analysis based on Laplace's equation with application to seepage problems was presented. Laplace's equation has been evaluated for seepage flow through porous media by many investigators. These investigations (5, 9) have shown that the results obtained from evaluating Laplace's equation for various boundary conditions agree quantitatively with experimental data.

The characteristics of an interceptor drain can be considered from two points of view, namely flow and shape. The first part of this review of literature will deal with the flow characteristics of an interceptor drain. The second portion of the review is concerned with the shape of the drawdown curve effected by the drain. A third portion in the review of literature will be devoted to considerations relative to the model study.

Flow to a Drain

In 1856, H. Darcy published the results of experimental studies on the flow of water through sand filters. His final analysis resulted in an equation,

$$V = K \frac{\Delta h}{L} \quad , \tag{1}$$

where V is the bulk velocity, or velocity over the gross cross-sectional area of flow; K is the hydraulic conductivity and is a constant depending upon the properties of the porous medium and the water; and Δh is the hydraulic head loss by flow through a length of flow path L in the porous medium. This equation is commonly called Darcy's Law. Numerous investigators have conducted tests to determine its range of validity. These tests indicate that this simple relationship holds at velocities lower than those associated with turbulent flow in the porous medium.

In 1939, Mavis and Tsui (10) presented a paper on percolation and capillary movement of water through sand prisms. Part of the study was conducted in a small horizontal glass-walled flume in order to collect data on stream paths, velocities, rates of flow, capillary rise, and drawdown curves. Tests were run on three grades of sand. Data was gathered from observations of the movement of dye streamers through the samples of sand in the small glass flume. Dye fronts were inserted above and below the drawdown curve. These investigators found that the horizontal component of the water velocity in that part of the sand prism below the drawdown curve is nearly constant at a given vertical section. They also found, that at a given vertical section, the horizontal component of the water velocity in the capillary zone varies. This variation ranges from zero at the top of the capillary fringe to the velocity of the water below the drawdown curve at the bottom of the fringe. They stated, that for the purpose of analysis, the mean horizontal water velocity in the capillary zone is approximately two-thirds of the horizontal water velocity in the saturated portion below the drawdown curve.

In 1945, Childs (1) submitted the second paper in a series of papers concerning the study of groundwater movements to drains by means of an electrical analogue. In this paper he stated that, when seepage flow is predominantly horizontal, any addition to the vertical thickness of the conductor, such as is afforded by the capillary zone, must have the effect of reducing the resistance to flow, yielding greater flow for the same boundary conditions.

In 1946, Childs (2) presented the fourth paper in the same series. This paper dealt exclusively with interceptor drainage. Electrical analogues were used for this study, and the assumption was made that the land surface extended to a great distance (effectively infinity) in all directions from any point. Childs stated the simplest problem arising in connection with foreign water. He postulated an impermeable bed, with a uniform slope s , upon which rests a layer of material of uniform hydraulic conductivity K. Down this bed ground water flows at a steady rate q_o per unit width when measured at right angles to the direction of fall. Childs studied the nature of the potential field which determines the streamlines and location of the drawdown curve, and, how it was affected by a drain installed in such a manner as to intercept the groundwater flow.

Childs pointed out that when a vertical soil section containing the direction of fall is considered in the absence of a drain, the flow net consists of a rectangular array of equipotential lines and streamlines. These streamlines will be parallel to the impermeable bed. The thickness T of the zone of flow, measured perpendicular to the bed, was given by an equation equivalent to

$$q_o = TKs. \qquad (2)$$

He stated that the distance of appreciable influence exerted by an interceptor drain is limited on the uphill side. Also, at remote distances, the flow in the intercepted system asymptotically approaches the undisturbed state described by Eq. 2. For the thickness t of the zone of flow on the downhill side of the interceptor drain and beyond the zone of local disturbance, Childs presented an equation similar to Eq. 2,

$$q_o(1-a) = tKs \ . \qquad (3)$$

In Eq. 3, a is the fraction of the incident flow q_o removed by the drain. Combining Eq. 2 and Eq. 3 yields

$$t/T = (1 - a) \ . \qquad (4)$$

If the capillary rise in a system is negligible when compared to the thickness of flow T, $T = H$, where H is the height of the original water table above the impermeable layer. If an interceptor drain installed in the system is completely effective, $t = h$, where h is equal to the height of the water in the drain above the impermeable layer. Using the electrical analogue, Childs found, that on slopes as steep as 1 in 30, both open ditch and covered tile line interceptor drains are completely effective. When the impermeable bed slope increases to 1 in 3, control by tile drain is imperfect and the efficiency of an open ditch may or may not be complete. His tests indicated that when the drain floor coincides with the impermeable bed, an open ditch is a complete interceptor; while a tile drain allows an additional 10 per cent of incident flow to pass to the downhill side.

When the capillary rise in a system is not negligible, $T > H$. According to the work of Mavis and Tsui, a reasonable assumption would be that

$$T = H + \frac{2}{3} C \ , \qquad (5)$$

where C is the height of capillary rise. If an interceptor drain installed in the system is completely effective an equation similar to Eq. 5 applies at the tail end,

$$t = h + \frac{2}{3} C \ . \qquad (6)$$

Shape of Drawdown Surface

Muskat (11) presented an analysis made by L. Hoff and E. Treffetz in 1921. This analysis treated the problem of the drainage of an infinitely long inclined water-saturated sand by a ditch dug at the top of the sand. The surface of seepage on the uphill bank of the ditch and the capillary fringe above the water table were both ignored in the study. The vertical distance D an interceptor drain lowers the water table on the downhill side of the drain was expressed by the equation, (11:298)

$$D = \frac{q_d}{q_o} , \qquad (7)$$

where q_d is the water intercepted by the drain and the other terms are as previously defined.

Part of the study presented by Mavis and Tsui (10) was conducted in a horizontal non-tilting flume without drain tiles. This flume was 18 feet long, 12 inches wide, and 34 inches deep. In other respects the flume used by Mavis and Tsui was very similar to the flume used by the author. Tests were conducted using three grades of sand such that the mean diameter of the coarse sand was three times that of the fine sand. The investigators found that the shape of the drawdown curve for like boundary conditions was independent of the gradation of the material.

These investigators also reported that the drawdown curve when $s = 0$ is not a parabola as indicated by Dupuit's formula,

$$\frac{H^2 - y^2}{H^2 - h^2} = \frac{(L - x)}{L} . \qquad (8)$$

In Eq. 8, H is the headwater depth above impervious datum; h is the tailwater depth above impervious datum, L is the length of the prism, and y is the elevation of the drawdown curve above the impermeable layer at a horizontal distance x from the drain. They revised the formula to the empirical form,

$$\frac{H^2 - y^2}{H^2 - h^2} = \left[\frac{L - x}{L}\right]^n , \qquad (9)$$

in which

$$n = 1/6\ (5 + h/H) \qquad (10)$$

This new formula agreed closely with their experimental data.

Childs (2), using the electrical analogue, found that the distance a given degree of control is exerted on the uphill side of an interceptor drain is dependent upon the slope of the impermeable bed. He maintained that, within certain limits, the degree of control extends a distance which is inversely proportional to the slope of the impermeable bed.

In 1953, Donnan (3) discussed a formula derived by R. E. Glover for the determination of the shape of the upstream drawdown curve for any interceptor drain condition of depth of flow or depth of interception. The formula was taken from the unpublished writings of Glover, and is the result of purely mathematical analysis. In deriving the formula, the assumption was made, but not stated, that at some point upstream from the interceptor drain the drawdown curve for the drained system coincides with the original water table. In effect the assumption was, that for equal head water depths, the flows are equal in the system before and after the installation of an interceptor drain.

The derivation of Glover's formula was begun by equating the Darcy equation for flow on the system at an infinite distance upstream from the drain to the flow in the system near the drain, yielding for steady flow,

$$KHs = Ky \left[s + \frac{dy}{dx} \right] . \qquad (11)$$

In Eq. 11, all the terms are as defined previously and slope datum is used. Canceling K from each side of Eq. 11, integrating, solving for the constant by letting $y = h$ when $x = 0$, and rearranging terms yields the final equation in the form,

$$x = \frac{H \log_e (H - h)/(H - y) - (y - h)}{s} . \qquad (12)$$

Donnan pointed out that the formula yields infinite values for x when $s = 0$ or $y = H$. Equation 11 becomes trivial when $s = 0$ and thus Eq. 12 fails at this point. In Eq. 12, when $y = H$, $x \rightarrow \infty$, which is in accordance with the original assumption.

Considerations Relating to the Model Study

Darcy's Law, Eq. 1, may be rewritten as given below, Israelsen (6:226);

$$V = \frac{K' \gamma}{\mu} \cdot \frac{\Delta h}{L} . \qquad (13)$$

In Eq. 13, K' is the permeability of the porous medium and has the physical dimensions of area or of length squared; γ is the specific weight of the fluid; μ is the dynamic viscosity; and V, Δh, and L are defined as before. At low temperatures the specific weight of water is nearly constant and may be included in the permeability constant. The viscosity, except for temperature variations, may also be included in the permeability. These concepts give rise to the accepted practice of correcting seepage flow measurements to a standard temperature by applying an appropriate viscosity correction.

Robinson and Rohwer (12:82), in a field study reported in 1954, found that applying temperature corrections for viscosity did not improve the correlation of the data. They indicated that ". . .seepage data should not be corrected for viscosity for the purpose of making comparisons with other data." They believed that deviations resulting from the accepted practice of applying a correction for viscosity to a standard temperature were possibly due to an air-water relationship ". . .involving the solubility of air in water and the process of solution or dissolution of the soil air."

A requirement of the porous medium used in the flume of the writer was that excessive turbulent flow should not be encountered. This was necessary in order that Darcy's Law, Eq. 1, apply at a particular temperature. Kiefer (8), in 1952, proposed that the point of transition between laminar and turbulent flow be described by a pore Reynolds number of approximately six. The pore Reynolds number R_p was expressed by

$$R_p = \frac{V \rho \, d_e}{\mu (1-n)} \; ; \qquad (14)$$

where V is the bulk velocity; μ is the dynamic viscosity of the fluid; ρ is the fluid density; n is the porosity of the porous medium; and d_e is the effective diameter of the particles in the porous medium. The effective diameter was calculated according to

$$d_e = \frac{\Sigma P}{\Sigma P / \sqrt{d_1 \, d_2}} \; , \qquad (15)$$

where P is the per cent by weight of material passing a screen of opening d_1 and retained on a screen of opening d_2 .

In setting up a laboratory model, the descriptions of equipment and methods used by other investigators was helpful. The work of several investigators who were concerned with various phases of the study of seepage flow through porous media was reviewed. Donnan (4) conducted a large scale model study of tile spacing for grid drain system. Karaki (7) conducted a model study of seepage flow from a canal to a shallow water table. The work by Mavis and Tsui (10) on seepage through sand prisms has been discussed above. From the work of these and other investigators, ideas for construction of the model and conducting the experiment were obtained.

Chapter III

THEORETICAL ANALYSIS

Certain rational conclusions can be made regarding the effect of installing an interceptor drain in a saturated medium of uniform slope. Some of these follow from simple deduction and others from dimensional analysis. These conclusions were very useful in designing the experiment and in selecting methods for analyzing the experimental data.

Drawdown Curve and Discharge Relationships

Simple deduction yields some information regarding the relationship of the discharge of the system and the drawdown curve. Consider an infinitely long porous medium of uniform slope s underlain by an impermeable boundary at constant depth, Fig. 1. Through this medium, water flows by seepage at uniform depth H corresponding to discharge q_o per unit width. Any finite portion of length L of the infinitely long porous medium corresponds to every other finite portion having the same length. Such a portion is shown in Fig. 2. If an open ditch or tile drain is placed at the lower end of the finite portion of the medium shown by Fig. 2, and the water at the upper end is maintained at the original depth H, the tail-water will be lowered to some new elevation corresponding to h_3, Fig. 3. The resulting steady-state discharge q_t must be greater than q_o, i.e., $q_t > q_o$; this may be inferred since more energy has been made available to the system by installation of the drain and since all other flow factors remain unchanged. Even though L, Fig. 3, be increased without limit by extending the upstream end, any drained system expends more energy in flow than the corresponding undrained system, thus q_t always exceeds q_o. This difference, $q_t - q_o$, becomes extremely small as L approaches infinity. Evidently some quantity of water q_e, in excess of q_o, must be supplied to the system at a finite distance L if equilibrium is to occur under drainage. One can conclude, therefore, that the point of tangency of the drawdown curve to the original water surface will be at infinity unless the total recharge q_r exceeds q_o. In the latter case, the drawdown curve may be expected to intercept the original water surface at the finite distance L.

The problem of locating the point where the drawdown curve induced by an open drain will intercept the original water table can be approached by another method. Fig. 4 is similar to Fig. 3 except that Fig. 4 shows several sections having different lengths L of ℓ, 2ℓ, . . .$n\ell$. Darcy's Law,

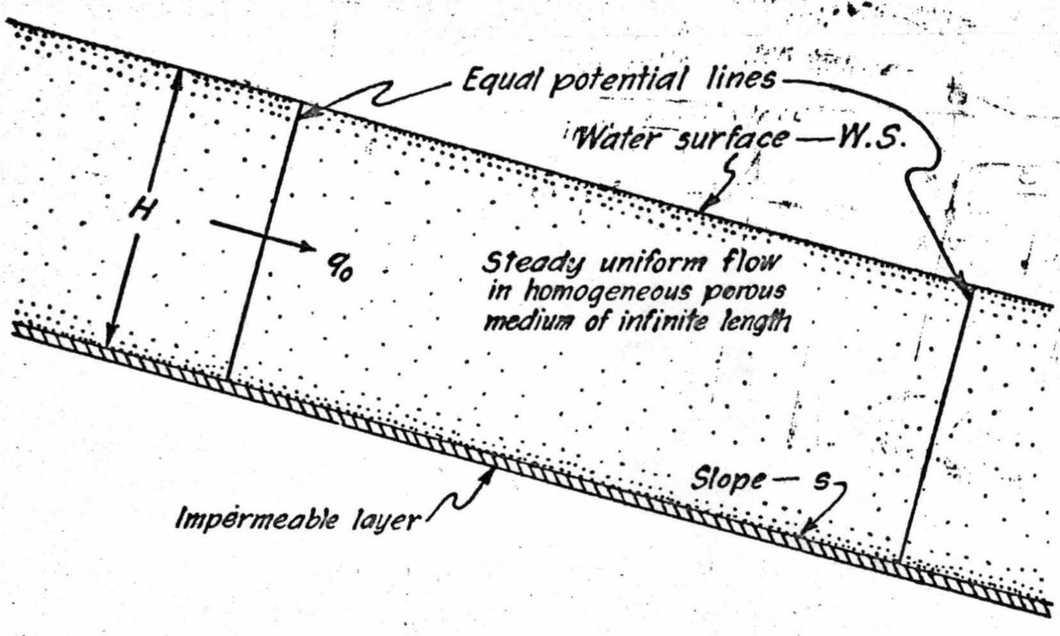

Fig. 1 Section of an infinitely extending medium.

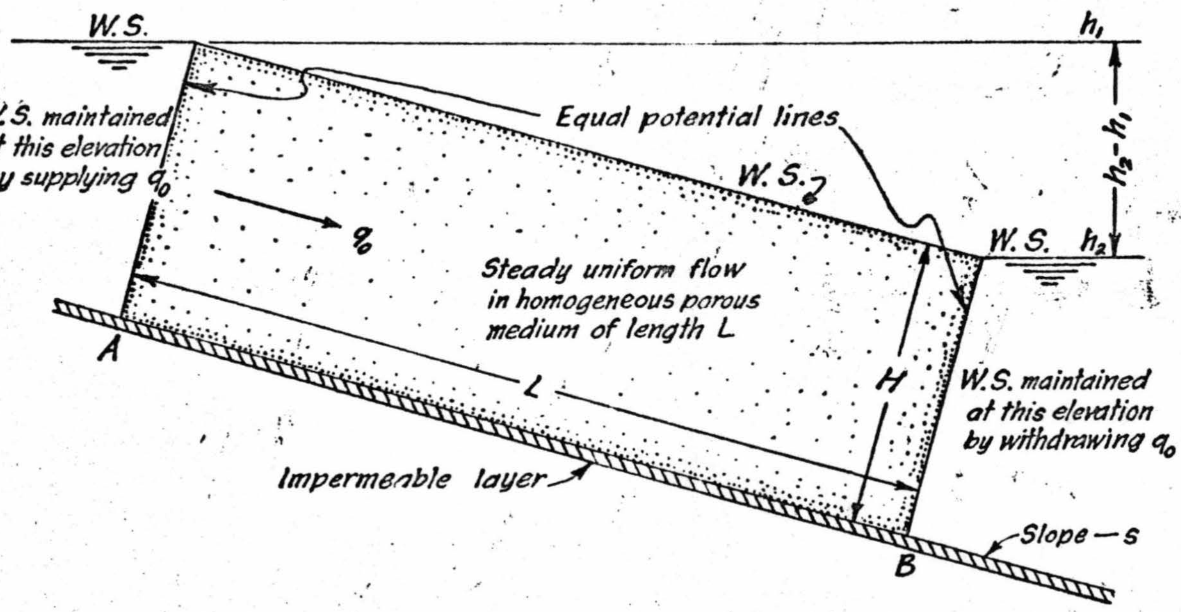

Fig. 2 Equivalent finite section.

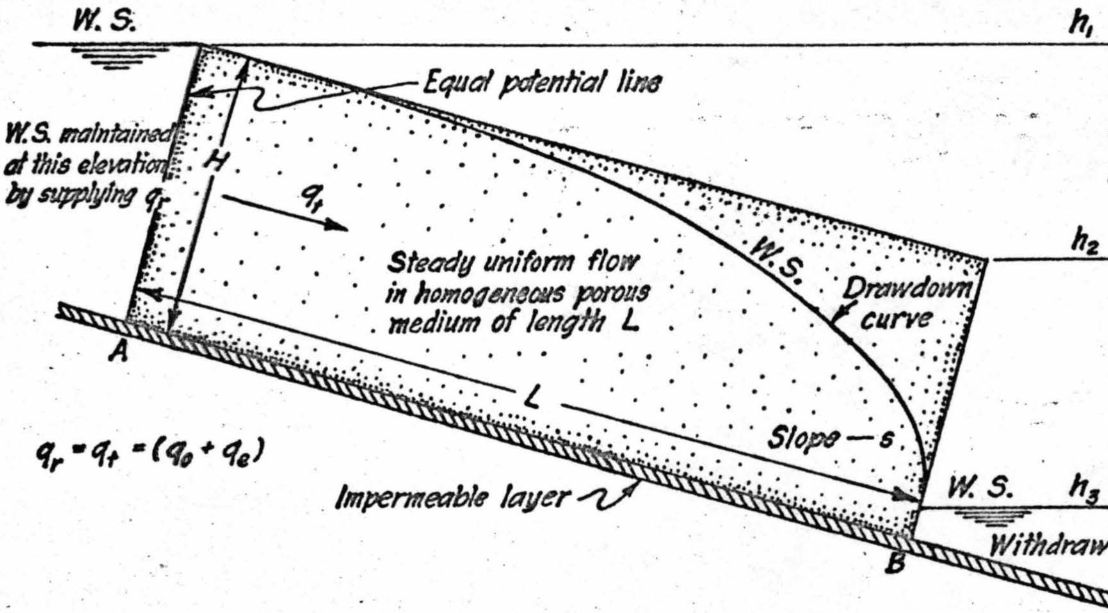

Fig. 3 Equivalent finite section with drain.

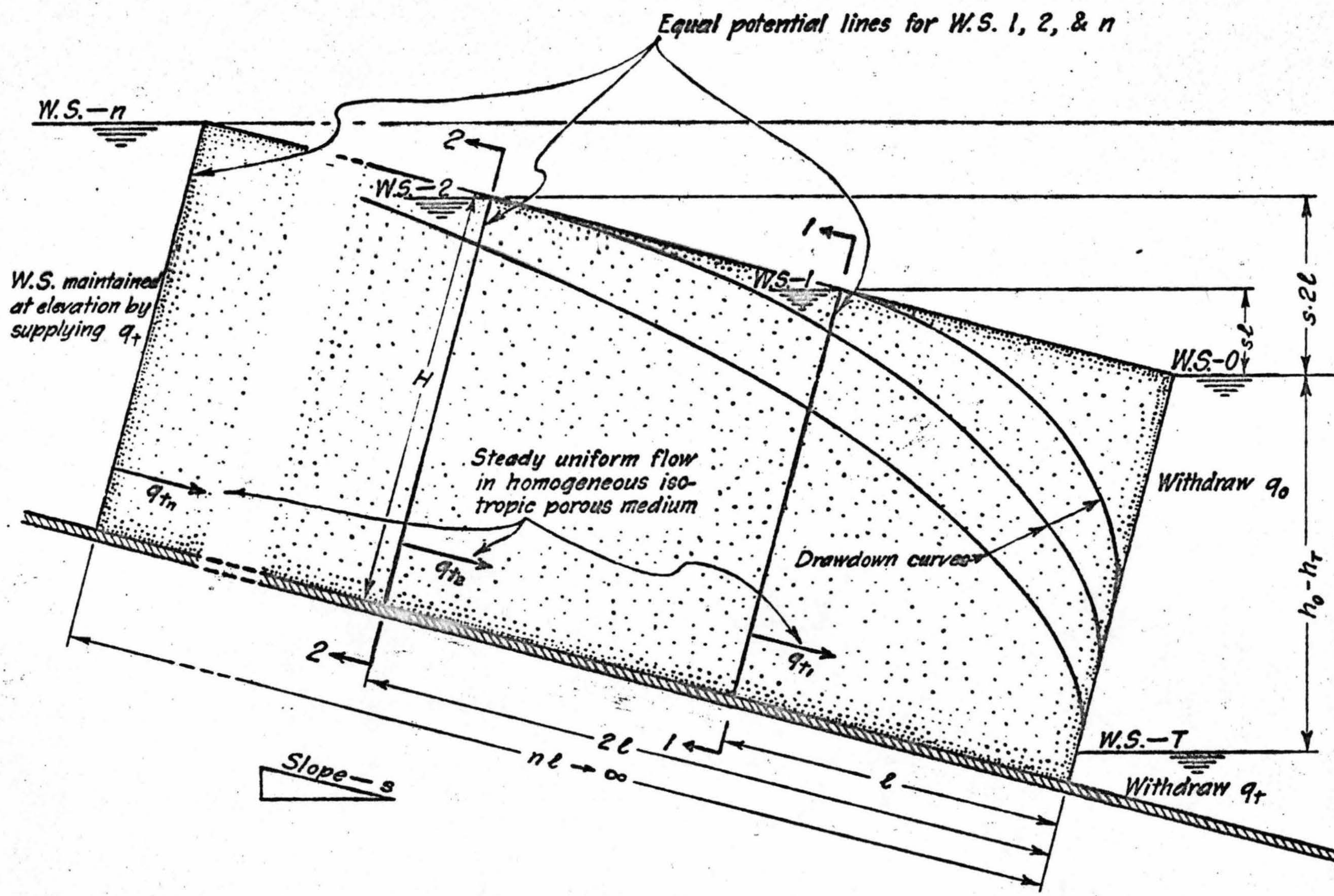

Fig. 4 Relationship of drawdown curve to length of section.

$Q = KAi$, demonstrates that the undrained discharge, Fig. 4, is the same regardless of the length of section chosen. Let $q_o = q_{o1} = q_{o2} = q_{on}$ be this discharge. The number in the subscript identifies the flow past the corresponding section in Fig. 4. Assume, then, that the tailwater surface is lowered from O to T by installation of artificial drainage. This results in a new discharge $q_t > q_o$.

Assume first that the drawdown curve intercepts the original water surface at cross section 1 with a corresponding discharge q_{t1}. Next assume that the drawdown curve intercepts the original water surface at cross section 2 with a corresponding discharge q_{t2}. The resulting drawdown curves are shown in the figure. Obviously $q_{t1} > q_{t2}$ since flow system 1 expends more energy than does flow system 2 in passing through the final segment of length, ℓ. This is shown in Fig. 4 by the relatively higher position of the drawdown curve of system 1 at cross section 1 as compared to that of system 2 at cross section 1. Similarly with successive increases in length of section:- 3ℓ, 4ℓ, . . . $n\ell$; $q_{t1} > q_{t2} > q_{t3} > \cdots q_{tn} > q_o$. As n increases without limit, $q_{tn} \rightarrow q_o$; thus, for any constant slope, the intercepting point will be at an infinite value of L if $q_r = q_o$. Conversely L will be finite and vary inversely as q_r for $q_r > q_o$. Fig. 5 is a general plot illustrating the foregoing conclusion.

One may deduce, therefore, that for the case of a drain intercepting flow down a constant slope, the drawdown curve caused by the drain will intersect the original water table surface at some finite distance only if there exists a source of recharge to the system within a finite distance such that $q_r > q_o$.

Flow Analysis

For the steady state system with no drain, Fig. 2, the relationship between the variables for a section of unit width may be expressed by

$$q_o = \phi_1 (s\ ,\ K\ ,\ H); \qquad (16)$$

where K is the hydraulic conductivity and the other terms are as defined previously. The hydraulic conductivity includes all of the fluid properties as well as the micro-properties of the medium, such as pore size and shape, porosity, etc. Choosing K and H as repeating variables, by dimensional analysis,

$$q_o/KH = \phi_2 (s) \quad . \qquad (17)$$

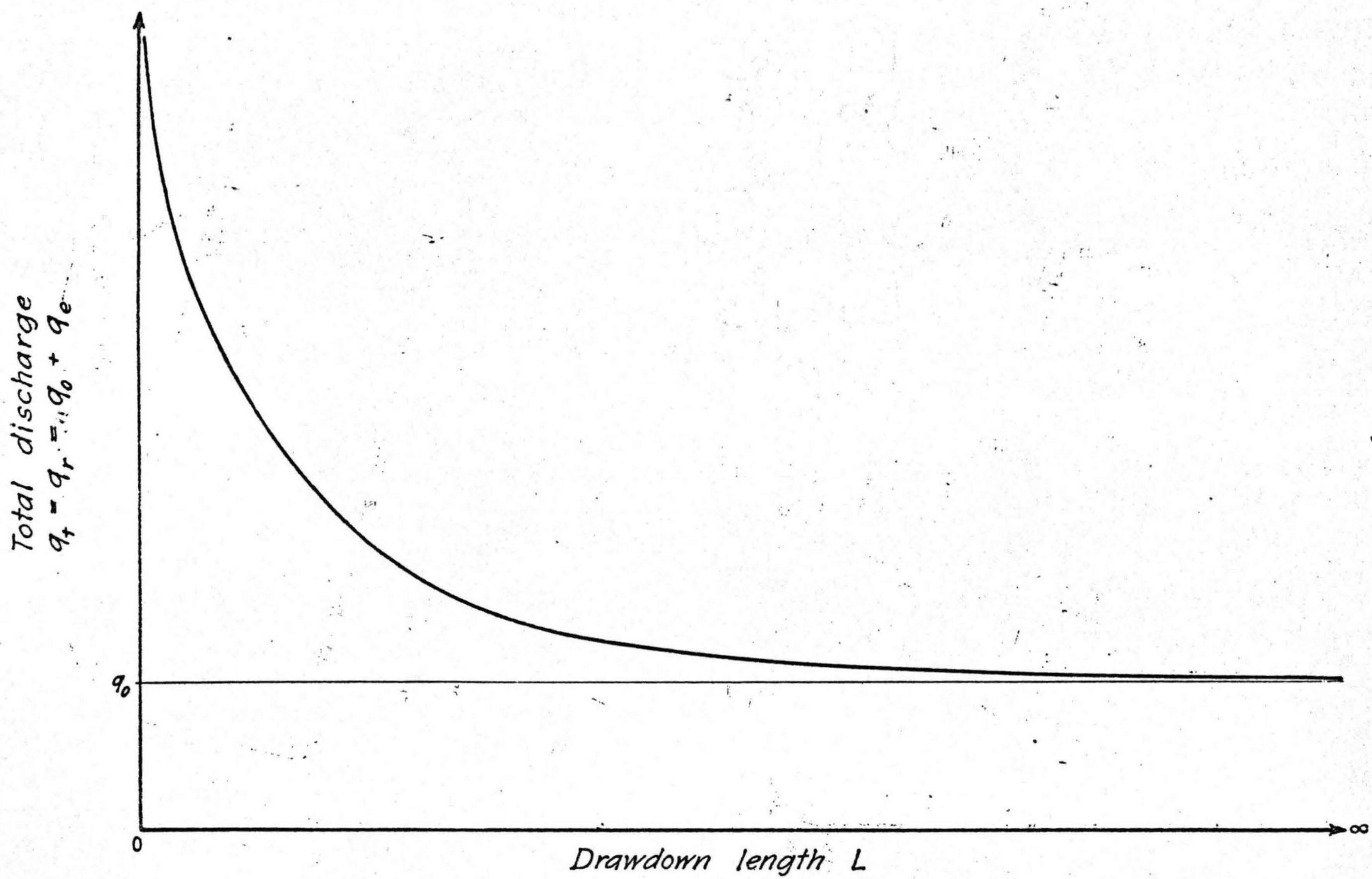

Fig. 5 Discharge as a function of drawdown length.

Darcy's experiments demonstrated empirically that ϕ_2 (s) = s; thus, for the system under consideration.

$$q_o = KHs \ . \qquad (18)$$

For this simple system, s equals the hydraulic gradient i .

With a drain installed as in Fig. 3, the relationship between the pertinent variables for a section of unit width, Fig. 6, might be expressed by,

$$q_t = \phi_3 \ (s \ , \ K \ , \ r \ , \ L \ , \ H) \ . \qquad (19)$$

In Eq. 19, r is the radius of the drain, and the other terms are as previously defined. These variables may be rearranged to yield,

$$q_t = \phi_4 \left[s \ , \ K \ , \ r \ , \ (H + sL), \ H \right] . \qquad (20)$$

Choosing K and H as repeating variables as before,

$$q_t/KH = \phi_5 \left[(H + sL)/H \ , \ r/H, \ s \right] . \qquad (21)$$

For an open drain, r/H could be eliminated. In the case of a tile drain, as long as the drain is of sufficient capacity to handle the discharge, r probably does not appreciably affect the flow system except perhaps in the immediate neighborhood of the drain. Accordingly r/H may be tentatively eliminated from the analysis, thus

$$q_t/KH = \phi_6 \left[(H + sL)/H \ , \ s \right] . \qquad (22)$$

One may infer from Eq. 22 that $q_t \neq q_o$ since, by comparing Eqs. 17 and 22, q_t appears to be dependent upon an additional parameter, (H + sL)/H. Dividing q_t/KH by the dimensionless parameter s yields,

$$q_t/KHs = \phi_7 \left[(H + sL)/H, \ s \right] . \qquad (23)$$

Darcy's equation yields $KHs = q_o$, uniquely relating K and s . By substituting q_o for KHs, elimination of s as a separate variable appears possible, thus

$$q_t/q_o = \phi_8 \left[(H + sL)/H \right] . \qquad (24)$$

In Eq. 24, q_t/q_o is always greater than unity.

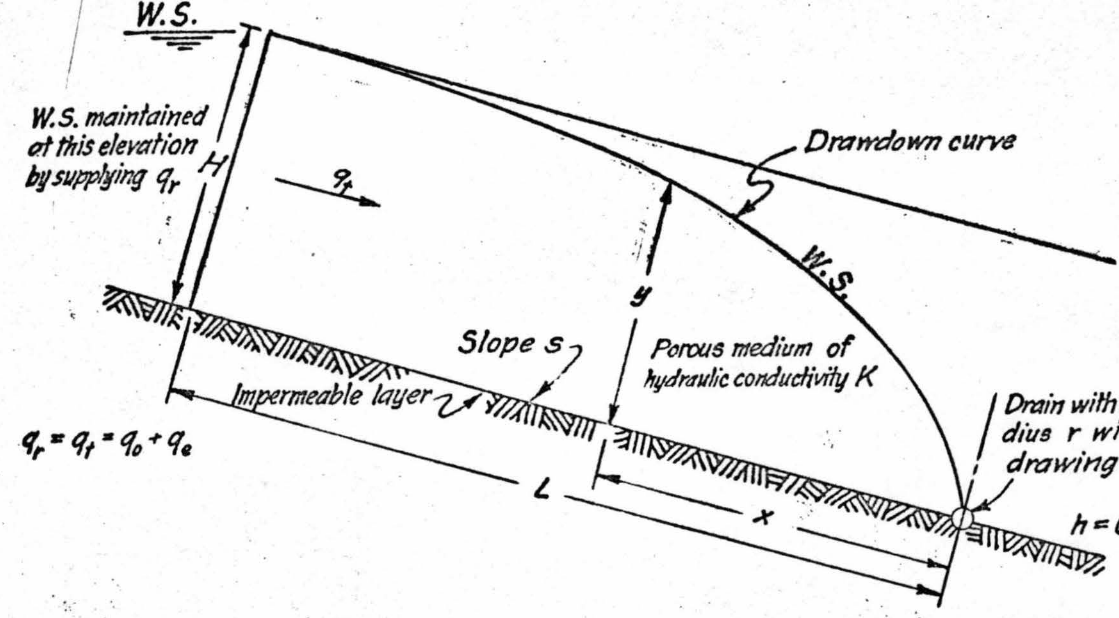

Fig. 6 Definition sketch for h equal to zero.

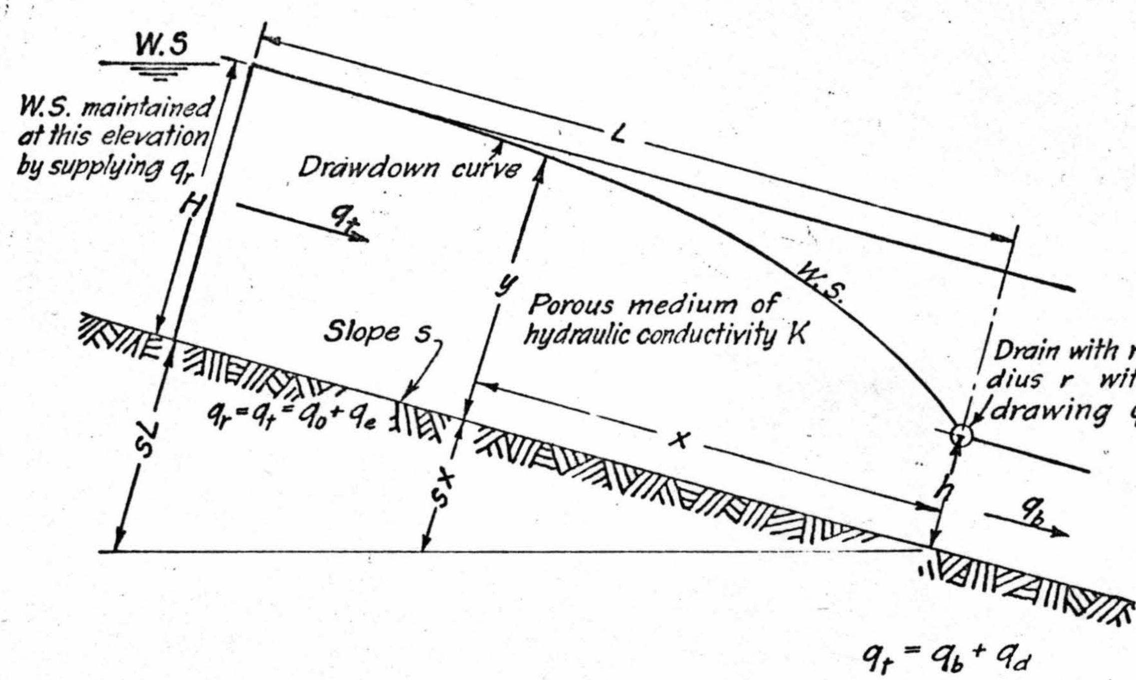

Fig. 7 Definition sketch for h greater than zero.

If the tile is installed above the impermeable layer at distance h, the system illustrated by Fig. 7 results. The additional variable h is introduced, and the general functional relationship may be expressed by:

$$q_t = \phi_9 \left(s, K, r, L, H, h \right),$$

or

$$q_t = \phi_{10} \left(s, K, r, (H + sL), H, h \right). \quad (25)$$

Choosing the repeating variables K and H and proceeding as before yields, if r/H is discarded:

$$q_t/KH = \phi_{11} \left[s, (H + sL)/H, h/H \right]. \quad (26)$$

Remembering that $q_o = KHs$, as before,

$$q_t/q_o = \phi_{12} \left[(H + sL)/H, h/H \right], \quad (27)$$

When the drain is installed as in Fig. 6, $h = 0$ and Eq. 27 reduces to Eq. 24.

If a tile is installed above a horizontal impermeable layer; i.e., if $s = 0$, the discharge intercepted by the drain q_d is equal to q_t since there is no bypass flow q_b and the general functional relationship may be expressed by:

$$q_d = \phi_{13} \left(K, L, H, h, r \right). \quad (28)$$

Choosing the repeating variables H and K and proceeding as before, if r/H is discarded:

$$q_d/KH = \phi_{14} \left(L/H, h/H \right). \quad (29)$$

<u>Shape Analysis</u>

If an interceptor drain results in a steady state system, the geometry should be independent of mass and time; that is, it should be expressible in terms of quantities having the dimension of length only. With a drain installed as in Fig. 7, the relationship between the various geometric variables might be expressed by,

$$\phi_{15} \left(H, h, L, x, s, y, r \right) = 0 \quad (30)$$

In Eq. 30, y is the elevation of the drawdown curve above the impermeable layer at a distance x from the drain. When small slopes are used horizontal datum and slope datum are interchangeable. Rearranging the variables, and dropping r as before,

$$\phi_{16}\left[H\ ,\ h\ ,\ L\ ,\ (H + sL)\ ,\ x\ ,\ (y + sx)\right] = 0 \quad (31)$$

Choosing L as the repeating variable,

$$\phi_{17}\left[H/L\ ,\ h/L\ ,\ (H + sL)/L\ ,\ x/L\ ,\ (y + sx)/L\right] = 0 \quad (32)$$

In order to acquire more useful terms for the purpose of this study, (y + sx)/L, H/L, and h/L can be divided by (H + sL)/L and (H + sL)/L retained to yield,

$$\phi_{18}\left[(y + sx)/(H + sL)\ ,\ x/L\ ,\ H/(H + sL)\ ,\ h/(H + sL),\ (H + sL)/L\right] = 0 \quad (33)$$

The parameter (H + sL)/L is the ratio of the total energy available and the length. The length is included in the term x/L; and the total energy available, H + sL, is included in the denominator of all of the other parameters. Because the variables involved appear to be represented otherwise, the parameter (H + sL)/L does not appear to be independent and may be discarded. If this be true,

$$\phi_{19}\left[(y + sx)/(H + sL),\ x/L,\ H/(H + sL),\ h/(H + sL)\right] = 0. \quad (34)$$

Chapter IV

APPARATUS AND PROCEDURE

The purpose of this thesis is to acquire more knowledge regarding the characteristics of interceptor drains. The boundary conditions are readily defined, thus, the problem lends itself well to model study. This chapter describes the equipment, materials, and procedure used in this phase of the investigation.

The laboratory equipment used for the model study consisted of a large tilting flume of rectangular cross section filled with a coarse sand, a water supply system, a network of piezometers connected to manometer tubes, a system of drains, and a means of measuring the water flowing from the flume.

Equipment

Since the purpose of the laboratory model study was to determine the relationships between the dimensionless variables defined in the previous chapter, the laboratory equipment was designed to meet the following requirements.

1. It was necessary that the flume be straight in alignment, with a uniform cross section, and provided with a means for tilting.

2. It was required that the flume possess sufficient strength and tightness to withstand the load of sand and water without excessive deformation or leakage.

3. The upstream end of the flume needed a water supply system designed to maintain a steady input flow at any desired depth.

4. A means for determining the average water temperature was required.

5. It was necessary to provide a method for determining the position of the drawdown curve established under any set of boundary conditions.

6. A system of tile drains for imposing any one of several different drainage conditions, coupled with a means of maintaining any desired water depth at the downstream end of the flume was required.

7. A method of measuring discharges was required.

Fig. 8 shows the general lay-out of the flume used in the model study. Its dimensions were 2 feet wide, 4 feet deep, and 72 feet long. It was supported by steel I-beams mounted on adjustable jacks. The walls and floor of the flume were made from 1/2-inch plywood which was attached to the 2x4-inch framing with screws. The joints in the flume were made watertight with rubber gaskets. All wooden parts received two coats of paint before being assembled. The flume was reinforced against warping by 2x4-inch wood braces bolted in place. In addition to the wood braces, $1\frac{1}{2}$ x $1\frac{1}{2}$ x 1/8-inch angle iron walers were placed longitudinally down each side of the flume. These walers were held in place by 3/8-inch steel tie rods passing through the flume. The top of the flume was kept from spreading by means of 1 x 1/8-inch steel tie bands. These bands were bent around and connected with turnbuckles to the floor outside the flume. The turnbuckles were adjusted to keep the walls of the flume perpendicular to the floor.

The upstream end of the flume was equipped with a head box, and the downstream end, with a tail box. These boxes were constructed by placing metal filter screens about 2 feet from each end of the flume. Each filter screen consisted of a 16-mesh brass screen supported by a series of increasingly larger screens attached to a steel frame. The distance between the head-box and tail-box screens was 811 inches. These screens provided a means by which water could enter or leave the ends of the porous medium with very little head loss. Each box was provided with a drain in its bottom. The drains were connected to adjustable overflow cans with 2-inch flexible tubes. Any desired combination of head-box and tail-box depths could be maintained by sliding the adjustable overflow cans to the proper positions and locking them in place with thumb screws. The overflow water fell into a sump beneath the laboratory floor.

A small centrifugal pump was used to pump water from the sump system under the laboratory into the head box. The quantity of water supplied to the head box was adjusted by means of a bypass valve and by means of various settings of a valve between the pump and the head box. Enough water was supplied to the head box to maintain a small amount of overflow. A small quantity of copper sulfate was added to the water on the head box to prevent the growth of algae.

The head box and the tail box were each supplied with a thermometer. The arithmatic mean of the head-box and the tail-box temperatures was assumed to be equal to the average water temperature in the flume.

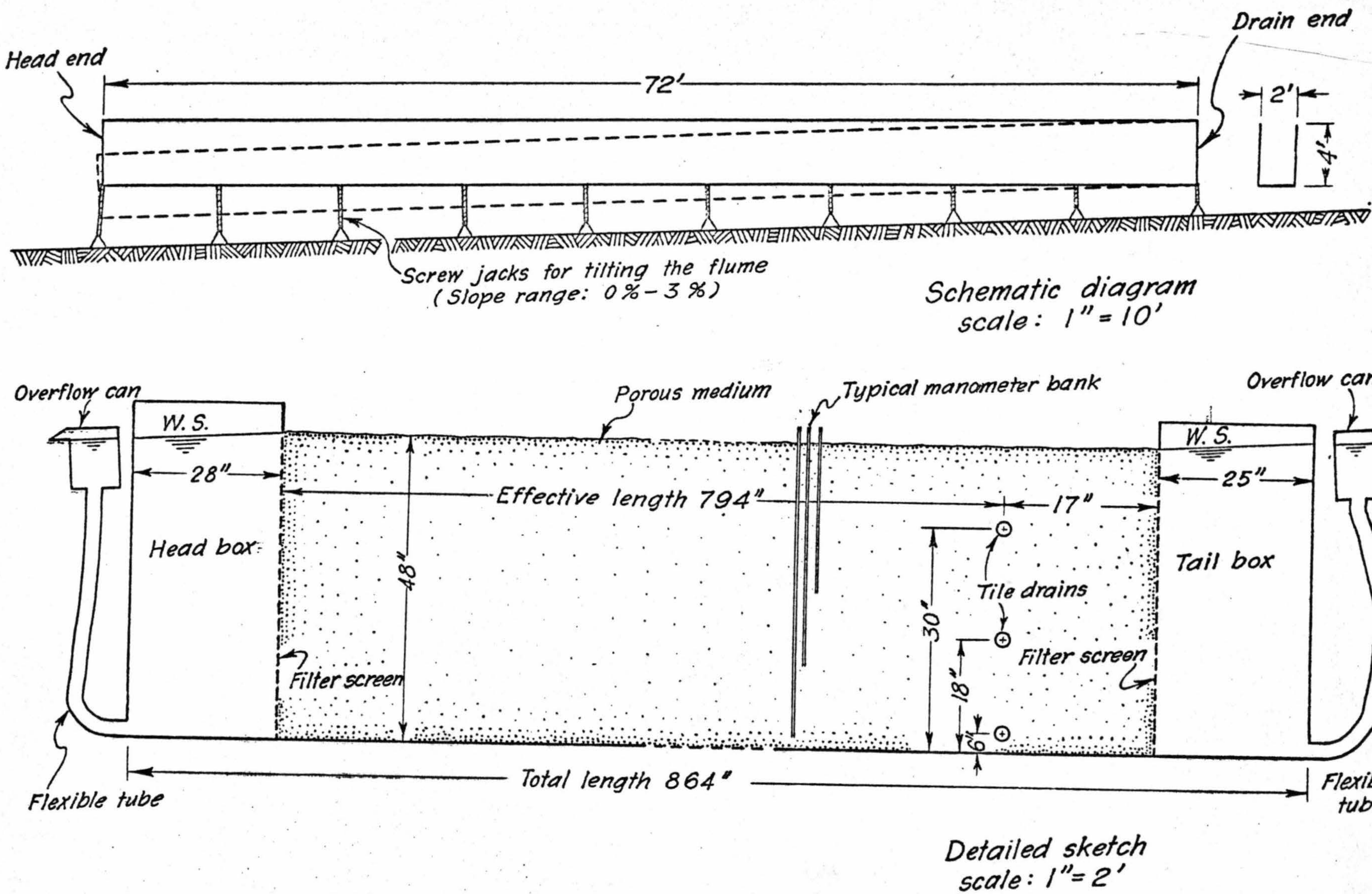

Fig. 8 Lay-out of tilting flume.

The flume was provided with a total of four drain tiles, three of which are shown in Fig. 8. Each drain tile was placed so that it penetrated one wall of the flume and extended through the porous medium to the opposite wall. Each tile was set perpendicular to the walls of the flume and parallel to the plane of the floor.

A single tile was provided with its centerline 16 inches above the floor, 482 inches from the head-box filter screen, and 329 inches from the tail-box filter screen. This tile was used principally to determine the shape of the drawdown curve downstream from a tile, and the percentage of the total flow intercepted by a drain tile placed above the floor. Control of both upstream and downstream water depths was possible because of the central location of this tile. This tile is designated by T/2 , and it was installed in the flume after the porous medium had been placed.

A bank of three tiles was provided with its vertical centerline 794 inches from the head-box filter screen and 17 inches from the tail-box filter screen. The centerline of the top tile was 30 inches above the floor of the flume; the centerline of the middle tile, 18 inches above the floor; and, the centerline of the bottom tile, 6 inches above the floor. These tiles were designated as follows: TT, MT, and BT. They were used primarily to determine the upstream drawdown curve.

Each of the four drain tiles was constructed of a piece of electrical conduit having a 2-inch inside diameter. The 2-foot portion of each drain tile extending through the flume and surrounded by the porous medium was provided with a series of $1\frac{1}{4}$ x 1/16-inch saw cuts. The single tile, T/2 , contained 200 cuts which removed approximately 15 per cent of the wall areas of the conduit. The three tiles in the tile bank each contained 100 saw cuts which removed approximately 8 per cent of the wall area of each conduit. The saw cuts in all tiles were located in a systematic manner so that water could enter the tile with equal resistance from any direction. A 2-inch flexible tube was provided for each tile to convey drained water to the sump or to the weighing tube. A tile was shut off by raising the end of the flexible tube above the free water surface inside the flume.

The rate of flow from the flume was measured by collecting a volume of water in a 12-gallon tub for a predetermined length of time. The quantity of water collected was determined by weight.

The shape of the drawdown curve was determined by means of water manometers. The piezometer openings for the manometers were located in banks along a side wall of the flume, in the bottom of the head and tail boxes, and in the bottom of each tile. The openings were flush with the inside wall of the flume and covered with wire cloth. The banks of piezometer openings along the side wall of the flume were spaced at 10-foot intervals except in the near vicinity of the drains. In the piezometer bank 1 foot upstream from the three downstream tiles the openings were at 3-inch intervals. In other piezometer banks near the tiles the interval was 5 inches; 10-inch intervals were used in piezometer banks 10 feet or more from the tiles.

Plastic tubing was used to connect the outlet ends of the piezometer openings to one of the four manometer boards, and all manometer readings were referred to the bottom of the flume as datum.

Porous Medium

In choosing a porous medium for the flume and placing the pervious material in the flume, the following considerations were necessary:

1. The entire mass of poraus medium filling the flume needed to be as nearly homogeneous and isotropic as possible.

2. The hydraulic conductivity of the porous medium needed to be sufficiently large in order that, under any particular set of boundary conditions, equilibrium would be established quickly and flows would be large enough to facilitate accurate measurement. Moreover, the hydraulic conductivity of the porous medium needed to be sufficiently small so that turbulent flow would not occur under the steepest slope conditions.

3. It was desirable that the capillary rise in the material be small in order that the effects of capillary flow be minimized.

4. Since approximately 20 cubic yards or 30 tons of material were required to fill the flume, a nearby commercial source of acceptable material was desirable.

The porous medium used was obtained from natural deposites at the Broughton Gravel Pit located about two miles east of Greeley, Colorado. This material was a coarse, decomposed granitic sand deposited by the Cache La Poudre River. During processing, this sand had been screened and washed twice and was very clean.

The following properties were determined from a sieve analysis of the sand. The uniformity coefficient of the sand, d_{60}/d_{10} was 2.0. The computed effective diameter of the sand, d_e was 0.103 inches (Eq. 15). Eighty-five per cent of the material passed through a U. S. No. 6 screen (0.131-inch square openings) and was retained on a U. S. No. 16 screen (0.065-inch square openings).

The hydraulic conductivity, when measured in a 5-inch diameter cylinder one foot long, was found to be 0.025 fps at 20° C. With a porous medium having this hydraulic conductivity, the flow through the flume was expected to range between 0.00 and 0.007 cfs.

According to Kiefer (8), the critical pore Reynolds number, or condition where turbulent flow begins, is approximately six. Assuming porosity of 35 per cent and water temperature of 15° C. in Eq. 14, a pore Reynolds number greater than six would be encountered only when the cross sectional area of flow was less than one square foot. This small flow area could occur only in the near vicinity of a drain and turbulence near a drain is of little adverse consequence. Also, owing to the irregular and capillary nature of the flow channels in the sand, the deviation from viscous flow would develop gradually. This was demonstrated experimentally with sand in a 5-inch cylinder. These experiments yielded no great deviation from Darcy's law with a pore Reynolds number as high as 15.

The entire mass of sand was placed in the flume before any effort was made to compact it. After placement, the sand mass was saturated, and compacted by probing with a high speed vibrator. The 5-foot probe of the vibrator was inserted vertically through the saturated sand at uniform intervals of one square foot of area. Hydraulic conductivity tests, with water at various depths in the tilted flume, however, indicated that non-uniform compaction occurred using this method. The hydraulic conductivity of the lower portion of the sand was found to be greater than that of the upper portion. This phenomenon was attributed to a binning effect and/or null points in the probe of the vibrator. Systematic vibration of the walls of the flume after saturating the sand with water remedied this undesirable condition.

The amount the sand settled during the study was negligible. This was determined by placing metal bench marks in the surface of the sand and measuring their perpendicular height above the floor of the flume at the beginning and at the end of the study. The height of each bench mark above the floor of the flume remained constant during the study.

The porosity of the in-place sand was measured at the termination of the study and found to be 36.8 per cent. For this purpose, an accurately measured volume of approximately 2 cubic feet was excavated from the flume and weighed. Moisture content and specific density determinations were made, and the porosity computed.

With the porosity of the sand at approximately 35 per cent, the capillary rise ranged between 3/4 and $1\frac{1}{2}$ inches. Capillary rise tests were conducted in a 5-inch diameter clear plastic cylinder. The general outline of the capillary fringe was observed by distinguishing between the voids filled with water and those filled with air. The height of rise was considered to be equal to the height of this fringe above the free water surface. About 95 per cent of the capillary rise took place within one hour.

The specific yield of the sand in the flume was 25.7 per cent. The determination was made with the flume at zero slope. The flume was first filled with water and the depth of the free water surface determined. A volume of water was then drained from the flume and measured. After a period of 24 hours the depth of the free water surface was again determined. This was repeated for several successive depths of water. The temperature ranged from 14.0° C. at the beginning of the test to 18.5° C. at the end. The overall specific yield of the material between a depth of 40 inches and a depth of 16 inches was 25.7 per cent. The specific yield of each layer tested between these depths agreed closely with this value.

Calibration

A test run was started by setting the flume at a one per cent slope, by means of the adjustable screw jacks. The slope was checked both before and after the flume was filled with water in order to note inaccuracies resulting from deflections; however, no appreciable differences occurred. Water was then pumped from the sump into the head box. With the four drain tiles shut off, the head-box and tail-box water depths were set equal by properly adjusting the overflow cans. Thus, $H = h$, and the flow out of the tail-box overflow was equal to the undrained flow q_o . After the overflow had become uniform, the head-box and tail-box temperatures were recorded, the rate of flow was measured, and the water manometers were read. This procedure was repeated for several depths of water.

Discharge was plotted as a function of depth of water and a straight line relationship was found to exist between the two quantities. For all depths, the free water surface, as measured by the manometers, was nearly parallel to the floor of the flume except for a small mound near the upstream end of the flume.

These two phenomena indicated that the porous medium was, for all practical purposes, homogeneous and isotropic.

The discharge without a drain q_o was measured for several head-box-tail-box depths on all of the slopes used in the experiment. From Darcy's equation, this discharge should be a linear function of the depth of water in the flume, for any particular slope; and a linear function of slope, for any particular depth. Fig. 9 shows q_o/s as a function of water depth H. From Fig. 9, q_o corresponding to any particular drainage condition was determined.

Fig. 9 shows data both uncorrected and corrected for viscosity effects resulting from temperature. The average temperatures encountered during the one-half and one per cent slope readings were 11.2° C. and 14.3° C. respectively, and the average temperatures for the other slope readings were between these values. According to theory, the data corrected for temperatures should yield a straight line. The uncorrected data, however, fit a straight line relationship better than the corrected data. This is possibly the result of entrapped air as suggested by Robinson (12). In analyzing the results of this study, data were corrected for temperature in order to conform with convention. Actually, temperature did not vary greatly during a set of runs at a single slope. In the analysis, the flow factor is expressed as q_d/q_o, and, since the temperature correction factors for the flow with a drain and without a drain were nearly equal in most cases, the temperature effects approximately cancel. The hydraulic conductivity of the sand in place in the flume was equivalent to 0.038 fps at 20° C. The average uncorrected hydraulic conductivity was 0.034 fps at temperatures ranging between 10° C. and 16° C.

In Fig. 9, the extended plotted lines do not intercept the q_o/s axis at zero. This is an indication of some flow above the free surface, or of some piping along the bottom of the flume. Flow above the free surface is capillary flow. Mavis (10) states that capillary flow in sands travels with an average horizontal velocity equal to about 2/3 the horizontal velocity of the gravity water. The apparent flow at zero depth is equal to one-half the flow at a depth of one inch. Considering a capillary rise of $1\frac{1}{2}$ inches, an assumption that the apparent discharge at zero depth is capillary flow seems reasonable.

For each undrained setting, the free surface curve was determined from the manometer readings. If the medium were perfectly isotropic and homogeneous, this curve should be a straight line. In each case, however, there was an increase in depth about twenty feet in length in the upstream portion of the flume. The magnitude of this hump increased as the depth was decreased and/or the slope was increased. Correction of this deviation by removing

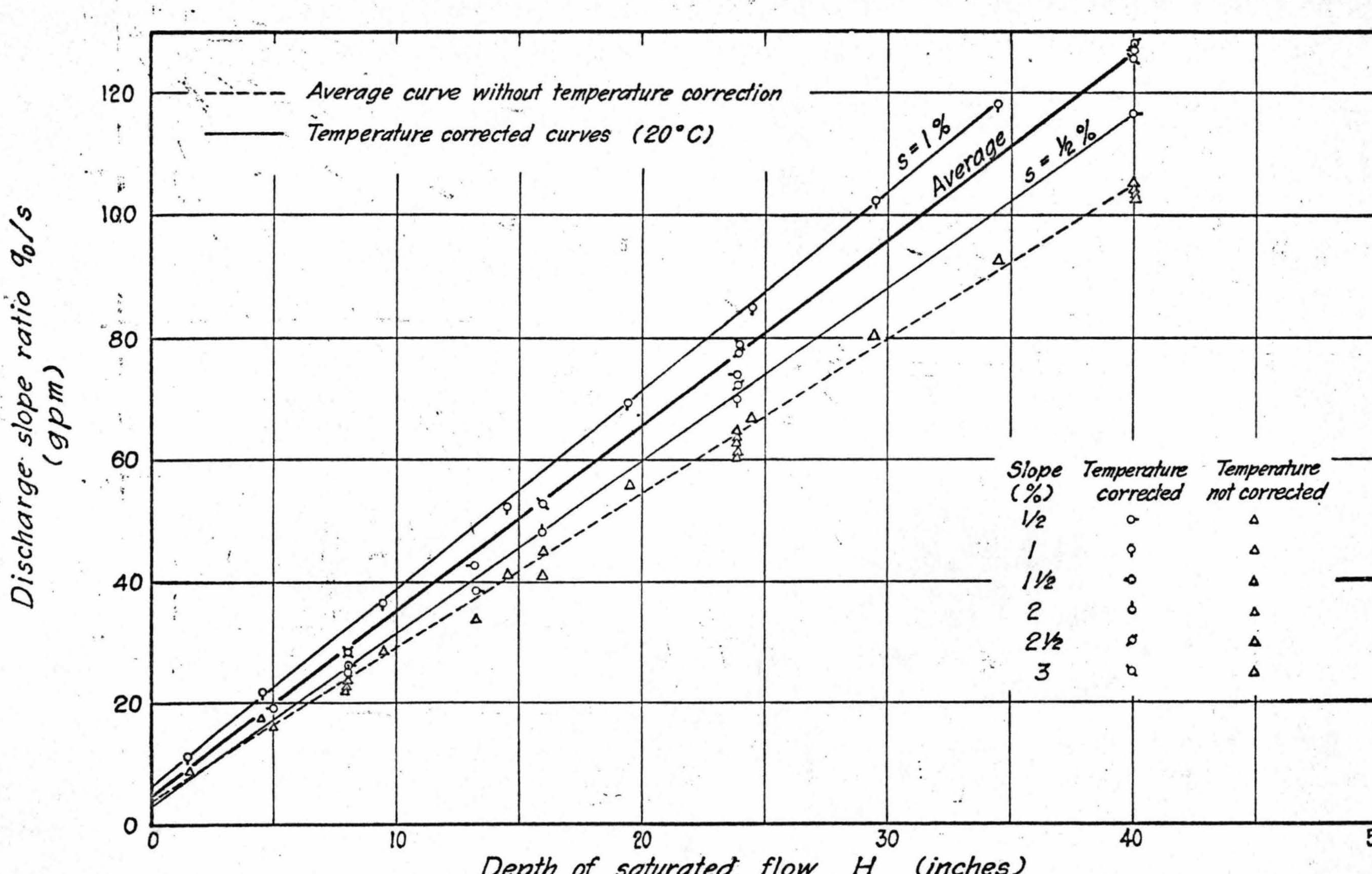

Fig. 9 Calibration of the flume.

and replacing the sand seemed to be impractical; however, a correction was made to the water depth, y , to allow for the inconsistency. This correction was made in the following manner. For each undrained run, that is $H = h$, the magnitude of the rise was determined at each piezometer bank by subtracting the head-box water depth from the water depth at the station. A plot was made for each piezometer bank and slope with $H = h$ as the absissa and the magnitude of the increase in depth as the ordinate. For a particular run, this curve gave the correction to be applied to an observed depth.

The T/2 tile, described earlier in this chapter, was installed primarily to determine what portion of the total flow is intercepted by a tile installed above the impermeable layer when the system downstream from the tile is infinitely long. This information was necessary in order to similate an extension of the flume by holding the water level at the proper elevation downstream from the bank of tiles.

Fig. 10 shows a typical drawdown curve using this tile. The water surface downstream from the tile was held parallel to the floor of the flume by adjusting the tailwater depth. The bypass flow q_b under these conditions was found to be approximately equal to the ratio of the height of the center line of the drain to the head water depth multiplied by the flow without a drain, i.e., $q_b = q_o h/H$. Further tests showed that adjustment of the tail-box water depth had little effect on the total flow through the system, although the division of flow changed. During each test where a tile was used the bypass flow was maintained such that $q_b = q_o h/H$. In order to compute the tile flow for analysis, the calculated bypass flow was subtracted from the total flow through the flume.

The manometer readings for all the piezometer openings in a given piezometer bank were identical except in the near vicinity of an operating drain. In taking the manometer readings, any piezometer opening was used in a given bank, except in those banks where the manometer readings for the highest and lowest submerged piezometer openings were appreciably different.

Procedure

The procedure used was such that the drawdown curve being investigated at any time was preceeded by a higher drawdown curve. A minimum of 3 hours was allowed after a given set of boundary conditions was imposed in order that equilibrium be established. The head-and tail-box water temperatures, the rate of flow from the flume, and the manometer readings were taken from each set of boundary conditions after equilibrium was established. The

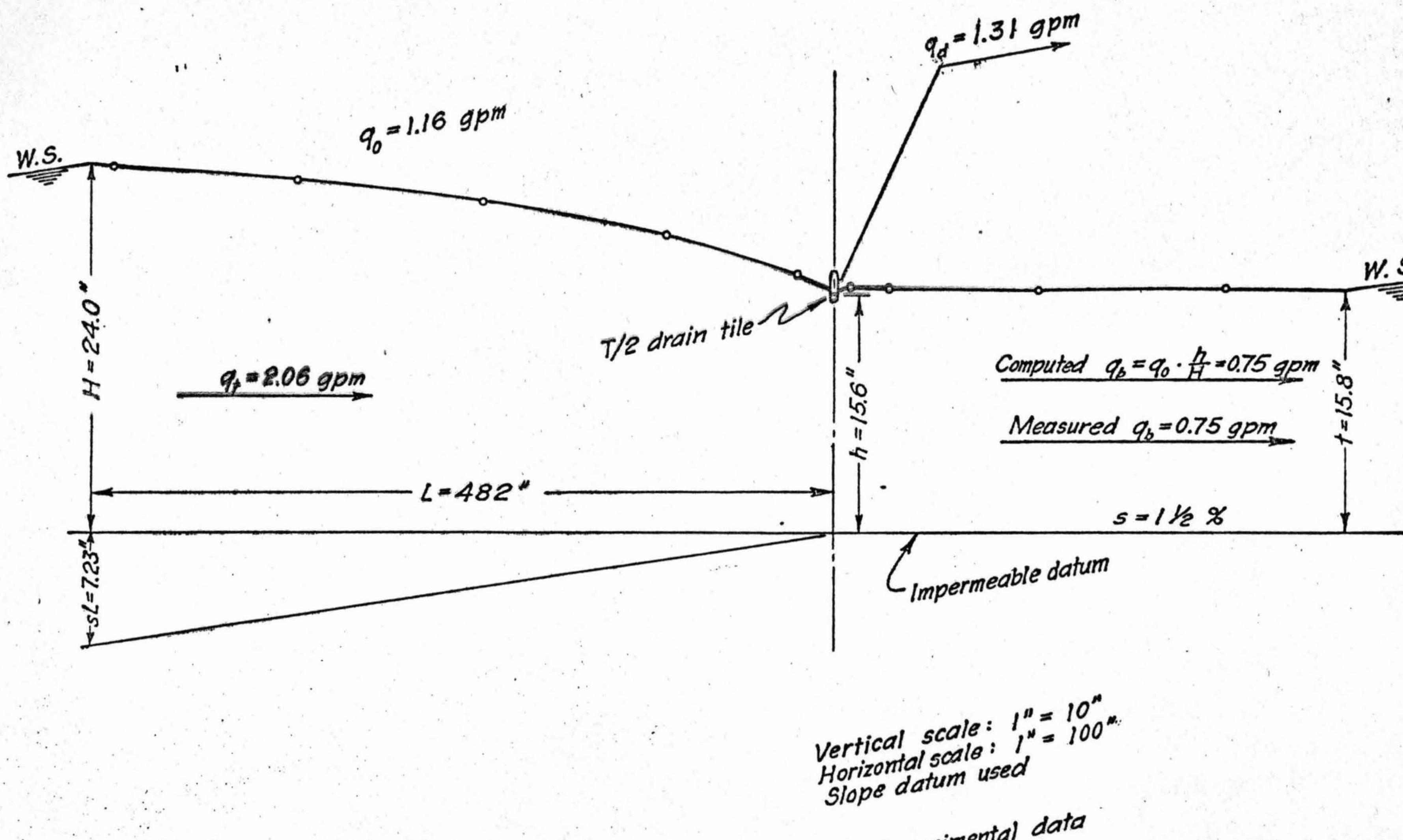

Fig. 10 Example of an interceptor drain drawdown curve.

general steps in the procedure ware as follows:

a. The flume was set at a uniform slope by means of the adjustable screw jacks. A surveyors level was used to establish the slope accurately.

b. With all four drain tiles shut off, and the head-box and tail-box overflows raised above the highest H value desired, water was slowly pumped into the head box end of the flume.

c. After the flume was filled in this manner, q_o was measured and manometer readings were taken for the highest $H = h$ desired.

d. With H held constant, one of a series of drainage conditions was imposed. This was done by either lowering a flexible tube connected to a tile and setting the tail-box water depth so that $q_b = q_o h/H$ or by setting the tail-box water depth at some h. In the latter method all drain flow passed through the tail box, and a situation somewhat similar to an open ditch drain was established.

e. After equilibrium was established for a given drainage system, q_t was measured and the manometer readings were taken.

f. The procedure was repeated for the following boundary conditions on each slope in the order listed below.

Head Box Depth H - inches	Drain Used*	Drain Depth h - inches
40	TB	40
	TT	30
	TB	25
	MT	18
	TB	15
	T/2	16
	BT	6
	TB	0
24	TB	24
	MT	18
	T/2	16
	TB	15
	TB	9
	BT	6
	TB	0

Head Box Depth H - inches	Drain Used*	Drain Depth h - inches
13.3	TB	13.3
	TB	8.3
	BT	6
	TB	0
8.0	TB	8
	TB	0

The above values were selected so that several equal values of h/H occurred for each slope.

g. Slopes used were 0, ½, 1, 1½, 2, 2½ and 3 per cent.

* TB = tail box, TT = top tile, MT = middle tile, BT = bottom tile, and T/2 = center tile.

Chapter V

ANALYSIS OF DATA AND DISCUSSION

The drainage engineer is interested in two distinct aspects of the characteristics of an interceptor drain. The discharge that the drain will intercept, the increase in total discharge through the system, and the discharge that will bypass the drain constitute one aspect of the problem. This portion of the problem will be treated under the heading "Flow Characteristics" in the material which follows. The second aspect of the problem is the determination of the amount the water table will be lowered by the installation of the interceptor drain. This portion of the problem will be treated under the heading "Shape of the Drawdown Curve."

Gravity is the driving force causing flow through an interceptor drain system. This system may be considered as a two-dimensional seepage flow problem. The use of analytical methods involving boundary-value functions to yield a solution of this two-dimensional problem has not proven successful. The electrical analogue has been employed (2) to study certain phases of an interceptor drain problem and similar seepage problems. An electrical analogue subjected to the identical boundary conditions described in the preceeding chapter could be expected to yield results similar to those presented in this paper.

In this study dimensional analysis was used to infer 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 secondary objective in conducting the experiment was to obtain data for comparison with the findings of other investigators.

If all the pretinent variables have been given proper consideration in a dimensional analysis, the resulting dimensionless parameters should be functionally related. This functional relationship can be demonstrated by graphical plots of the resulting parameters when reliable experimental data are available. The correctness of the dimensional analysis can be ascertained by how systematically the data fall on the various plots. If an important variable, or variables, has not been properly considered, the resulting plots may not yield any systematic information. However, if a minor variable, or variables, has been neglected, a slight amount of scatter may be the only evidence. On the other hand, if the analysis is known to possess a high degree of correctness, the reliability of the experimental data can be ascertained from the degree of alignment and from the amount of scatter in the various plots. The value of the dimensional analysis is reflected by the degree to which the data are summarized, condensed, or generalized by plotting them in the implied dimensionless forms.

Flow Characteristics

In the analysis of the flow data, three assumptions were made. The first assumption was that the capillary flow was negligible; therefore, $H = T$ and the elevation y of a point on the drawdown curve above impermeable bed datum was equal to the thickness of the flow at that section. The second assumption was that the interceptor tile drains were completely effective, thus, $h = t$ when $C = 0$. The third and final assumption was that an open ditch interceptor drainage system was simulated by shutting off all the tile drains and using the tail box to intercept all the flow.

Specific values of the dimensionless parameters in Eq. 27 were computed from the experimental flow data. With q_t/q_o as the absicssa, various plots of equal-h/H values were tried on rectangular semi-log and log-log coordinate paper using the reciprocal and/or the converse* of the reciprocal of the parameter $(H + sL)/H$ as the ordinate. Any one of the plots demonstrated the validity of the dimensional analysis and the reliability of the experimental data. A log-log plot using $sL/(H + sL)$ produced the most satisfactory results; however, the spread between the equal-h/H curves was small. These curves all converged to a point since, as the value of sL increases without limit, the values of $sL/(H + sL)$ and q_t/q_o approach unity. By substituting the parameter q_d/q_o for the parameter q_t/q_o the equal-h/H curves do not converge. This substitution is valid since $q_d/q_o = q_t/q_o - q_b/q_o$, and $q_b/q_o = h/H$. Therefore, q_d/q_o differs from q_t/q_o by the constant amount h/H . The value of sL may be increased without limit by increasing L and/or s without limit; however, extreme values of s are not practical. The term sL represents the energy a system possesses because of its slope. When sL is large, the energy added by the installation of a drain becomes relatively insignificant, that is q_t/q_o 1. Therefore, as the value of sL increases without limit, the value of the parameter $q_d/q_o \rightarrow 1 - h/H$.

Figure 11 shows a plot of the dimensionless parameters $sL/(H + sL)$, q_d/q_o , and h/H computed from the experimental data. The value of the parameter $sL/(H + sL)$ in Fig. 11 may range from zero to one as the value of sL either decreases to zero or increases without limit. The value of q_d/q_o when $sL/(H + sL)$ approaches unity has already been discussed. The

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* That is, $sL/(H + sL)$.

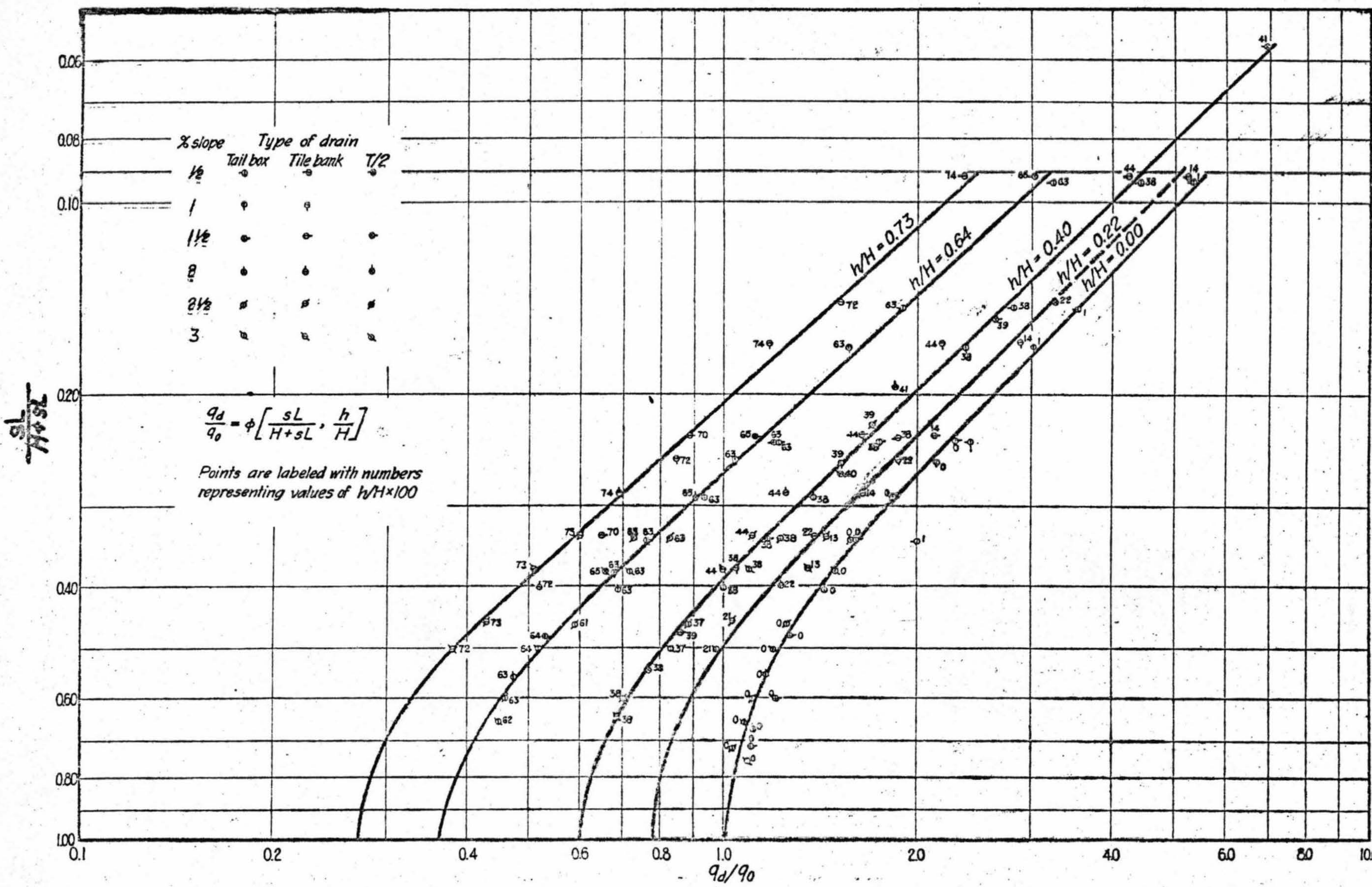

Fig. 11 Discharge of interceptor drains for slopes greater than zero.

value of sL approaches zero as s and/or L approach zero; however, extremely small values of L are not practical. As s approaches zero, q_o approaches zero, and q_d/q_o increases without limit since there will exist a finite quantity of flow q_d in a horizontal system. For values of $sL/(H + sL)$ less than 0.3 in Fig. 11 the h/H curves are straight lines and are nearly parallel. Extension of these h/H curves beyond the scope of the experimental data accordingly seems permissible especially since q_d/q_o is known to increase without limit as $sL/(H + sL)$ decreases to zero.

Figure 11 may best be explained by discussing a hypothetical example. Given an impermeable bed with a slope of two per cent overlain by an aquifer 10 feet thick having a hydraulic conductivity of 10^{-4} fps in which an interceptor drain is installed, what is the flow per foot length in the drain when the drain is installed 4 feet above the impermeable bed and 250 feet from the source of recharge to the system? Solving for the known variables yields, $sL/(H + sL) = .33$ and $h/H = .40$. Entering Fig. 11 with these values yields,

$$q_d/q_o = 1.15, \text{ or } q_d = 1.15\, q_o \,.$$

Remembering that q_o equals the flow before the interceptor drain installation,

$$q_o = HKs = (10)\,(.02)10^{-4} = 2 \times 10^{-5} \text{ cfs per linear ft.},$$

therefore,

$$q_d = 1.15 \times 2 \times 10^{-5} = 2.3 \times 10^{-5} \text{ cfs per linear ft.}$$

When an interceptor drain is placed above a horizontal impermeable bed, i.e., $s = 0$, the data may not be treated as above because both q_o and q_b are zero. A dimensional analysis for this case yielded Eq. 29. Figure 12 shows a plot of the dimensionless parameters q_d/KH , h/H , and L/H computed from the experimental data for the case of zero slope. The value of h/H ranges from zero to one, according to the position of the tile in the saturated medium. When $h/H = 1$, the drain is above the saturated layer, and $q_d/KH = 0$. When $h/H = 0$, the drain receives the maximum rate of flow possible for the particular L/H values in question. Rectangular coordinates were necessary for this analysis since it was desirable to plot zero values on both axes.

A more useful plot for Eq. 29 is shown in Fig. 13. The curves in Fig. 13 were determined by plotting values of L/H and q_d/KH taken from Fig. 12 for constant values of h/H. The log-log coordinate system yielded the most readable curves. The use

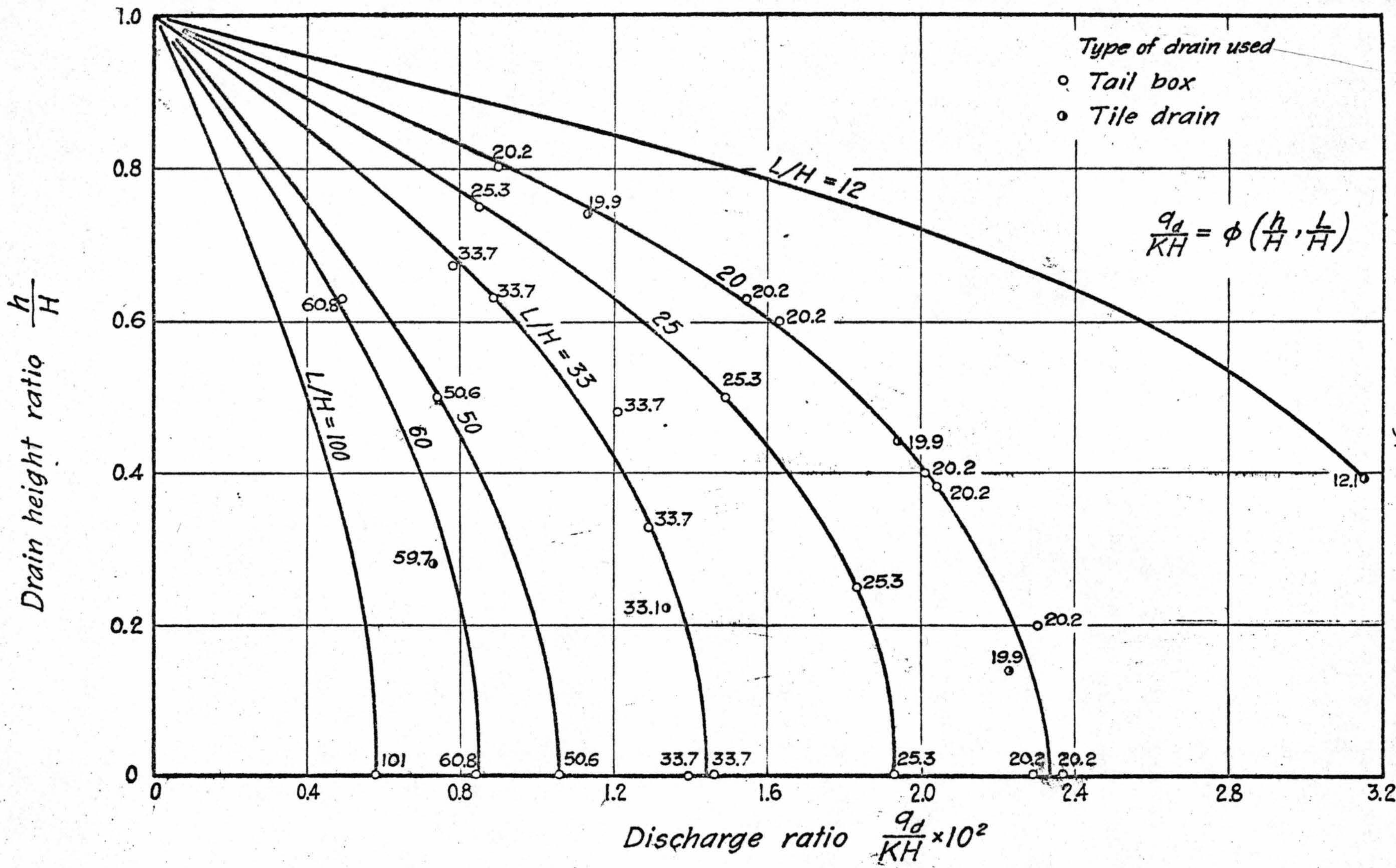

Fig. 12 Discharge as a function of drain height for slope equal to zero.

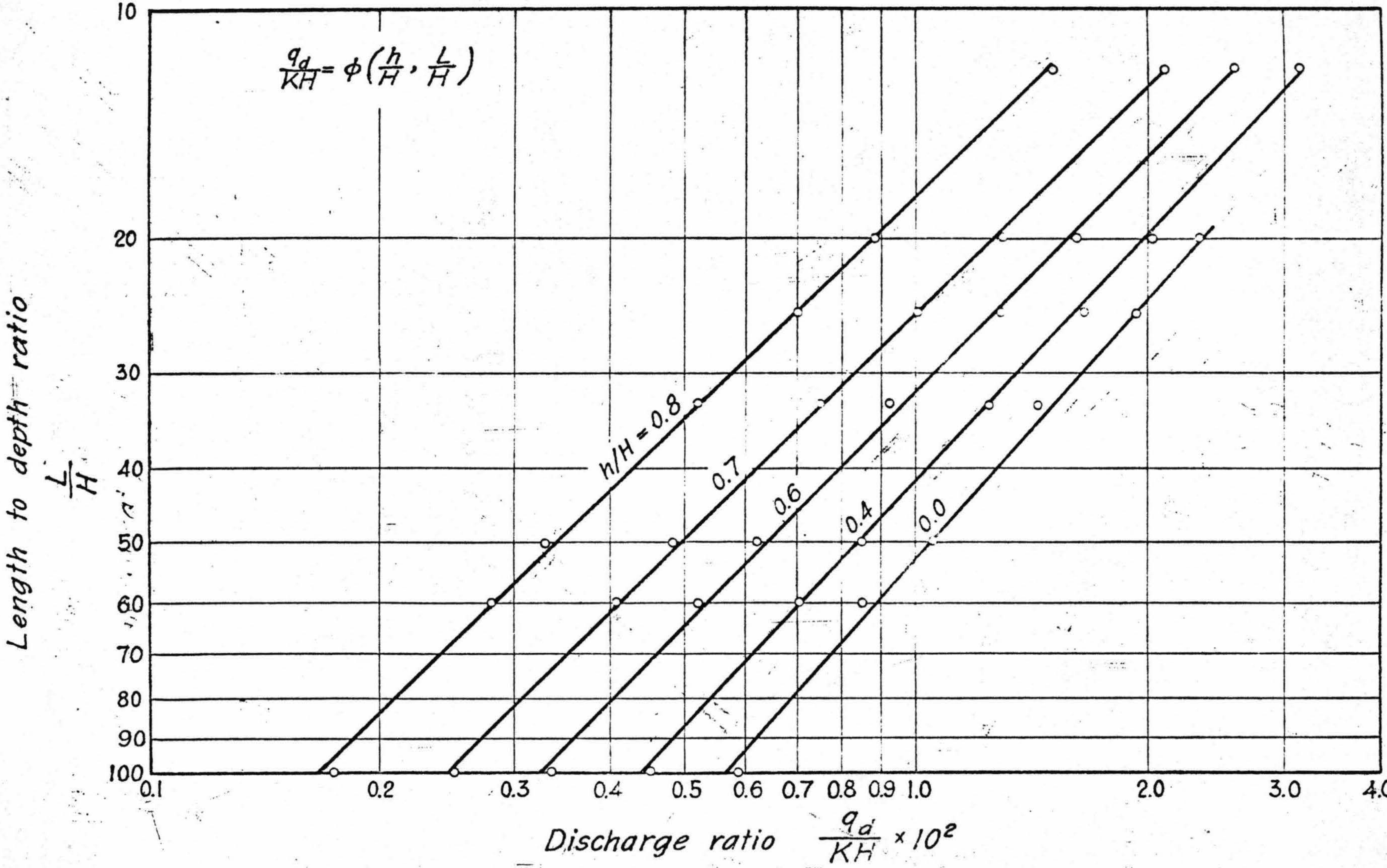

Fig. 13 Discharge as a function of distance to source for slope equal to zero.

of the curves in Fig. 13 is similar to those of Fig. 11 where $s > 0$.

Discussion.--The assumption that the capillary flow was negligible will be discussed first. If the capillary flow is appreciable, $T \neq H$. Using the estimated capillary rise of 1.5 inches, as noted in the previous chapter, for C in Eq. 5, $T = H + 1.0$. This increase in effective depth of flow may be considered in Eq. 27 by changing the parameter $sL/(H + sL)$ to $sL/(H + sL + 1.0)$. In Fig. 14, data for the case when $h/H = 0$ have been plotted in this form, both including and omitting capillary flow. Mean curves drawn through these data are very similar. The scatter is possibly less where the capillary flow was considered. Treatment of the problem using the foregoing method for including capillary effects shows promise, especially when a material having a greater capillary rise is involved. Because of the low capillarity of the material used in this study, further investigations involving sands having greater capillarity are needed in order to study this possibility.

As q_d increases and the drain becomes less efficient, t , the depth downstream from the tile, becomes increasingly greater than h because more energy is expended in entering the drain tile and the height of the seepage face increases. The assumption that $t = h$ when $C = 0$ is not particularly significant unless $h << t$; however, this was never the case in this study. This can be pointed out by the fact that a drain becomes less efficient as q_d increases, and as q_d increases, the efficiency of the drain becomes less significant. Childs (2) pointed out that, even on steep slopes when $C = 0$, $t = h$ may be true for closed drains; $t = h$ is always true for open drains.

The validity of the final assumption that the tail box simulated an open ditch drain is apparent from Fig. 11. The fact that the various h/H curves fall in logical order, regardless of the type of drain used, substantiates this and the previous assumption.

Shape of the Drawdown Curve

For the shape analysis, the second and third assumptions used in the flow analysis, namely that the drains were completely effective and that the tail box acted as an interceptor drain, were made. The first assumption, concerning capillary flow, was not relevant. Mavis and Tsui (10) demonstrated that the shape of the drawdown curve for like boundary conditions was independent of the gradation of the coarse materials that they used for the porous medium. Consequently, a reasonable assumption is that the shape of the drawdown curve is independent of the height of capillary rise when the porous medium is a coarse material. The shape of the upstream drawdown curve should not be greatly affected by

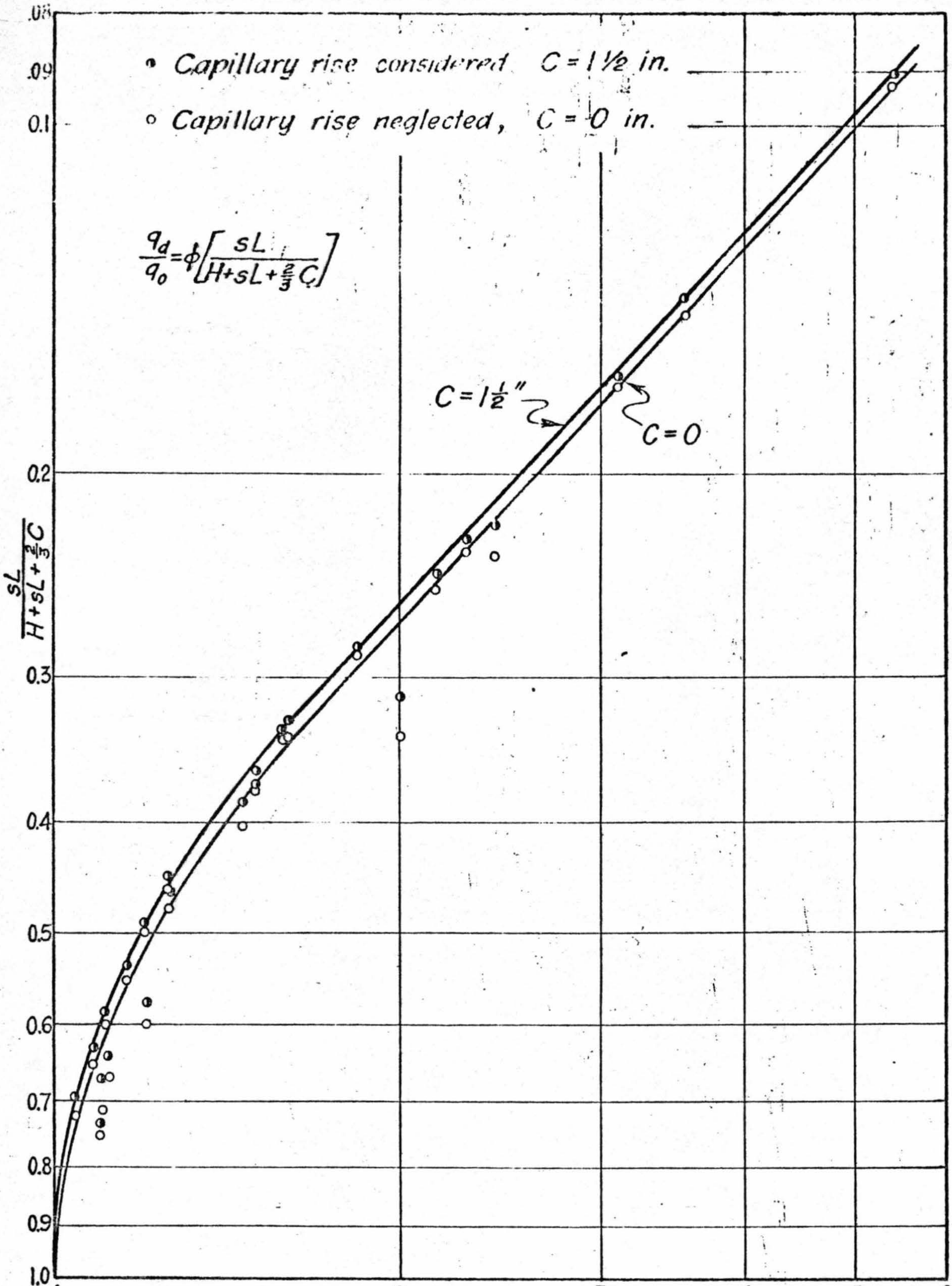

Fig. 14 Effect of capillary rise on discharge of interceptor drains for slopes greater than zero when the bypass flow is zero.

the efficiency of the drain, unless the efficiency is very poor. However, the efficiency of an interceptor drain indirectly affects the lowering of the water table downhill from the drain in an infinitely long system as shown by Eq. 7 presented by Muskat (11). A similar equation for a finite system is

$$D = H(q_o - q_b)/q_o \; . \qquad (35)$$

The bypass flow q_b is directly dependent on the efficiency of the drain; therefore, from Eq. 35 the water-table drawdown D is also dependent upon the efficiency of the drain in a finite system. If the second assumption is true, $D = (H - h)$. The third assumption is assumed to be valid on the basis of its treatment in the section on flow characteristics.

The approach used in adopting a desirable method for the shape analysis was more straight forward than that used for the flow analysis. In Eq. 34, the dimensionless parameters $H/(H+sL)$ and $h/(H + sL)$ are special cases of the parameter $(y + sx)/(H + sL)$ when $x/L = 0$. With $(y + sx)/(H + sL)$ as the ordinate and x/L as the abscissa, the parameters $H/(H + sL)$ and $h/(H + sL)$ define the drawdown curve in question; since, when $x/L = 0$, $y = H$ before, and $y = h$ after the installation of a drain. Zero values on both axes were important; therefore, rectangular coordinate paper was chosen for the plot.

The curves in Fig. 15 are based on a random selection of $h = 0$ data, arranged as indicated by Eq. 34, for various values of s , L , and H when $h = 0$. The value of $(y + sx)/(H+sL)$, besides describing the drawdown curve, is also the portion of the total specific energy, $H + sL$, possessed by the system at the relative horizontal distance x/L. The value of $H/(H + sL)$ depends upon the combination of sL and H used.

For each sL and H combination, the relative specific energy curves for both the undisturbed system and the corresponding drained system were plotted. The relative specific energy curve for an undisturbed system is a straight line between $(y + sx)/(H + sL) = 1$ when $x/L = 1$, and $(y + sx)/(H + sL) = H/(H + sL)$ when $x/L = 0$. The relative specific energy curves for systems with drains are parabolic type curves tangent to the energy lines for the corresponding undrained case when $x/L \doteq 1$ and dropping to a value of $h/(H + sL)$ when $x/L = 0$. Identical $H/(H + sL)$ values are possible for various combinations of H and sL. In Fig. 15, the relative specific energy curves for $H = 40$ and $sL = 20.3$ and for $H = 24$ and $sL = 12.2$ are plotted. Both of these curves have the same value of $H/(H + sL)$ of 0.66. The fact that these curves coincide substantiates the validity of eliminating the parameter $(H + sL)/L$ in Eq. 33 to yield Eq. 34.

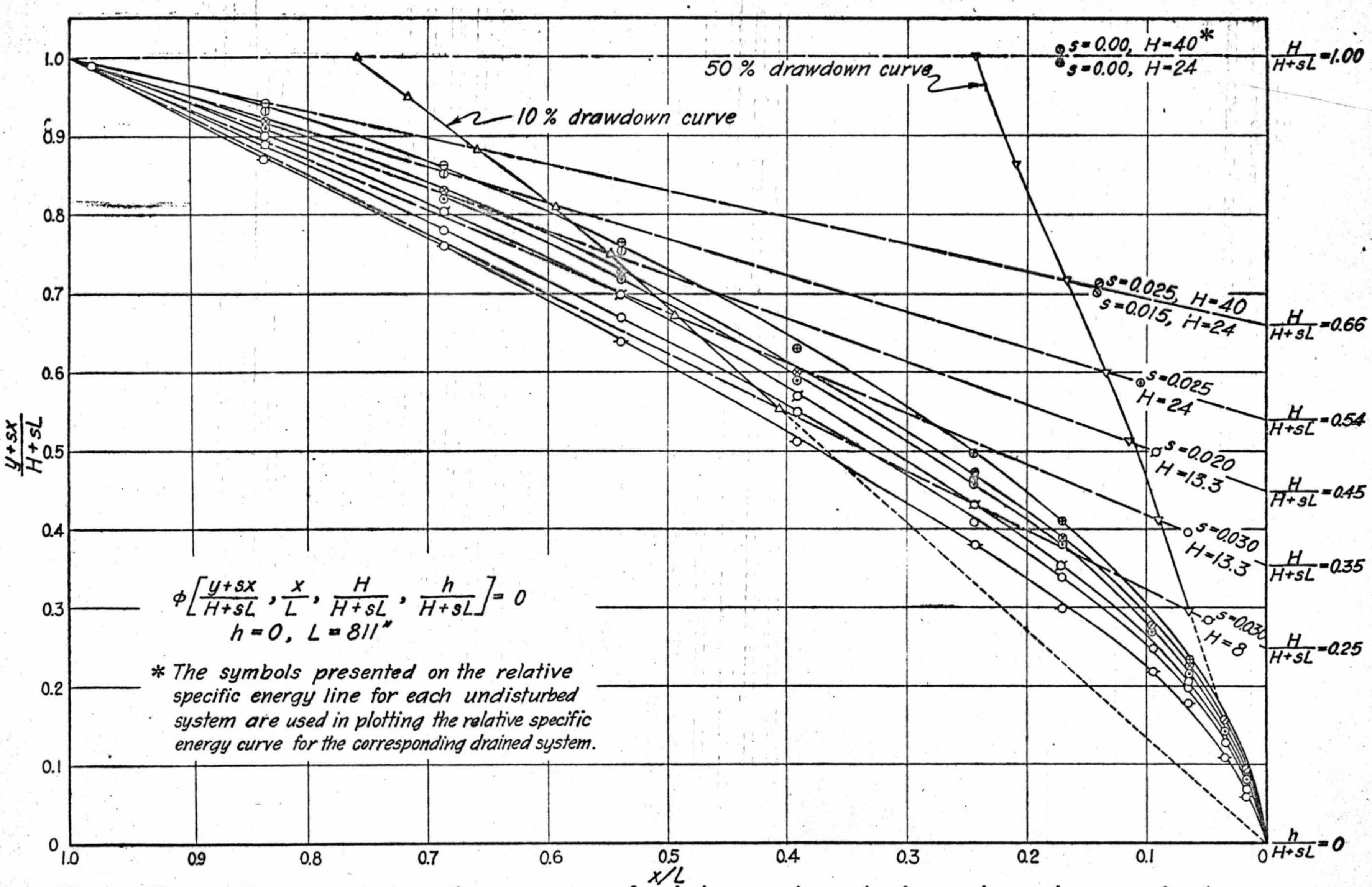

Fig. 15 Observed drawdown curves for interceptor drains when h equals to zero.

In Fig. 15, the 10 per cent drawdown curve is shown. This is a curve connecting the points on the original water surfaces such that drainage causes the water surface to be lowered by $0.1H$. The explanation of the per cent drawdown curves of Fig. 15 is given by use of a hypothetical example. Given an impermeable bed with a slope of 1 per cent overlain by an aquifer 10 feet thick in which an interceptor drain is installed. The drain is installed adjacent to the impermeable layer and 500 feet from the source of recharge to the system. How far from the uphill side of the drain will the water-table drawdown be greater than 1 foot. A drawdown of 1 foot when $H = 10$ represents a 10 per cent drawdown. The proper original water surface or specific energy line is determined since $H/(H + sL) = 0.66$. From Fig. 15 the 10-per cent drawdown curve intersects this energy line at $x/L = 0.66$, thus $x = 0.66(500) = 330$ ft.

The lowest value of $H/(H + sL)$ available from the data was 0.25; however, this ratio approaches zero as L increases without limit. As $H/(H + sL) \rightarrow 0$, the total energy in the system without a drain approaches the total energy in the system with a drain, i.e., $sL \rightarrow [(H - h) + sL]$, and the original specific energy line must nearly coincide with the specific energy curve under drainage except in the vicinity of the drain. In the near vicinity of the drain $x/L \rightarrow 0$ as L increases without limit. On the basis of these inferences, the 10-per cent drawdown curve was extended to the point where $x/L = 0$ and $(y + sx)/(H + sL)=0$. This extension also seemed reasonable in view of the alignment of the points on the curve above the specific energy line when $H/(H + sL) = 0.25$.

Any number of specific drawdown curves could be constructed on Fig. 15. The 50-per cent drawdown curve was constructed to illustrate that the extension of any curve to the origin of both axis is reasonable in view of the alignment of the points on the curve above the energy line where $H/(H + sL) = 0.25$. A figure similar to Fig. 15 could be constructed for other $h/(H + sL)$ values. A useful set of figures might be a set consisting of 10-, 20-, --- and 80-per cent drawdown curves for each of the h/H ratios.

Comparisons of Data.--As a solution for the shape of the uphill drawdown curve of an interceptor drain system, Donnan (3) presented Glover's formula, which is restated here for convenience,

$$x = \frac{H \log_e (H - h)/(H - y) - (y - h)}{s} . \qquad (12)$$

In deriving Glover's formula, as presented by Donnan, the assumption was made, but not stated, that at some point upstream from an interceptor drain the drawdown curve for the drained system intersects the original water table. In explaining the use of Glover's formula, Donnan (3) does not mention whether this distance is finite or infinite. The writer believes that the explanation of Glover's formula as given by Donnan is valid for interceptor drainage systems of effectively infinite length; however, any finite system can be expressed as a portion of a finite system. For this purpose, rewrite Eq. 12 in the form,

$$x = \frac{H' \log_e (H' - h)/(H' - y) - (y - h)}{s} \qquad (36)$$

In Eq. 36, H' is the depth of water at $x = \infty$, or better perhaps, simply a length characteristic describing the drawdown curve. One may use a specific value of y at a known value of x to evaluate H'. The resulting equation then describes the drawdown curve passing through the known point, x, y. In the nomenclature of this thesis, such a point would ordinarily be at $x = L$, $y = H$.

Figure 16 shows a comparison of Glover's formula as stated by Eq. 12 and Eq. 36 with the writer's experimental data. If H is taken as H' for the case $H = 40$, $h = 0$, $L = 811$, $s = .03$, the dashed curve results, which does not agree with the writer's data. Using these values to compute H' as above, Glover's equation agrees closely, however. Similar agreement occurs for the case when $H = 13.3$, $h = 50$, $L = 794$, $s = .03$. In the latter case, however, $H' \cong H$ so that the difference between the two curves was small. A criterion could be established for a value of $H/H - h+sL$ where the difference in H and H' could be considered negligible, and the assumption made that $H = H'$. It is believed by the writer that Glover's formula as interpreted herein can be used to extend the experimental data to complete the series of figures suggested by Fig. 15.

Glover's formula fails when $s = 0$. For this special case Mavis and Tsui (10) suggested that Eqs. 9 and 10 be used to replace the more commonly used formula of Dupuit, Eq. 8. Equation 9 is identical to Eq. 8 when $n = 1$, and the greatest difference between the empirical formula and Dupuit's formula exists when $n = 5/6$, as determined by Eq. 10 using $h = 0$. In Fig. 17, a drawdown curve observed by the writer for $h = 0$ is compared with Dupuit's formula and with the empirical formula developed by Mavis and Tsui. The observed drawdown curve is substantially in agreement with Dupuit's formula but varies considerably from the Mavis-Tsui formula. In Fig. 17, two other drawdown curves observed by the writer are presented. These curves also show close agreement with the parabolic profiles corresponding to Dupuit's formula.

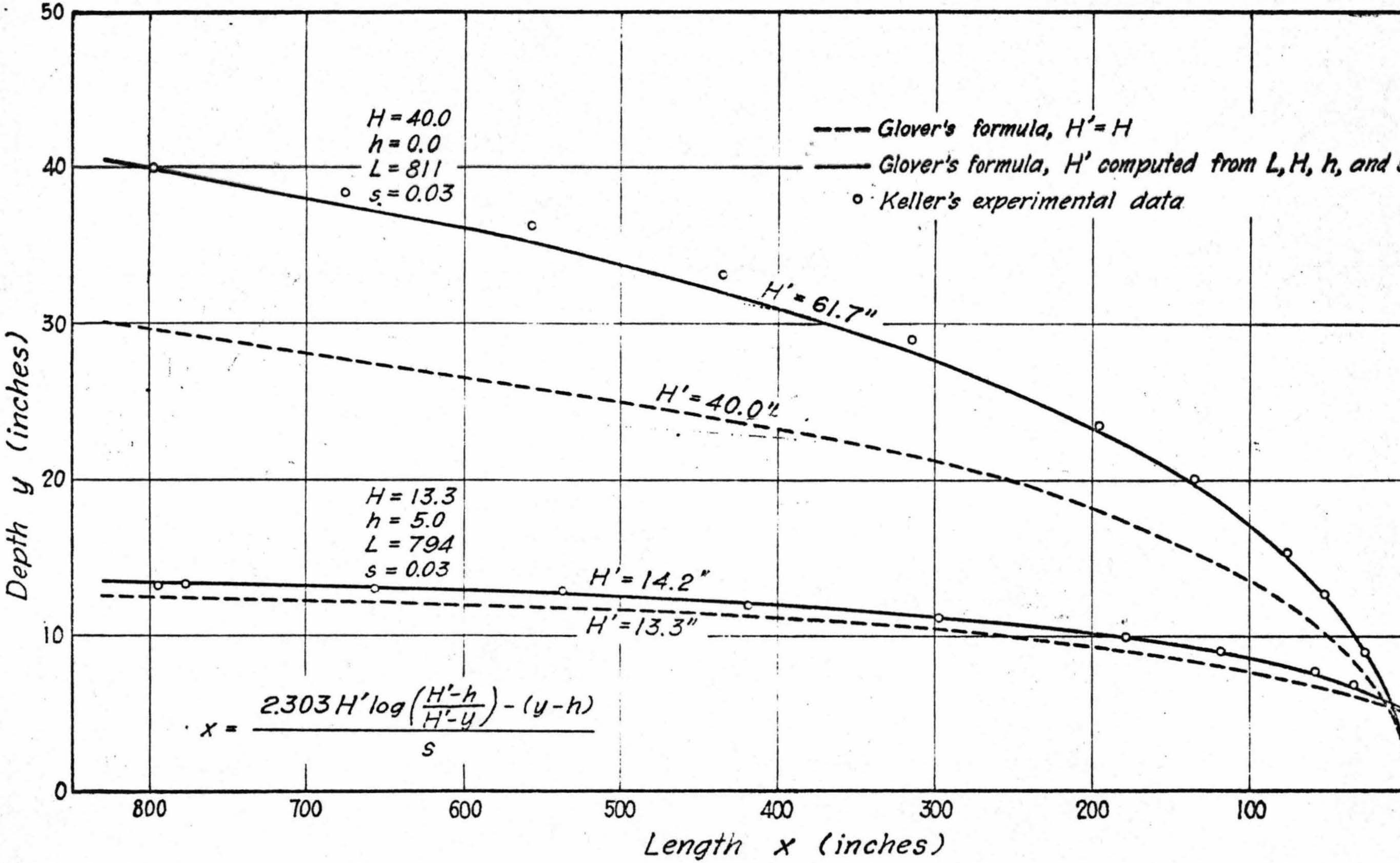

Fig. 16 Comparison of observed drawdown curves with Glover's formula for slopes greater than zero.

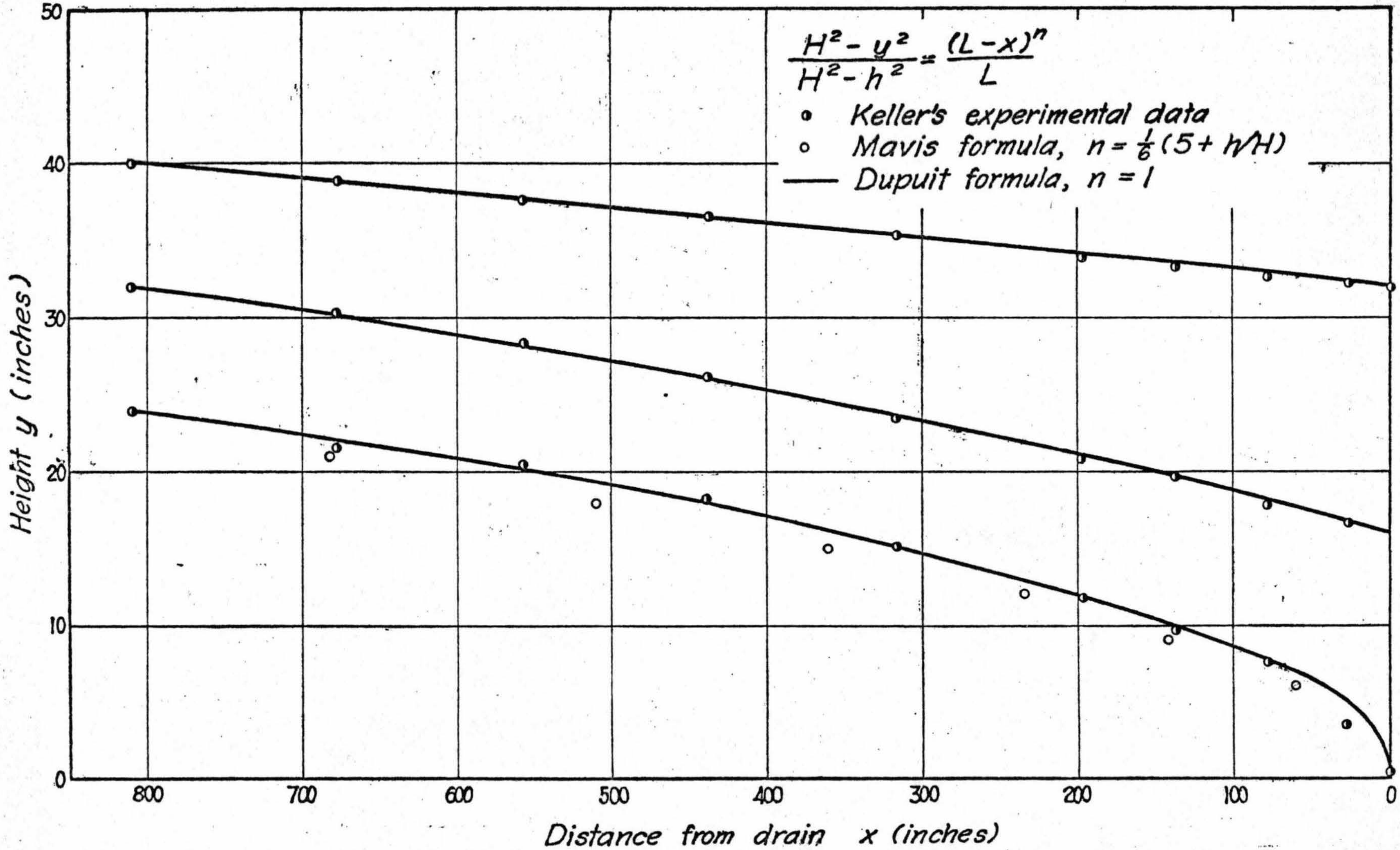

Fig. 17 Comparison of observed and calculated drawdown curves for slope equal to zero.

The author believes that the method used by Mavis and Tsui for placing the sand in the model probably did not yield an ideally homogeneous medium. The writer's difficulties in this regard are reported in Chapter IV. Because of these deviations from the ideal, the empirical formula developed by Mavis and Tsui from their laboratory data does not agree with the writer's data. In view of the simplicity of Dupuit's formula, its fundamental nature, and its agreement with the writer's data, Eq. 8 appears correct for solving those problems concerning the shape of the drawdown curve effected by the installation of an interceptor drain above a horizontal impervious layer.

SUMMARY

The objective of this study was to determine the shape of the watertable drawdown curve and the discharge resulting from an interceptor drain installation. The problem was investigated by using a large laboratory model of an idealized uniformly-sloping homogeneous water-saturated sand. Variables considered included the slope s, the thickness of the water-bearing sand H, the depth of the tile above the impermeable stratum h, and the distance from the drain to the upstream source L. The hydraulic conductivity K was constant since the same sand was used for the entire experiment.

The data obtained from the discharge measurements were analyzed by plotting in terms of dimensionless parameters which were arranged by combining the pertinent variables. In presenting the data, the capillary rise in the porous medium was generally considered negligible since a coarse sand was used in the model study. When the discharge data for the cases of slopes greater than zero were plotted in the form,

$$q_d/q_o = \phi_{20} \left[sL/(H + sL),\ h/H \right] , \qquad (37)$$

on log-log coordinate paper, a family of curves for various values of h/H resulted. In Eq. 37, q_d is the unit discharge under drainage and q_o is equal to KHs, the undrained discharge.

Following the suggestion of Mavis and Tsui and in order to study the effects of capillarity, a correction was made by adding a length equal to two-thirds of the capillary rise C to H in Eq. 37 for the case $h = 0$. This procedure seemed to improve the correlation although capillarity was so small that the significance could not be judged. A similar study using a sand having a higher capillary rise is recommended to evaluate the significance of this correction for capillarity.

When the slope of the impermeable layer is equal to zero, Eq. 37 is meaningless, since $q_o = 0$. For this case, the data were plotted in the form,

$$q_d/KH = \phi_{14}\ (L/H,\ h/H) , \qquad (29)$$

on log-log coordinate paper. A family of equal-h/H curves similar to those described by Eq. 37 resulted.

The dimensionless plots corresponding to Eq. 37 and Eq. 29 can be used to determine the increase in seepage that the instal-

lation of an interceptor drain might cause as well as the flow that the drain can be expected to intercept. In order to use the plots, K, H, and s must be measured or estimated in the field. The source of recharge should be located and its ability to supply flows in excess of q_o, i.e., q_e, evaluated. Also, values must be assigned to L and h according to the designated location of the drain which is to be installed.

The data relative to the shape of the drawdown curves were plotted in terms of dimensionless parameters and also were used for comparison with the findings of other investigators. Data selected at random for all slopes for the case h = 0 were plotted in the form,

$$\phi_{19}\left[(y + sx)/(H + sL),\ x/L,\ H/(H + sL),\ h/(H + sL)\right] = 0, \quad (34)$$

on rectangular coordinate paper using the parameter (y + sx)/(H + sL) as the ordinate and the parameter x/L as the abscissa. The parameters H/(H + sL) and h/(H + sL) delineate any particular drawdown curve. On the same figure were plotted curves connecting all points of 10- and 50-per cent drawdown. These curves can be used to determine the x/L-ratio where the drawdown is 10 or 50 per cent of H - h for any particular case. Construction of similar per cent drawdown curves for other constant h/H ratios, i.e. $h \neq 0$, appears possible, In order to use the per cent drawdown curves H and s must be measured in the field. The source of recharge should be located and its ability to supply q_e evaluated. Also, a value must be assigned to L considering the location of the drain.

When the slope of the impermeable bed was equal to zero, the experimental data for the shape of the drawdown curve compared very well with Dupuit's formula,

$$\frac{H^2 - y^2}{H^2 - h^2} = \frac{L - x}{L} \qquad (8)$$

For impermeable bed slopes greater than zero, the experimental data agreed closely with Glover's formula in the form,

$$x = \frac{2.303\ H' \log (H' - h)/(H' - y) - (y - h)}{s}, \qquad (36)$$

where H' is solved for by substituting the values of x = L and y = H and the known values of s and h in Eq. 36. It was suggested that Glover's and Dupuit's formulas be used to extend the scope of the experimental data by plotting families of per cent drawdown curves for various ratios of h/H.

APPENDIX

Symbol		Definition
a	--	fraction of the incident flow removed by the drain.
C	--	height of capillary rise, (L)*.
D	--	drawdown or the amount an interceptor drain lowers the water table on the downhill side of the drain, (L).
H	--	height of the original water table or headwater depth above the impervious layer measured perpendicular to the slope, (L). (When small slopes are being considered, heights measured perpendicular to the slope or vertically are interchangeable.)
H'	--	a length characteristic used in Glover's formula to describe the drawdown curve, (L) .
h	--	height of the water in the tile drain above the impervious layer measured perpendicular to the slope, or tailwater depth when a tile drain is not used, (L).
i	--	hydraulic gradient, i.e., $\Delta h/L$.
K	--	hydraulic conductivity, a constant depending upon the properties of the porous medium, and the water, (LT^{-1}).
K'	--	permeability of the porous medium, a constant depending upon the properties of the porous medium, (L^2).
L	--	upstream length of the interceptor drainage system, (L).
r	--	radius of the tile drain, (L).
s	--	slope of original water surface or slope of the impervious layer.
T	--	thickness of the original zone of flow, (L).
t	--	thickness of the zone of flow on the downhill side of the drain, (L).

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* Dimensional characteristics are indicated in parentheses.
L = length and T = time.

Symbol	Definition
V --	bulk velocity, or velocity over the gross cross-sectional area of flow, (LT^{-1}).
x --	slope distance from the drain to a point on the drawdown curve, (L). (When small slopes are being considered horizontal and slope distances are interchangeable.)
y --	elevation of the drawdown curve above the impermeable layer measured perpendicular to the slope at a slope distance x from the drain, (L).
z --	width of system, (L).
Δh --	hydraulic head loss by flow through a length of flow path L in the porous medium, (L).
q_b --	discharge per unit width passing below the drain when the drain is placed above the impervious layer, i.e., $q_t - q_d$, (L^2T^{-1}).
q_d --	discharge per unit width intercepted by the drain or flow into the drain, i.e., $q_t - q_o$, (L^2T^{-1}).
q_e --	extra recharge per unit width required to maintain a constant head water depth when a drain is installed, i.e., $q_t - q_o$ or $q_r - q_o$, $(L^2 T^{-1})$.
q_o --	discharge per unit width from a system before the installa-ation of an interceptor drain, where $H = h$, $(L^2 T^{-1})$'.
q_r --	total recharge flow per unit width supplied to a system, i.e., $q_e + q_o$, $(L^2 T^{-1})$.
q_t --	total discharge per unit width from a system with a drain, i.e., $q_d + q_b$ or $q_e + q_o$, (L^2T^{-1}).

BIBLIOGRAPHY

1. Childs, E. C. The water table, equipotentials, and stream lines in drained land: II. Soil Science, 59:313-327, April, 1945.

2. Childs, E. C. The water table, equipotentials, and stream lines in drained land: IV. Drainage of foreign water. Soil Science, 62:183-192, August, 1946.

3. Donnan, W. W. Drainage of agricultural lands using interceptor lines; Provisional. Los Angeles, California, U. S. Soil Conservation Service, Division of Irrigation Research, 1953. 13 p. Mimeographed.

4. Donnan, W. W. Model tests of a tile-spacing formula. Soil Science Society of America. Proceedings, 11:131-136, 1946.

5. Harding, S. W. and J. K. Wood. Model tests of flow into drains. Soil Science Society of America. Proceedings, 6:117-119, 1941.

6. Israelsen, O. W. Irrigation principles and practices; 2nd ed. New York, John Wiley and Sons, 1950, 405 p.

7. Karaki, S. Model study of seepage flow from a canal to a shallow water table. Master's Thesis, 1954. Colorado A & M College. 62 p. Typewritten.

8. Kiefer, F. W., Jr. Reynolds number for flow through porous media. Master's Thesis, 1953. Colorado A & M College, 143 p. Typewritten.

9. Kirkham, D. Pressure and streamline distribution in waterlogged land overlying an impervious layer. Soil Science Society of America. Proceedings, 5:65-68, 1940.

10. Mavis, F. T. and T. P. Tsui. Percolation and capillary movements of water through sand prisms. Iowa University. University of Iowa Studies, Studies in Engineering, Bulletin, 18:1-31, April, 1939.

11. Muskat, M. Flow of homogeneous fluids through porous media. New York, McGraw-Hill Book Company, 1937. 763 p.

12. Robinson, A. R. and C. Rohwer. Measurement of seepage. Fort Collins, Colorado. U. S. Agricultural Research Service, Soil and Water Conservation Research Branch, 1954. 99 p. Mimeographed.