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Seepage Losses from Irrigation Channels

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and
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from
Irrigation Channels[1]

CARL ROHWER, Senior Irrigation Engineer
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INTRODUCTION

Nearly one-hundred million acre-feet of water are diverted annually from streams, reservoirs and ground-water basins to irrigate crops in the arid regions of the West. From one-third to one-half of this amount is lost before it reaches the farmers' fields. Seepage probably accounts for the major portion of this loss on most irrigation projects, and since water is the limiting factor in the agricultural development of this region, it is important that efforts be made to reduce the seepage loss. However, before suitable conservation methods can be developed, a careful study of all the factors that influence seepage losses must be made. Various agencies have investigated this problem in the past, but it was not until 1922 that a comprehensive study of the problem was initiated by the Division of Irrigation, United States Department of Agriculture, in cooperation with the California Agricultural Experiment Station and other agencies. This project consisted of a study of the factors that cause seepage, the development and testing of various methods of measuring seepage, and the measurement of seepage losses from lined and unlined canals, laterals and farm ditches under different flow conditions. Work on this project has been carried on intermittently since that date as funds and personnel were available. The results of these studies are reported in this bulletin.

1. Most of the field work covered by this bulletin was conducted by the authors in California under a cooperative agreement between the Division of Agricultural Engineering, Bureau of Public Roads and the California Agricultural Experiment Station. Dr. Samuel Fortier, Associate Chief of the Division of Agricultural Engineering, was in charge for the Bureau of Public Roads and Professor Frank Adams for the California Agricultural Experiment Station. After the death of Major O. V. P. Stout, the project was assigned to Carl Rohwer, who completed the report under the direction of W. W. McLaughlin, Chief of the Division of Irrigation, Soil Conservation Service. This work was carried on under a cooperative agreement between the Soil Conservation Service and the Colorado Agricultural Experiment Station.

Valuable assistance in the preparation of this report was received from the members of the Division of Irrigation, officials of irrigation projects, and engineers of the Bureau of Reclamation, which is gratefully acknowledged.

2. Deceased.

According to the last Federal census (13) there were 125,-034.8 miles of canals and laterals and 28,508.3 miles of pipe lines used in bringing irrigation water supplies to farms in the 17 Western States in 1939. (See table 1.) Of the canals and laterals, a total of 4,648.8 miles had been lined. No data are available as to the length of small ditches used in distributing water on the farms, but the total is probably far in excess of that for canals and laterals. Although the seepage loss from some canals is relatively small, many canals lose a large proportion of the water carried, and in the aggregate the loss is enormous.

The percentage of the water diverted from surface supplies for irrigation that is lost by seepage, evaporation and leakage in each of the Western States is also set out in table 1. These percentages were computed from the 1940 Federal census data on the amount of water diverted from surface sources and the amount of water delivered to the farmer, per acre of irrigated land. They do not purport to be the true values because they are based on only a part of the area irrigated; furthermore, the area for which the total diversions were measured may not be the same as the area for which the total deliveries to the farmers were measured. The percentages are, however, based on such large acreages that

TABLE 1. *Summary of irrigated areas, irrigation water deliveries and losses, and lengths of channels of various types required to deliver irrigation water supplies in the 17 Western States.*[1]

No.	State	Irrigated area	Total water delivered to farmers	Total loss between diversion and delivery		Length of canals and laterals diverting water from surface supplies			Length of pipe lines
						Total	Earth	Lined	
		Acres	Acre-feet	Per cent	Acre-feet	Miles	Miles	Miles	Miles
1	Arizona	653,263	2,136,334	60	3,200,000	4,178.2	3,986	192.2	344.3
2	California	5,069,568	13,419,802	46	11,430,000	19,799.1	18,217	1,582.1	22,690.2
3	Colorado	3,220,685	7,758,458	17	1,590,000	19,864.0	19,707	157.0	245.1
4	Idaho	2,277,857	9,399,232	31	4,220,000	13,602.1	13,489	113.1	299.9
5	Kansas	99,980	160,505	....	40,000[2]	292.5	292	0.5	24.1
6	Montana	1,711,409	3,369,689	42	2,440,000	15,702.5	15,555	147.5	148.1
7	Nebraska	610,379	1,195,541	24	378,000	3,331.3	3,287	44.3	126.1
8	Nevada	739,863	2,163,236	19	508,000	2,897.2	2,845	52.2	104.7
9	New Mexico	554,039	1,446,418	55	1,566,000	4,647.9	4,567	80.9	36.5
10	North Dakota	21,615	39,309	60	59,000	159.2	158	1.2	3.8
11	Oklahoma	4,160	5,671	....	1,500[2]	42.2	41	1.2	24.4
12	Oregon	1,049,176	3,196,788	21	850,000	8,518.0	8,222	296.0	665.2
13	South Dakota	60,198	58,797	55	48,000	1,049.3	1,038	11.3	17.3
14	Texas	1,045,224	2,436,310	48	2,250,000	5,936.1	4,944	992.1	923.1
15	Utah	1,176,116	2,932,890	20	733,000	9,004.5	8,742	262.5	172.5
16	Washington	615,013	3,252,733	27	1,204,000	4,248.6	3,581	667.6	2,612.7
17	Wyoming	1,486,498	2,889,613	67	4,100,000	11,762.1	11,715	47.1	70.3
	Total	20,395,043	55,861,326	38	34,617,500	125,034.8	120,386	4,648.8	28,508.3

[1] Compiled from 1940 Irrigation Census of the United States.

[2] Data inconsistent, loss estimated.

they are believed to be representative. The data show that the losses range from 17 to 67 percent of the total diversions. These values are probably too high because of the tendency to give full measure to the farmer when making deliveries, but even if the turned-out excess averaged less than 10 percent, the effect on the apparent seepage loss in those states where the losses are small would be considerable.

Complete data on the total diversions by all canals in the Western States are not reported by the Bureau of the Census, but the total deliveries to farmers, based on measured and estimated quantities, are given in the irrigation reports. The data for 1939 are set out in table 1, together with the total losses computed from the relation between the quantity of water diverted per acre and that delivered to the farmer. The total loss derived in this manner, although admittedly only an approximation, is 34,617,500 acre-feet per annum, or 38 percent of all water diverted. Part of the loss is due to evaporation, leakage from faulty structures, and inaccurate measurement of deliveries, however, seepage probably causes a greater loss than all the other influences combined. Most of the water lost seeps downward and finally becomes a part of the ground water, which may reappear in the streams as return flow or may be recovered by pumping from wells; but some of it is lost to the drainage basin by migration to other basins (as occurs in the Platte Valley), and a considerable portion accumulates in water-logged areas from which it is dissipated by evaporation and transpiration. The magnitude of each of these quantities cannot be determined, but the importance of the losses from canals is apparent from the quoted figures.

FACTORS AFFECTING SEEPAGE

Seepage is customarily defined as the emergence of water from saturated soil as well as the disappearance of water into the soil. According to Tolman (12) seepage is the movement of water into or out of the ground. A similar definition is given by Meinzer (7), but for movement he substitutes percolation which Tolman uses only in the restricted sense of movement of water in saturated media. In this report, seepage is used to refer to the movement of water into or out of irrigation channels through the bed material. The amount of seepage may be measured in cubic feet per square foot of water surface or wetted surface, per 24 hours; in cubic feet per second per mile; or in percentage of total flow per mile. Of these terms cubic feet per square foot of wetted surface per 24 hours is believed to be most generally useful. It is the unit adopted for this report.

Seepage is a complex hydrologic phenomenon, and because of the many variables involved no general law for computing the rate of seepage has been developed. When water is flowing in a canal, the water in contact with the bottom and the banks of the channel immediately starts to move into the interstices between the particles making up the lining of the channel. This movement is a combination of capillary flow and percolation. The capillary flow is caused by the capillary attraction of the fine passages between the particles of the bed material, whereas percolation is caused by the action of gravity in forcing the water through the pores of the bed material. The action of gravity is always downward, but capillary attraction operates in all directions and may cause the water to rise many feet above the level of the water in the channel. Capillary movement is extremely slow and for this reason it is ordinarily small in comparison with percolation.

The canal banks and bottom in contact with the water are always saturated, but as the water seeping from the canal leaves this zone this condition may or may not exist. If the water table in the area is in contact with the bottom of the canal or has risen above it, then all the soil in the immediate vicinity of the canal below this level is saturated, and the seepage follows the laws of percolation[3]. If the water table is above the level of the water in the canal, then the direction of flow will be toward the canal and a gain in water will occur. If, however, the water table is below the level of the bottom of the canal, the region immediately below the canal will not be saturated except in special cases, such as when the underlying material is less pervious than the bed of the channel. Where this occurs a pressure zone is built up around the wetted portion of the channel and the flow occurs as percolation. Generally, however, when the water table is below the bottom of the canal, the water seeping from the canal flows downward by gravity as a film of water surrounding the particles of the soil. Under these circumstances, the water lost from the canal flows downward in a zone directly beneath the canal with little, if any, lateral spreading.

The permeability[4] of the material forming the lining of the canal, whether it be the natural soil or an artificial lining, is in general the most important factor in determining the rate of seepage. Permeability is influenced by the size of the pores and the percentage of pore space or porosity of the material. For a

[3] Percolation (laminar flow) is the slow movement of water in interconnected pores of saturated granular materials under hydraulic gradients commonly developed underground. (12)

[4] Permeability as here used is the capacity for transmitting water under pressure. (7)

given size of pores, the permeability increases with the porosity, but materials such as clay which have a high porosity are relatively impermeable. This results from the fact that permeability also varies roughly as the square of the diameter of the pore spaces (12, page 45), and since in clay the pore spaces are very small, the permeability is also small in spite of the high porosity. The presence of gravel in most materials decreases the permeability because it reduces the porosity. Gravel alone, if made up of particles of uniform size, has a high permeability because the interstices between the particles are not filled with finer material and consequently the pore spaces are relatively large. Soils made up of gravel in a matrix of clay are practically impervious to water and are quite stable.

According to Darcy's (1) law, the velocity of flow through water-bearing sands is directly proportional to the head consumed. This law is generally assumed to apply to all saturated water-bearing materials in which the pores are of capillary size and the flow is laminar. Experiments at the hydrologic laboratory of the Geological Survey show that samples of coarse gravel may transmit water at a rate 450,000,000 times that of clayey silt (14, page 11). The porosity of the clayey silt was 58 percent and that of the gravel 38 percent. For a head of 1 foot per foot of this gravel, the discharge was 90,000 gallons per square foot of cross-sectional area per 24 hours. Under similar conditions the discharge through the clayey silt was only 0.0002 gallon per 24 hours. The wide range of possible seepage is apparent from these figures. The actual losses from canals, however, are far less than the maximum because the bed materials of canals are partially sealed by silt and clay carried in the water. Furthermore, although the depth of water in canals may be as great as 5 feet and sometimes even more, field studies show (page 43), that the seepage is only loosely correlated with the depth. This lack of correlation between depth and rate of seepage has been reported also by Lane (12, page 243). In a study of water spreading for storage underground, Mitchelson (9, page 80) observed that the seepage rate decreased materially when the water table reached the level of the water-spreading ground, but the depth to ground water had no effect so long as the water table was below the surface of the ground. Maximum seepage rates occurred during the period that the water table was dropping after it had risen to the ground level. This effect disappears, however, when the ground-water level has dropped several feet. The combination of these factors, together with others the influence of which is not recognized at the present time, makes it difficult to evaluate the influence of depth of water on seepage.

Seepage is also affected by the temperature of the water. According to Poiseuille's (10) law, the velocity of a liquid through a capillary tube varies directly with the specific gravity of the liquid, the head and the square of the diameter of the tube, but inversely with the viscosity of the liquid. Since the viscosity decreases as the temperature increases, seepage should increase as the water grows warmer. For ordinary temperature ranges the coefficient of viscosity changes about 1 percent per degree Fahrenheit (2, page 209). A similar change should be expected in the seepage for each degree change in temperature of the water. Although it is generally accepted that an appreciable change in the seepage should take place, actual seepage measurements involve so many uncertainties that it is rarely possible to differentiate between the effects of the various factors (See page 43).

The theoretical relation between the various factors and the seepage according to Darcy's law (14, page 11), is expressed by the formula $Q = K_d IA$, in which Q is the quantity of water in unit time,

K_d[5] is the coefficient of hydraulic permeability (The subscript d is used here to differentiate the Darcy coefficient from other values of K appearing in this report.)

I is the hydraulic gradient, and

A is the wetted area of the canal bed and banks.

This formula may also be expressed in terms of the head available, as $Q = \frac{K_d hA}{l}$

in which Q, K_d and A have the same significance as before, h is the head producing seepage, and l is the length of the column of material through which seepage is taking place under the head, h.

[5] The coefficient of permeability of a material, as defined by Meinzer (11, page 148), is the rate of flow in gallons a day through a square foot of its cross-section, under a hydraulic gradient of 100 percent, at a temperature of 60° F. Other investigators have defined the coefficient in terms of cubic feet per day. When the permeability is extremely small, the coefficient may be expressed in gallons or cubic feet per year. Israelsen (4) has suggested a different coefficient Kg, which he calls the specific water conductivity and defines as "The volume of water that will flow in unit time through a soil column of unit cross-section area due to the driving force per unit mass corresponding to unit potential gradient." Whichever coefficient is adopted determines the unit in which Q in Darcy's law must be expressed.

All these variables are readily susceptible of direct measurement in seepage experiments except K_d, the permeability, and I, the hydraulic gradient. Several formulas have been developed by means of which K_d can be computed from the temperature or viscosity of the water, the porosity and the mechanical analysis of the sand. None of these formulas has been found entirely satisfactory. Permeability may also be determined by direct measurement of the flow by means of permeameters, by measurement of velocity of flow with dyes or chemicals, or by computation from the data for drawdown and discharge from pumped wells (15). The last-named method has the advantage of giving an average value for the water-bearing material in the region of observation wells.

The problem is complicated still further by the fact that the material comprising the bed of a channel is not uniform and the permeability determined for a sample of the bed taken at one place may or may not apply at any other place. Also, the permeability measurements should be made on undisturbed materials because marked changes in the structure of the material usually result if the sample is broken up and then repacked for testing. For this reason it is desirable to make the test in place if possible.

Difficulties are also encountered in determining the effective head causing the seepage from canals because of the great variation in conditions. The depth in the canal is only one of the factors. According to Darcy's law, the pressure on both ends of the column through which seepage is occurring affects the rate of seepage, and their magnitude must be known before the effective head can be determined. Measurement of the area through which seepage is taking place presents no difficulty. Most canals are of fairly uniform section and a few profiles across a canal will give a reasonably close approximation of the area. Where the canal is irregular, sufficient accuracy can be obtained by taking more profiles.

Because of the difficulty in measuring the parameters, Darcy's formula $Q = K_d IA$ cannot be used in determining the actual seepage from a canal, but it is very useful in disclosing the relationship between the various factors and the seepage. Even though the value of K_d, the coefficient of permeability or of I, the slope, may be unknown, it is readily apparent from the formula that the seepage is directly proportional to each of these factors and any changes in them will affect the seepage in like proportion. This is true also of the area A.

METHODS OF MEASURING SEEPAGE

Seepage measurements on canals may be divided into two classes: Those made on a relatively long section of the canal, and those made on a specific portion of the canal bed or on material from one or more points in the canal in the belief that these measurements would be representative of the whole canal. Inflow and outflow measurements for the section under test and sinkage measurements on a pool formed in the section are of the first type. Measurement of seepage from pits in the canal or in material similar to that in the canal, and measurement of flow through undisturbed material from the canal bed by means of permeameters are of the second type. Measurements of the first type are believed to be much more satisfactory than those of the second type, although it is realized that none of the methods is without defects.

Inflow and Outflow Measurements

When the inflow and outflow method is used, the quantity of water entering the section and the outflow must be accurately determined, together with all leaks, increments and diversions. If the seepage losses are small, evaporation and precipitation must also be taken into consideration. Changes in bank and channel storage caused by rising or falling stages of the canal have an important effect on the quantity leaving the section being tested, and must not be neglected if accurate results are to be obtained. Because of the unavoidable inaccuracies in the methods of measuring flowing water, it is important that the section of the canal chosen for testing be as long as possible. Otherwise, the errors in measurement may be greater than the seepage loss.

The simplest kind of seepage measurement by the inflow-outflow method consists of single determinations of the inflow and outflow during a time when the stage of the canal is constant. If the seepage loss is large and the flow measurements are carefully made, this method may give results of sufficient accuracy. Usually, however, better results will be obtained if several measurements are taken at each end of the section being tested and the averages used in computing the loss. If the seepage loss is small, even though care is used in making the measurements, the differences in flow may indicate a gain when this method is used.

More accurate results will be obtained if the total inflow and outflow over a period of days is determined To do this, water stage recorders are installed in the canal at the ends of the section being tested. By measuring the flow at each station throughout the range of stage of the canal, discharge curves for the stations are determined. From these data the total inflow and outflow over any period during the test can be readily computed. If the period is chosen so that the stage of the canal at the beginning and end of the interval is the same, then the difference between the total inflow and outflow will be the net loss. If there is a difference in stage, a correction must be made for channel storage. The difference when corrected for evaporation, precipitation, leaks and diversions will give the seepage loss.

There are several methods by which the flow in the canal can be measured, but under most conditions, the current meter method is the most practicable. Where there are weirs at the ends of the section or if there are checks where weirs can be installed, their use will make it possible to get more accurate discharge measurements. In most instances, however, there is not sufficient fall in the canal to justify weirs and for that reason current meters are generally used. The gaging stations at which the current meter measurements are to be made should be chosen where the cross-section of the channel is well defined and preferably in a lined section so that the dimensions can be accurately measured The station should be free of obstructions which cause disturbances in the water and the velocity should preferably be between 2 and 6 feet per second and as nearly the same at the two stations as is possible.

Any standard type meter in good condition and accurately calibrated may be used to make the measurements. The integration method is best adapted for measuring small canals and laterals and the two-and-eight-tenths method is recommended for the larger canals. The multiple point method may also be used in measuring large canals but errors may be introduced owing to change in stage of the canal because of the additional time required to make the measurement. The six-tenths method is recommended for shallow canals, but it is doubtful whether this method is accurate enough for seepage loss determinations unless the losses are very large. Whichever method is being used, it is essential that a bridge be provided for the work. Wading measurements are too much subject to error for this type of work. The same meter should preferably be used for both inflow and outflow measurements, as this procedure tends to eliminate errors in calibration of the meters. Duplicate measurements by two different types of meters are desirable, because averaging the results helps to eliminate the effect of peculiarities of either meter.

In seepage determinations with a current meter, the accuracy of the result is governed by the nature of the errors in the measurements. If Q_1 and Q_2 are respectively the true inflow and outflow, and n_1 and n_2 the corresponding errors in measurement expressed as ratios (which may be either positive or negative), then the true loss is $Q_1—Q_2$ and the measured loss is

$$Q_1 (1 \pm n_1) — Q_2 (1 \pm n_2).$$

The difference between the true and measured loss is

$$Q_1—Q_2—Q_1 (1 \pm n_1) + Q_2 (1 \pm n_2) = \mp n_1 Q_1 \pm n_2 Q_2.$$

Hence, if the error in both measurements is the same, i.e., if $n_1Q_1 = n_2Q_2$, there is no error in the result. If the percentage of error is the same in both measurements then

$$n_1 = n_2 = n \text{ and } \mp n_1 Q_1 \pm n_2 Q_2 = \mp n (Q_1—Q_2),$$

or the percentage of error in the result is the same as the percentage of error in the measurements; and since the percentage of error in the measurements is small it will be small in the result. If the values of n differ both in magnitude and sign, then

$$\mp n_1 Q_1 \pm n_2 Q_2 \text{ becomes } \mp n_1 Q_1 \mp n_2 Q_2.$$

The error in the result is the sum of the actual errors in the measurements which may easily exceed the loss and, depending on the sign, may either materially increase the measured loss or change it to a gain.

From the foregoing analysis it is evident that a special attempt must be made to have the errors the same in sign and equal in magnitude. It is for this reason that both the inflow and outflow measurements should be made by the same observer; that similar gaging stations be chosen for the inflow and outflow measurements; that sections with large diversions be avoided; and that the same meter and same method be used wherever possible. Under the most favorable conditions, an experienced hydrographer should be able to make gagings in which the error in the discharge is as small as 1 percent. Generally, however, not all conditions are favorable, and when this occurs, errors of 2 percent or larger can reasonably be expected.

Leakage through structures and diversions from the canal cannot be determined with a current meter with sufficient accuracy. Small leaks can usually be measured most satisfactorily by means of 60-degree triangular weirs and diversions can be measured most easily and most accurately by rectangular weirs.

Very small leaks can usually be estimated with sufficient accuracy except where the seepage losses are small, as from lined canals; then even the small leaks have to be measured accurately. Volumetric measurements are most satisfactory for these small flows.

The effect of precipitation and evaporation is relatively small and for that reason precipitation can be determined from the rain caught in any straight-sided can and the evaporation from a large can suspended in the canal. Evaporation measured in this manner is not the true evaporation from the canal surface, but even a relatively large error in the evaporation will have very little effect on the accuracy of the seepage measurement, because evaporation is an insignificant part of the loss except in the tightest lined canals. For most seepage measurements, published data on evaporation (8) are sufficiently accurate.

If there is an appreciable change in the stage of the canal during the time that seepage measurements are being made by single measurements of the inflow and the outflow, the results will be uncertain, but when the seepage is being determined by the total inflow and outflow over a period, the effect of any difference in stage between the beginning and end of the period can be eliminated by converting the volume change into the inflow or outflow in cubic feet per second necessary to produce the change in the elapsed period of time. The volume change will be the mean area of the water surface times the change in stage.

Pits and Pools

A simple method of estimating the seepage that will occur from a new or proposed canal is to note the rate at which water drops in a pit dug in the soil adjacent to the canal. If the pit is the same depth as the canal and is kept filled with water until a stable condition is established in the soil surrounding the pit, the rate of seepage will become practically constant and will roughly approximate the loss from the canal. The rate will not be the same as that from the canal because the lateral movement of the water through the soil is less restricted in the pit than in the canal. (See page 31.) Moreover, this method does not take into account the effect of silt deposits in the canal. The seepage from the canal may approach but will probably never exceed the loss from the pit. Data obtained in this manner are of limited scientific value but they provide a means of making at least reasonable assumptions as to the losses from new or proposed canals.

Probably the most accurate method of measuring the seepage loss from a canal is by observing the rate of drop in a pool formed by damming both ends of a section of it. The only objection to this method is that the seepage rate may be different in still water than in flowing water. However, if there is a difference it is probably small in comparison with the errors which the engineer cannot avoid when making seepage determinations by other methods. When this method is used, the length of section may be short or long depending on conditions, because the length does not affect the accuracy of the results. This is not the case when the loss is measured by the current meter method. In general, the grade of the canal will determine the length of the section chosen. Since the water in the pool will be level, the depth at the upper end will depend on the length of the section and the grade of the canal, and if the section is too long there will be considerable difference in the depth of water from one end to the other as well as in the width of the water surface and in the temperature of the water. These variations make it difficult to determine the true seepage.

A section between two checks in the canal makes a satisfactory pool, but because checks are usually not water tight it is necessary to bank earth against them in order to make sure that there is no leakage. Usually, however, it is necessary to build earth dams at each end of the pool, because checks or gates are not available where needed in the section to be tested.

The rate of drop in the pool can best be measured by means of scales or hook gages attached to stakes driven in the canal or by plumb-bob gages which measure the distance to the water surface from index points established on bridges or other structures. Because wind may cause the water to pile up in one end of the pool, at least two gages should be used, preferably at the ends of the pool. The number of observations each day will depend on the rate of loss from the pool. Ordinarily readings once or twice daily will be sufficient, but if the canal is seeping badly it may be necessary to take hourly observations. The time of each observation should be noted. Because the seepage rate changes with time, the measurements should be made after the canal has been carrying water long enough to stabilize conditions. If the canal has not been used recently it should be kept filled for a day or more before the observations are started. Gains or losses from precipitation, evaporation and leaks should be measured as explained previously.

The principal advantage of this method is that it makes possible the determination of small losses much more accurately than can be done by the inflow-outflow method. By lengthening the time between observations the drop in the water surface can be increased until it is large enough to be measured without appreciable error, and this can be done without introducing new errors. Depth measurements can be made much more accurately than discharge measurements and they are also much easier to make. It should not be inferred that seepage measurements on pools are free of errors. Results frequently show considerable variation, but such large discrepancies as are often found in the losses determined by the inflow-outflow method are seldom, if ever, encountered.

Permeameters

Various types of permeameters have been developed for measuring the permeability of soils in the laboratory, but most of them are not suitable for measuring the seepage from canals because the conditions are not the same. Such a device consists of a cylinder filled with the soil to be tested, a supply reservoir which is connected to one end of the cylinder so that the pressure developed causes the water to percolate through the soil, and a receptacle for catching the water that passes through the soil. Gages are provided for measuring the water pressure on each end of the sample. The coefficient of permeability is computed from the length and cross-section of the sample, the difference in head, the quantity of water passing through, the temperature, and the time. Since the permeability is affected by the arrangement of the soil particles, it is important that an undisturbed sample of the soil be used in making the tests. This is especially important when testing the material from canal beds because the water carried usually deposits a layer of impervious silt or clay in the canal. Unless this layer remains intact in the sample being tested, the observations on permeability will be erroneous. To determine the seepage from the canal, the pressure gradient in the canal must be known as well as the permeability coefficient which is based on unit head and unit length of sample. Because the pressure varies from point to point in the canal bed, measurement of the pressure gradient is seldom attempted. The computation of seepage is based on the measured coefficient of permeability and an assumption as to the pressure gradient. Seepage determined by this method is obviously of uncertain value.

Another method of determining the seepage with a permeameter is to install the permeameter (figure 1) in the bed of the canal and then measure the seepage from the permeameter while the water is in the canal. The drop in the water level in the permeameter and the time and the head with reference to the water level in the canal are observed and plotted. From the curve drawn through the plotted points, the constants of the general permeameter equations (page 21) can be determined. The solution of these equations for the conditions when the water level in the permeameter is the same as the water level in the canal gives the rate of seepage through the bed of the canal. Although preliminary experiments with this permeameter yielded inconsistent results, it is believed that this device has merit.

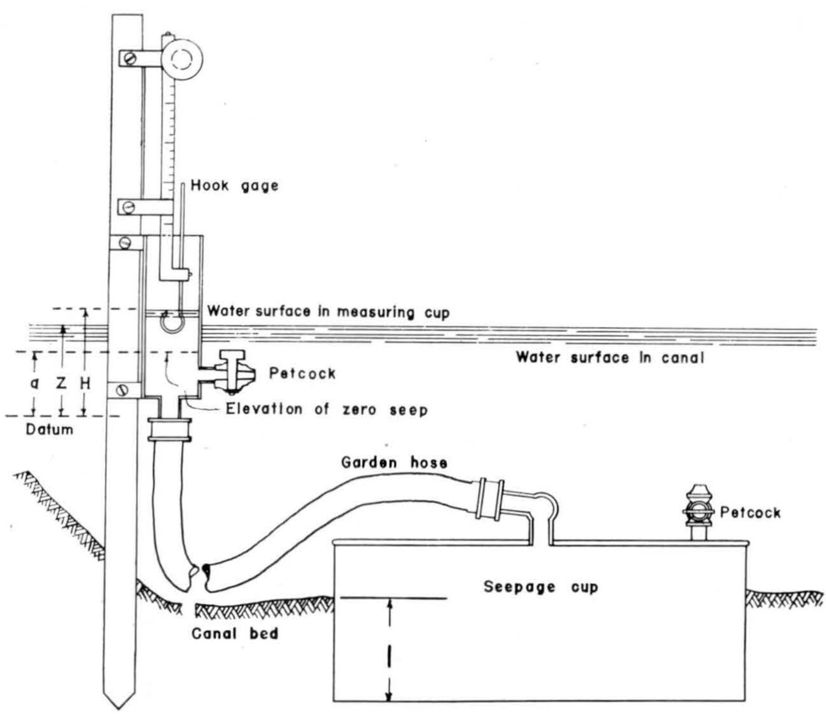

Figure 1.—Section of seepage cup permeameter showing how it is installed in a canal. An improvement in design would make the top of the seepage cup conical.

A recently developed device of this type (5) measures directly the seepage that is occurring through the canal bed under the conditions existing at the time of test. This device consists of a cylinder open at the bottom, which is pressed into the canal bed, and a flexible rubber reservoir, which is attached to the top of the cylinder by means of a hose. (See figure 2.) The rubber reservoir is kept submerged in the canal so that the pressure causing seepage through the soil in the cylinder is the same as that on the canal bed. The loss is determined by raising the rubber reservoir above the water surface and then weighing it. From the elapsed time and the area of the cylinder the seepage can readily be determined. This device seems to have considerable merit and it is hoped that sufficient comparative data will be available soon so that it will be possible to tell whether the results obtained are sufficiently reliable for general use in measuring seepage losses.

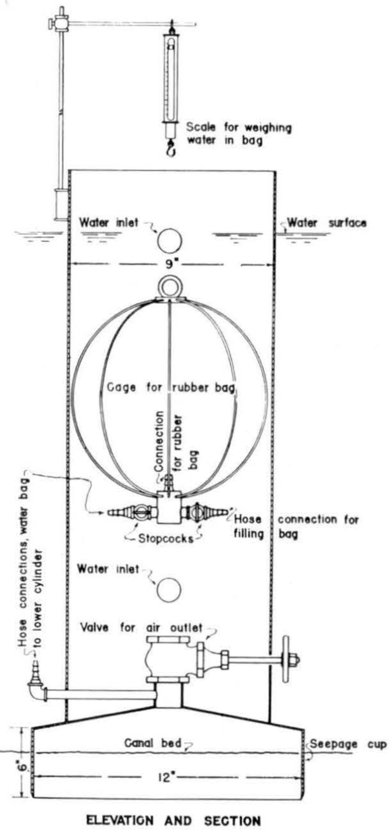

Figure 2.—Permeameter developed by Salinity Laboratory at Riverside, California.

The variable head permeameter is a device of this type adapted to the measurement of the permeability of very tight soils. It consists of a cylinder with a conical top to which is attached a vertical glass tube of small diameter. The cylinder is pressed into the soil to a known depth and then the whole apparatus is filled with water. As the water seeps through the disk of soil in the cylinder the water in the glass tube drops. Since the cylinder is usually made with an area 100 or more times that of the glass tube, a small amount of seepage registers as a large drop in the glass tube. The permeability K, can be computed from the initial and final reading of the head in the glass tube (h_1—h_2), the time interval (T), the thickness of the soil disk in the cylinder l, and the ratio of the area of the glass tube to that of the cylinder $\frac{a}{A}$ (3). The formula is

$$K = 2.3 \frac{al}{AT} \log_{10} \frac{h_1}{h_2}$$

This variable head permeameter when tested by the Utah Agricultural Experiment Station (5) was found to be effective in measuring the permeability of very tight soils. The principal difficulty encountered was the tendency of the cylinder to rise because of the pressure exerted on the inside of the cylinder by the column of water in the glass tube. To overcome this tendency a load was placed on top of the cylinder.

The variable head permeameter measures the permeability but not the seepage rate. In order to determine the seepage it is necessary to know the hydraulic gradient of the flow through the canal bed. This type of permeameter is suitable for comparing the permeability of different soils for use in canal lining. The permeameters previously described are better adapted to measuring the actual seepage from canals. However, until more data are available indicating the accuracy of these devices, the most certain method of obtaining reliable seepage records will be to build a pool in the section of the canal to be tested.

EXPERIMENTAL DATA

The experimental data for this report were obtained from tests made in California and Colorado on seepage losses from lined and unlined canals and laterals. Observations were also made on the losses from pits and trenches. The use of permeameters in measuring seepage was investigated in attempting to find a simple and accurate method of determining the amount of the seepage loss and where it was occurring. In making the seepage measurements on canals and laterals, the loss was determined either by noting the drop of the water surface in a pool in the channel or by measuring the inflow and outflow from a section of the channel with a current meter. Where it was deemed advisable, borings were made along lines perpendicular to the channels to develop information for preparing ground-water profiles. The results of these investigations are presented in the following pages.

Seepage-Cup Permeameter Experiments

Seepage from long sections of canal where the losses are high can be determined with reasonable accuracy by means of current meter measurements, and the seepage from short sections of canal can be determined with precision by measurement of the rate of drop in pools, even in tight soils. However, when planning to line a canal it is necessary to know definitely where the seepage is occurring, and for this reason it would be desirable if direct measurement of the rate of seepage could be made at any point of the bed or sides of the channel. Neither of the methods just mentioned is suitable for the purpose. The permeameter referred to on page 16 and shown in figure 1 was developed to measure seepage under these conditions.

As shown in figure 1, this device consists of a cylindrical bell hereafter referred to as the seepage cup, which is forced into bed or side of the canal, and a smaller cylindrical cup, which is attached to a stake driven into the canal bank in such a manner that the cup will be partially submerged. A hose connects the bottom of the small cup with the top of the seepage cup. A hook gage for reading the water level in the small cup and in the canal is fastened to the stake driven into the canal bank. The latter reading is obtained by opening the petcock in the cup and allowing water from the canal to enter. A petcock is soldered to the top of the seepage cup to allow entrapped air to escape. The diameter of the small cup should be 1/5 to 1/10 that of the seepage cup in order to magnify the seepage so that the rate can be more accurately measured. In the equipment shown in figure 1, the ratio of the diameters was 1 to 6.6, the area of the small cup being 0.01293 square foot and the area of the seepage cup 0.5675 square foot. As a result, the rate of drop is magnified approximately 44 times.

To determine the seepage from a canal, the seepage cup should be carefully forced into the bed of the canal at the point chosen so that the material will be disturbed as little as possible. Since a tight seal must be maintained along the walls of the cup, it should be forced straight down into the bed and preferably should be allowed to stand long enough for disturbed material to settle back into place. All air should be driven out of the seepage cup before it is forced into the bed, by inverting it under water. After the seepage cup has been firmly seated, the hose, which has previously been filled with water, should be attached, as shown in figure 1. The petcock should be closed as soon as the hose has been attached and any air that may have accumulated has been allowed to escape. If all the connections are tight, the level in the cup should begin to drop immediately. If it does not drop below the level of the water in the canal, leaks should be suspected. Leakage is most likely to take place under the lip of the seepage cup, and when this occurs careful tamping around the sides will usually seal small leaks. If the leak cannot be stopped, the cup should be reset.

To determine the seepage rate, the water level in the canal is measured and then water is added to the small cup until the depth is about 0.1 foot above the water level in the canal. The water level is read and the rate of drop is carefully measured by observing the time, with a stop watch, required for the level to drop definite distances, usually 0.01 foot or less, until a level is reached beyond which the water will not fall. This is the elevation of zero seep. At this level the forces causing seepage from the cup are balanced by the forces from outside the cup tending to drive water into it. After this level is reached, the petcock on the cup should be opened and the elevation of the water in the canal again determined.

The seepage through the area enclosed by the seepage cup and consequently the rate of drop in the small cup, decreases as the elevation above the water surface in the canal decreases. It is equal to the seepage from the canal when the water in the small cup is at the same level as that in the canal, because the water pressure on the canal bed inside the seepage cup is then the same as that outside.

This rate can be found graphically by computing the average seepage rate between successive depths in the cup and then plotting these values against the mean gage reading during the interval. The curve drawn through these points gives the seepage rate for any gage reading within the limits covered. The value taken from the curve at the gage reading of the level of the water in the canal is the seepage. However, if the level of the canal rose or fell during the period of the test, the point where the level in the cup and in the canal were the same can be found by plotting the gage reading in the cup and the gage readings in the canal, against time. The desired gage height is where the resulting curves intersect.

The seepage rate can also be computed by formulas derived from the theoretical relations between discharge, permeability, and head. According to Darcy's law. $Q = K_d A \frac{h}{l}$ (page 8). In the case of the seepage cup permeameter, $h = (H - a)$ when

H = elevation of water in small cup with reference to hook gage datum and a = elevation of zero seep measured from the same datum (figure 1). The length of the column of material l, through which seepage is occurring, is equal to the depth of penetration of the seepage cup into the bed of the canal. If these values are substituted in Darcy's formula, then

$$Q = K_d A \frac{(H - a)}{l}.$$ [6]

The area of the portion of the bed of the canal cut by the seepage cup is F, which is the same as A in Darcy's formula. If F is substituted for A, then

$$Q = \frac{K_d F}{l}(H - a) \text{ and}$$

if a new coefficient K is substituted for $\frac{K_d F}{l}$, then

$$Q = K(H - a) \tag{1}$$

The drop in the water surface in the small cup in time dt is dH and since the area is A,

$$Q = -A \frac{dH}{dt} \tag{2}$$

Since Q is the same in formulas (1) and (2)

$$-A \frac{dH}{dt} = K(H - a).$$

For conditions when the elevation of the water surface Z in the canal remains constant a, the elevation of zero seep is a constant, and by integrating

$$-A \frac{dH}{dt} = K(H - a)$$

the formula

$$t = 2.303 \frac{A}{K} \left\{ \log_{10}(H_0 - a) - \log_{10}(H - a) \right\}$$

[6]See notation page for list of symbols and definitions (page 25).

is obtained. Then if K_s is substituted for 2.303 A/K

$$K_s = 2.303\ A/K \qquad (3)$$

and

$$t = K_s[\log_{10}(H_0 - a) - \log_{10}(H - a)] \qquad (4)$$

The formula

$$H = \frac{H_0 - a}{10^{t/K_s}} + a \qquad (5)$$

is obtained by solving formula (4) for H.
Differentiating (5) with respect to t gives

$$-\frac{dH}{dt} = \frac{2.303}{K_s}(H - a) = \frac{2.303}{K_s}\ \frac{(H_0 - a)}{10^{t/K_s}} \qquad (6)$$

If t_m equals 1/2 t_n and H_m is the corresponding value of H, then from equation (4) it can be shown that $(H_m - a)$ is a mean proportional between $(H_0 - a)$ and $(H_n - a)$ and therefore

$$a = \frac{H_0H_n - H_m^2}{H_0 + H_n - 2H_m} \qquad (7)$$

Likewise, if the final values t_n and H_n of t and H are substituted in equation (4) and the resulting equation is solved for K_s

$$K_s = \frac{t_n}{\log_{10}(H_0 - a) - \log_{10}(H_n - a)} \qquad (8)$$

The relation obtained by substituting the values of a and K_s computed from (7) and (8) in (4) is the equation of the curve passing through the initial, final, and midpoints of the curve of observations.

By combining equations (2) and (6) and substituting for H its value Z, when the water in the cup is the same elevation as that in the canal, the cubic foot loss from the cup per 24 hours is

$$2.303 \times \frac{0.01293}{K_s} \times 86400\,(Z - a) = \frac{2572.8}{K_s}(Z - a) \qquad (9)$$

where 0.01293 is the area of the small cup in square feet and 86400 is the number of seconds in a day. The loss in cubic feet per square foot per 24 hours through the disk inside the seepage cup is then

$$Q_z = \frac{2572.8}{0.5675} \frac{(Z - a)}{K_s} = 4534 \frac{(Z - a)}{K_s} \tag{10}$$

where 0.5675 is the area of the seepage cup in square feet and Q_z is the seepage in cubic feet per square foot per 24 hours.

To compute the seepage by equation (10) it is necessary to determine a, the elevation of zero seep, experimentally in the field or by substituting observed values of H in equation (7), and after a has been found, K_s can be determined by solving equation (8). Equation (10) was derived on the assumption that the water level in the canal was constant, and for this condition Z is the hook gage reading.

When the water level in the canal is rising or falling while the observations on the seepage cup are being made, the elevation of zero seep also varies. Special equations must be derived to find the seepage under these conditions. Since seepage cup observations should not be made except when the canal stage is reasonably constant, to simplify the derivation of the formula a uniform change of stage is assumed to take place between the initial and the final hook gage reading of the water level. The elevation of zero seep follows the changes in the water surface of the canal and is also assumed to change at a uniform rate which is the same as that assumed for the canal.

On this assumption, if a_0 and Z_0 are respectively the initial elevations of zero seep and the water surface in the canal, and c the rate of change, then at any time t

$$a = a_0 + ct \qquad \text{and}$$
$$Z = Z_0 + ct.$$

The equations necessary to determine the seepage from the experimental data can then be derived in a manner similar to that previously explained.

$$t = K_s[\log_{10}(H_0 - g) - \log_{10}(H - g - ct)] \tag{11}$$

where $g = a_0 - 0.434cK_s$ (12)

$$Q = K(H - a) = A\left\{2.303 \frac{H_0 - g}{K_s 10^{t/K_s}} - c\right\} \tag{13}$$

$$H = \frac{H_0 - g}{10^{t/K_s}} + g + ct, \text{ from equation (11)}. \qquad (14)$$

Differentiating (14) with respect to t gives

$$-\frac{dH}{dt} = 2.303 \left\{ \frac{H_0 - g}{K_s 10^{t/K_s}} \right\} - c = \frac{2.303}{K_s} (H - g - ct) - c \qquad (15)$$

When $t_m = 1/2\ t_n$, $H_m - g - ct_m$ is a mean proportional between $H_0 - g$ and $H_n - g - ct_n$. From which

$$g = \frac{H_0 H_n - H_m^2}{H_0 + H_n - 2H_m} - \frac{2(H_0 - H_m) + ct_m}{H_0 + H_n - 2H_m}(ct_m) \qquad (16)$$

If H_n is substituted for H and t_n for t in equation (11) and the equation is then solved for K_s,

$$K_s = \frac{t_n}{\log_{10}(H_0 - g) - \log_{10}(H_n - g - ct_n)} \qquad (17)$$

By multiplying equation (15) by A and substituting $Z = Z_0 + ct$ for H

$$\text{Cu. ft. per 24 hr. from cup} = \frac{2572.8}{K_s}(Z_0 - g) - 1117c \quad (18)$$

$$Q_z = \frac{4534}{K_s}(Z_0 - g) - 1968c \qquad (19)$$

To solve equation (19) for Q_z the seepage rate, g is obtained from (16), K_s from (17) and c is computed from the initial and final value of Z and the time. When the water level in the canal is constant c equals zero. By substituting zero for c in the equations derived for varying canal levels the corresponding equations for a constant water level are obtained.

The form of these equations is determined by the fundamental seepage theory based on Darcy's law. It is assumed that the observed data will conform so that when the equation is adjusted to fit the initial, final and midpoint in time for the determination of a, the elevation of zero seep, and the initial and final point for the determination of K_s, the special seepage coefficient, the remaining portion of the data also will fit the equation. In general this was found to be true, which indicates that the theory was quite closely in accord with the facts. The deviations that did occur were such as would be expected from the unavoidable errors in the original data caused by the difficulty of eliminating small leaks and in ascertaining the exact time when the water level in the cup reached the point of the hook gage.

(NOTATION)

The symbols used in the derivation of the equations for computing the seepage from seepage cup observations are defined where first used. They are assembled here for convenient reference.

H = elevation of water surface in feet in small cup reckoned from hook gage datum.

H_0 = initial value of H.

a = elevation of zero seep in feet referred to same datum as H.

a_0 = value of a when $H = H_0$.

Z = elevation of water surface in feet in channel referred to same datum as H.

Z_0 = value of Z when $H = H_0$.

t = elapsed time from observation of H_0 in seconds.

t_0, t_n and t_m = initial, final and mean value of t, respectively.

c = rate of change of a or Z. Computed from observations of Z and t and assumed uniform for period being considered.

A = 0.01293 sq. ft. area of horizontal section of small cup.

F = 0.5675 sq. ft. area of horizontal section of seepage cup.

l = depth of penetration of seepage cup into bed of canal in feet.

Q = volume-time rate of seepage through disk of soil under seepage cup.

Q_z = computed rate of seepage from channel, in cu. ft. per sq. ft. per 24 hr.

K = a seepage coefficient $= \frac{F}{l} K_d$.

K_d = Darcy coefficient.

K_s = 2.303 A/K, a coefficient introduced to simplify equations.

$g = a_0 - 0.434c\,K_s$, a symbol introduced to simplify equations.

Computation of seepage from the seepage cup data requires care because the formulas are complicated and some of the quantities involved are small. For this reason the slide rule is not satisfactory for the work. Examples of the computations by the formulas and by the graphical method are included here to show the procedure and to compare the results.

The data obtained from one series of observations on the East Branch of the Alta Canal at Dinuba, California (pages 77 and 78), are given in table 2. Pertinent data derived from the obser-

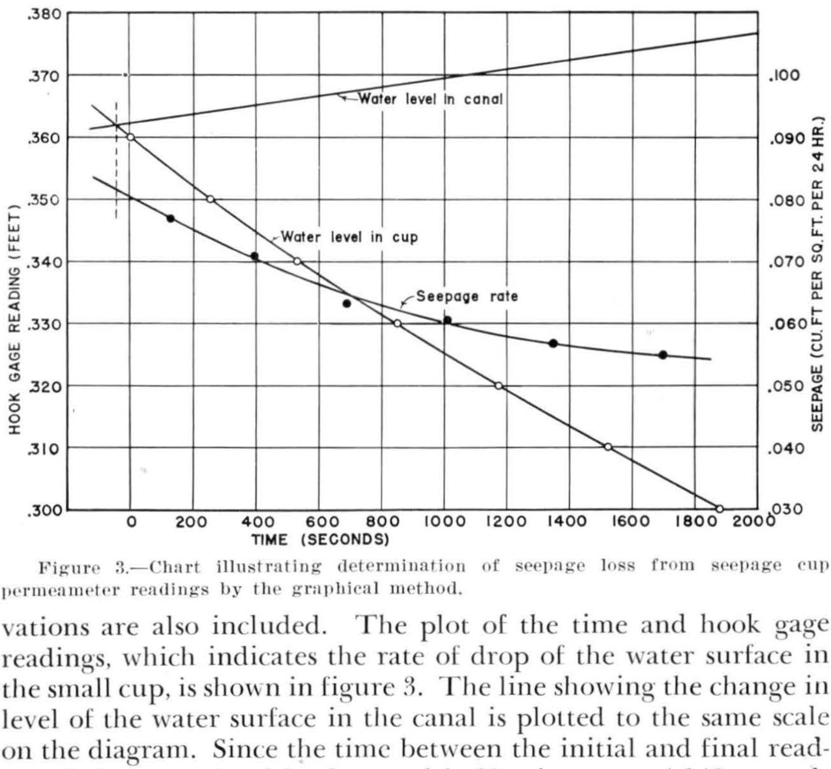

Figure 3.—Chart illustrating determination of seepage loss from seepage cup permeameter readings by the graphical method.

vations are also included. The plot of the time and hook gage readings, which indicates the rate of drop of the water surface in the small cup, is shown in figure 3. The line showing the change in level of the water surface in the canal is plotted to the same scale on the diagram. Since the time between the initial and final reading of the water level in the canal is 69 minutes or 4,140 seconds (table 2), and the rise in the water surface is 0.030 foot, the value of c in the formula $Z = Z_0 + ct$ is $\frac{0.030}{4{,}140} = 0.00000725$. The intersection of the two lines is the point where the water levels in the cup and in the canal are the same. In this case the gage height at the intersection is 0.362 to the nearest 0.001 foot. Since the total time the drop in the cup was observed is 1,879 seconds (table 2), the gage height H_m at the midpoint of time, which must be determined before the formulas can be solved, is 0.327 foot as shown by the time-hook gage curve.

The data required for solving the equations are then as follows:

$c = .00000725.$
$Z_0 = 0.363$ ft.
$H_0 = 0.360$ ft.
$H_n = 0.300$ ft.
$t_0 = 0.$
$t_n = 1{,}879$ sec.
$t_m = \frac{1}{2}t_n = 940$ sec.
$H_m = 0.327$ ft.

TABLE 2. *Observed data and pertinent derived values from seepage cup experiment on East Branch of Alta Canal, Dinuba, California.*

Series	Elevation of water surface in Cup Observed	Cup Computed	Canal Observed	Time interval	Elapsed time	Watch time	Average seepage per sq. ft. per 24 hr.
	Foot	Foot	Foot	Seconds	Seconds		Cu. ft.
			0.362			4:47 p.m.	
1	0.360	0.3600				4:50 p.m.	
2	.350	.3505		256	256		0.0770
3	.340	.3402		279	535		.0710
4	.330	.3298		312	847		.0631
5	.320	.3195		326	1173		.0605
6	.310	.3092		348	1521		.0567
7	.300	.2995		358	1879		.0550
			.392			5:56 p.m.	

In this problem, the level of the water in the canal is rising and therefore equation (16)

$$g = \frac{H_0H_n - H_m^2}{H_0 + H_n - 2H_m} - \frac{2(H_0 - H_m) + ct_m}{H_0 + H_n - 2H_m}(ct_m)$$

will have to be used to find the elevation of zero seep. When the values given above are substituted in the equation

$$g = \frac{(.360)\ (.300) - .327^2}{.360 + .300 - 2\,(.327)} - \frac{2(.360 - .327) + .00000725(940)}{.360 + .300 - 2\,(.327)}$$

$$\times .00000725\,(940) = .178 - .083 = 0.095.$$

After g has been determined, K_s can be obtained by substituting the known values in equation (17),

$$K_s = \frac{t_n}{\log_{10}(H_0 - g) - \log_{10}(H_n - g - ct_n)} =$$

$$\frac{1879}{\log_{10}\dfrac{.360 - .095}{.300 - .095 - .00000725\,(1879)}} = \frac{1879}{\log_{10}1.387} = \frac{1879}{.14208} = 13220$$

Equation (19)

$$Q_z = \frac{4534}{K_s}(Z_0 - g) - 1968c$$

can now be solved for Q_z, the seepage rate, by substituting for K_s Z_0, g and c the values previously determined,

$$Q_z = \frac{4534}{13220}(.363 - .095) - 1968\,(.00000725)$$

$$= .0919 - .0143 = 0.0776 \text{ cu. ft. per sq. ft. per 24 hr.}$$

As previously stated, the derivation of the formulas used in computing the seepage from the seepage cup observations is based on the assumption that the drop of the water surface in the cup follows certain fundamental laws. Then if the observed data fit these formulas it is reasonably assured that the assumption was correct. The equation of the time-hook gage curve derived on this basis is

$$H = \frac{H_0 - g}{10^{t/K_s}} + g + ct$$

and if the theory is correct, the values of H computed by this formula should check the observed values of H. How closely they agree is shown in table 2. In no case is the deviation as much as 0.001 foot.

To find the seepage rate by the graphical method, the average rate of loss in cubic feet per square foot per 24 hours for each interval, is computed from the observed data. These values are given in table 2 and shown plotted in figure 3. The data are plotted to the same time scale on the abscissa as the time-hook gage curve, but the ordinate scale is changed from gage height in feet to seepage in cubic feet per square foot per 24 hours. Since the seepage rate during each interval is the average rate for the interval, it is plotted at the midpoint in time. Since the intersection of the time-hook gage curve with the line showing the change in canal level determines the time and the gage height when the level in the cup is the same as that in the canal, the seepage from the cup under these conditions is the same as that from the canal. It is found by reading the seepage rate from the seepage curve at the point vertically below this intersection. In this case the seepage rate is 0.0815 cubic feet per square foot per 24 hours. The rate computed by the formula is 0.0776 cubic feet per square foot per 24 hours, which, in view of the small rate of seepage, is considered a satisfactory agreement. A closer check would probably have been obtained if the intersections had come on the curves within the limits of the observed data, as would normally occur if the initial level in the cup had been above the water surface in the canal.

Additional experiments were conducted with the seepage cup at various places in California to test the effectiveness of the device in measuring seepage. The results of these experiments, computed by the formulas on pages 21 to 25, are summarized in table 3.

Two series of experiments were conducted at Livingston, California, on an old lateral of the Merced Irrigation District, which is located in very sandy soil classified as Oakley and Fresno sands, undifferentiated. In the first series of observations, the seepage cup was installed in the bottom of the lateral and in the second series it was installed on the side of the lateral below the water's edge. The tests showed that the seepage from the bottom of the lateral was consistently greater than that from the bank, but the rates were much less than were anticipated in view of the sandiness of the soil. With one exception, the tests of each series gave reasonably consistent results. The seepage determined from the first test of the series made in the bottom of the lateral was definitely lower than the others. No reason was apparent for this difference because the conditions remained the same as nearly as could be determined.

Observations on seepage from the East Branch Canal of the Alta Irrigation District near Dinuba, California, were made with the seepage cup installed in the bottom of the canal. The bed of the canal, which was about 25 feet wide, was sandy and was underlain with hardpan at a depth of 4 or 5 feet. The seepage rates de-

TABLE 3. *Summary of results of seepage cup permeameter observations.*

Location	District	Channel	Material	Setting	Computed seepage per sq. ft. per 24 hr.		
					Test No. 1	Test No. 2	Test No. 3
					Cu.ft.	Cu.ft.	Cu.ft.
Livingston	Merced	Lateral	Sand	Bottom	0.1679	0.2944	0.2955
Livingston	Merced	Lateral	Sand	Side	0.0895	0.0714	0.116
Dinuba	Alta	East Branch	Sandy loam	Bottom	0.0482	0.0436	0.0520
Dinuba	Alta	East Branch	Sandy loam	Bottom	0.0776	0.0766	
Red Bluff	Anderson Cottonwood	Green Lateral	Gravelly loam	Bottom	0.186		
Davis	University Farm	Lateral	Sandy loam	Bottom	2.41	4.38	5.64
Davis	University Farm	Lateral	Sandy loam	Bottom	0.805		
Davis	University Farm	Lateral	Sandy loam	Bottom	0.157	[1]	

[1] Observations resulted in data which yield no solution by formulas.

termined by substituting the observed data in the formulas were very small (table 3), which confirms the results of the current meter measurements previously made on this canal when it was found that the losses were so small that they could not be detected by current meter gagings. (See table 9.) The second series of observations at Dinuba, although taken under identical conditions except that 24 hours had elapsed since the previous observations, showed a definitely higher rate of seepage. A similar increase in the rate with time was observed in the experiments at Livingston.

Current meter measurements on a portion of the Green Lateral of the Anderson-Cottonwood District near Red Bluff, California, disclosed a seepage rate of over 5 cubic feet per square foot per 24 hours. (See table 9.) The lateral was excavated in a gravelly loam and there were visible indications of seepage. Observations with the seepage cup at a point in the upper end of the lateral showed a loss of only 0.186 cubic feet per square foot per 24 hours. This extreme variability is explained by the fact that the seepage cup measurements were made in a section where there were no visible indications of seepage. Furthermore, pools in the main canal in the same gravelly loam were found subject to practically no loss from seepage.

Several series of observations were made with the seepage cup on a lateral at the University Farm, Davis, California. This lateral, which was supplied by a pump, was excavated in Yolo sandy loam. The results of the first series of measurements (table 3) are of the same order of magnitude, but the increase in the rate with time is evident also from these observations. The seepage rate determined by the seepage cup agrees reasonably well with the direct measurements from the trench on the college grounds (table 5). The trench was excavated in a somewhat heavier soil classified as Yolo clay loam; the difference in soil would, however, be overshadowed by the fact that the lateral had been in use for a number of years. When the seepage cup was reset a much lower rate of loss was found, and when the seepage cup was moved a second time, a further decrease occurred.

The last series of observations was duplicated and in this case the water level in the cup would not fall below the water level in the lateral. This led to the conclusion that air was trapped in the hose. Since the elevation of zero seep is usually only a short distance below that of the water in the canal, a small amount of air in the hose would result in entirely erroneous readings. For the same reason it can be assumed that entrapped air caused the apparent drop in the seepage rate shown by the second and third series of observations.

Although the results of the seepage cup experiments were not conclusive, it is believed that this device has merit and if greater care had been exercised in getting the air out of the hose and seepage cup before starting observations, more consistent results would have been obtained. Making the seepage cup with a conical top would help avoid trapping air. Seepage rates determined by seepage cup observations should be checked against seepage measurements by other means where possible in order to find out whether the results are accurate.

IMPERIAL VALLEY TRENCH AND PIT EXPERIMENTS

The observations of some engineers and irrigators have led them to believe that the seepage through the sides of a canal is generally greater than through the bottom. If this be true, a considerable saving in the cost of lining large canals could be achieved because it would not be necessary to line the bottom of the canal which comprises a large portion of the perimeter. No feasible method of measuring the bank and bottom seepage from a canal independently has been perfected, but it was thought that if measurements could be made of the seepage from a series of pits and trenches in the same type of soil and differing only in dimensions, mathematical relations could be developed which would make it possible to segregate the bottom and side seepage.

Since the seepage near the corners of the pits and trenches would be different from that through the sides, this effect was eliminated by making the pits square in plan with the sides and depth equal to the width and depth of the corresponding trenches and then subtracting the pit seepage from the trench seepage. The quantity remaining would represent the seepage from the sides and bottom unaffected by the corners. The trenches were of different lengths and widths, but the depths were the same except in one instance. Consequently, for some of the trenches the bottom area was greatest and for others the side area was greatest.

The trenches and pits were seasoned by keeping them filled with water until the seepage rate became fairly constant. After conditions became stabilized, seepage observations were made by noting the time required for a measured quantity of water to seep out of the trench or pit. As previously mentioned, the seepage from each pit was subtracted from the corresponding trench. The remaining seepage represented the unit rate of seepage through the sides, times the area of the sides, plus the unit rate of seepage through the bottom of the trench, times the area of the bottom. This relation was expressed in an equation in which the seepage rates were the unknowns. Similar independent equations were formed for the trenches of other widths and lengths. Since there are only two unknowns—the unit rates of seepage through the sides and through the bottom—the equations were combined by addition to make the number of equations equal the number of unknowns.

Figure 4.—Trenches in Holtville silty clay loam near El Centro, California.

One series of pits and trenches was excavated in a moderately heavy soil on a tract of land about 4 miles west of El Centro, California. This soil is classified as Holtville silty clay loam and is underlain by a somewhat lighter textured soil. Figure 4 shows the trenches in this tract. Another series of pits and trenches was excavated in sandy soil on a tract about 3 miles southwest of El Centro. This soil is classified as Meloland fine sandy loam. The underlying material in this case is a heavier soil. The same procedure was followed in making the observations on seepage from the different soils.

The total seepage per 24 hours was computed for each trench and pit from the rates during the period of the test and the seepage from each pit was subtracted from the corresponding trench. The remainder, which represented the seepage from the trench without the ends, was used in the seepage equations. After the equations were combined, the bottom and side seepage was computed by solving the equations. These unit rates were multiplied by the areas of the bottoms and sides of the trenches exclusive of the ends. The sum of these products for each trench should equal the observed seepage for this portion of the trench. A summary of the results is given in table 4.

Although the observed and computed rates were in close agreement, in the Holtville silty clay loam the side seepage exceeded the bottom seepage,

TABLE 4. *Summary of results of tests to determine bottom and side seepage from trenches in medium and sandy soils.*

	Holtville silty clay loam			Meloland fine sandy loam		
Dimensions	Trench No. 1	Trench No. 2	Trench No. 3	Trench No. 1	Trench No. 2	Trench No. 3
Length (ft.)	10	10	5	5	5	5
Width (ft.)	0.60	1.667	0.583	0.604	0.604	1.667
Depth of water (ft.)	0.627	0.629	0.645	1.257	0.647	0.637
Conditions						
Observed total seepage (cu.ft. per 24 hr.)	9.93	17.64	5.25	10.90	6.67	10.97
Computed seepage sides (cu.ft. per sq.ft. per 24 hr.)	0.402	0.402	0.402	0.71	0.71	0.71
Computed seepage bottoms (cu.ft. per sq.ft. per 24 hr.)	0.648	0.648	0.648	0.47	0.47	0.47
Computed seepage sides and bottom (cu.ft. per 24 hr.) (ends eliminated)	8.493	13.216	3.926	8.93	5.57	5.60
Observed seepage ends eliminated (cu.ft. per 24 hr.)	8.543	13.215	3.823	8.89	5.66	5.58

whereas in the Meloland fine sandy loam the reverse was true. This difference may have been due to the difference in permeability of the soils or might have resulted from differences in the underlying strata. The material removed from the excavations appeared to be quite uniform, and contrary to expectations, the bottom rate exceeded the side rate where the heavier substratum occurred. However, the ratio of side seepage to bottom seepage obtained by this method of segregation could be changed by using a different combination of trenches and pits in computing the results. This fact is probably due to differences in the unit rates of seepage from the trenches and pits, even though the excavations were in material that appeared to be uniform. For this reason the mathematical method of segregating side and bottom seepage was abandoned in favor of a direct method which is described on pages 33 to 40.

DAVIS TRENCH EXPERIMENTS

In preparation for experiments at Davis, California, to determine the difference between side and bottom seepage, two trenches were dug in the experimental plots on the west side of the Agricultural College campus. The trenches were excavated along the same centerline and the ends were separated by an interval of 5 feet. Each trench was 5 feet long, 2 feet wide and little more than 2 feet deep. The soil in the area is Yolo clay loam. Figure 5 shows the relative locations of these trenches and the tanks from which they were supplied with water. The trenches were divided into compartments by means of metal partitions so as to permit the measurement, separately from each other, of the seepage from the middle 2 feet of length of the bottom, the middle 2 feet of length of the sides and the end sections. Figure 6 shows the details of the metal partitions.

The water surface in all compartments of the trench was maintained at the desired elevation by means of the Mariotte control apparatus shown in figure 7. Variations of from one to two hundredths of a foot, which occurred at times, were corrected as soon as observed by adjusting the control apparatus. Trench No. 1 was not used in making tests, because it was found during preliminary observations that excessive leakage was occurring through holes that developed in the bottom of the trench.

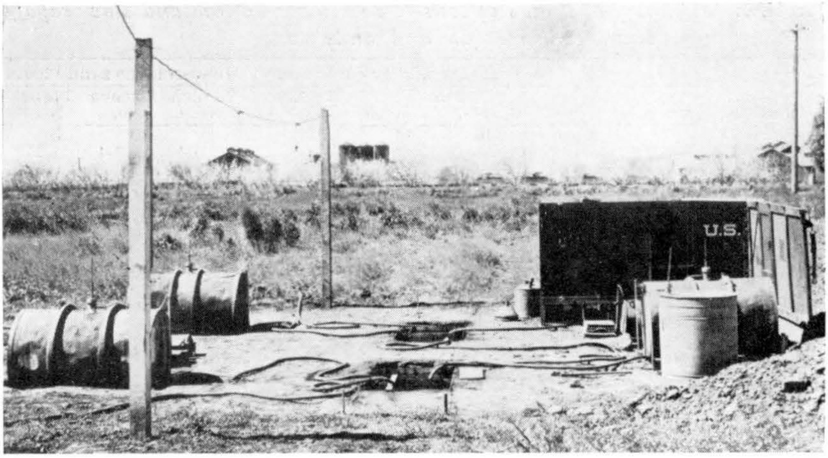

Figure 5.—Trenches and control apparatus for maintaining constant elevation of water surface in compartments of trenches, Davis, California.

Prior to beginning observations on the seepage from trench No. 2, it was seasoned by maintaining the water at the depth proposed for the series of observations until the seepage rates became fairly constant. During the considerable intervals of time between series of observations, the trench was covered so that its bottom and vertical surfaces did not dry out.

EFFECT OF TIME ON SEEPAGE FROM TRENCH

Several series of tests were made when the water in the trench was 2 feet deep to determine whether the seepage was constant or variable. These tests were made in March and April. It was found that the seepage from the bottom compartment decreased as time went on, whereas that from the side compartments increased. The total seepage also increased. The rate of seepage in cubic feet per square foot of wetted area from the sides was consistently greater than that from the bottom. That the seepage from the bottom should decrease with time was to be expected because of the accumulation of silt on the bottom, and as the seepage from the bottom decreased, the seepage from the sides should increase to some extent because of decreased resistance to flow toward the region under the trench. See pages 31 and 32. It was not apparent, however, why the total seepage should increase with time unless it was that the movement of the water through the soil dissolved entrapped air and increased the passageways for the water faster than they were constricted by the swelling of the soil. Some variation in the rates occurred from hour to hour, but these differences were not great enough to obscure the general trend. Observations of seepage at a depth of 1 foot showed also that the side seepage was greater than that from the bottom.

EFFECT OF DEPTH ON SEEPAGE FROM TRENCH

Another series of observations was made to determine the effect of depth of water on the seepage. These tests were made in January of the following

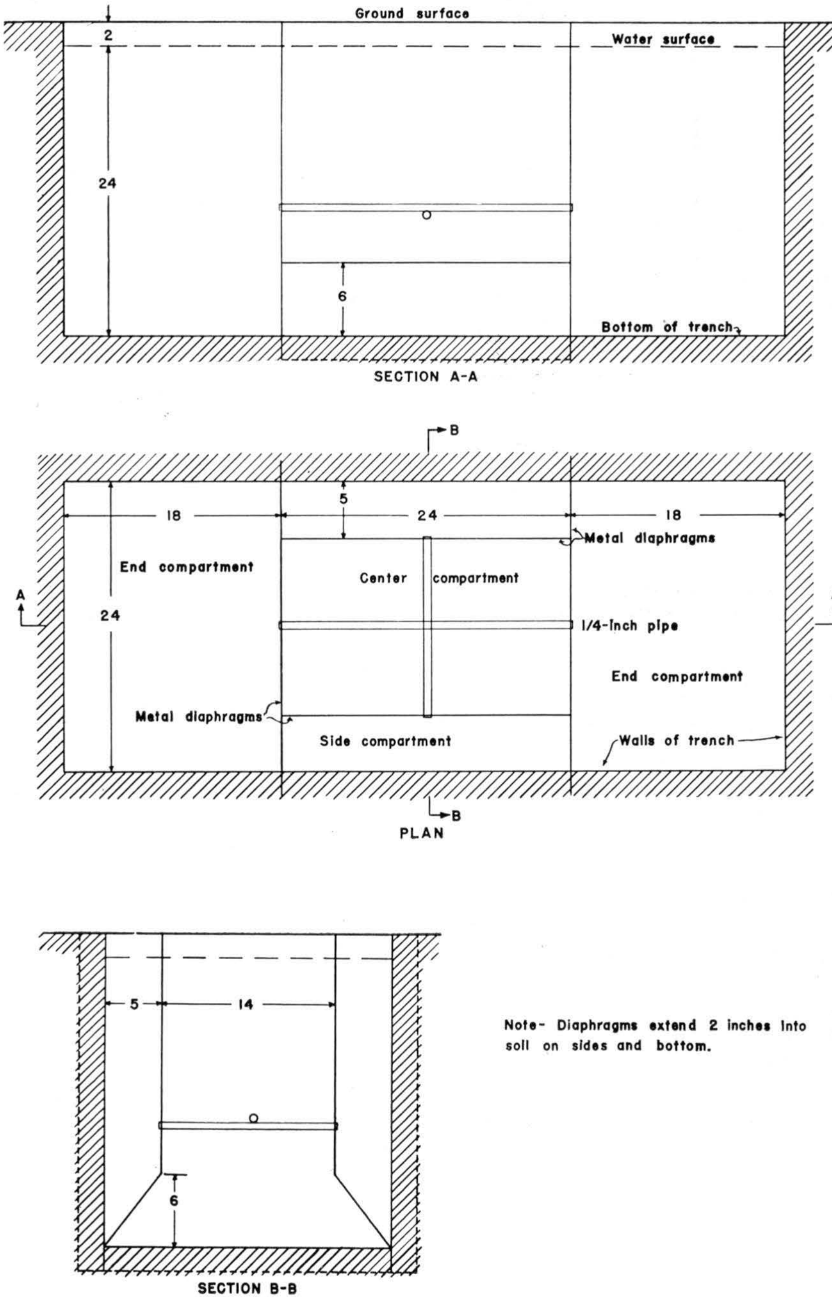

Figure 6.—Plan and sections of Davis trenches showing arrangement of compartments.

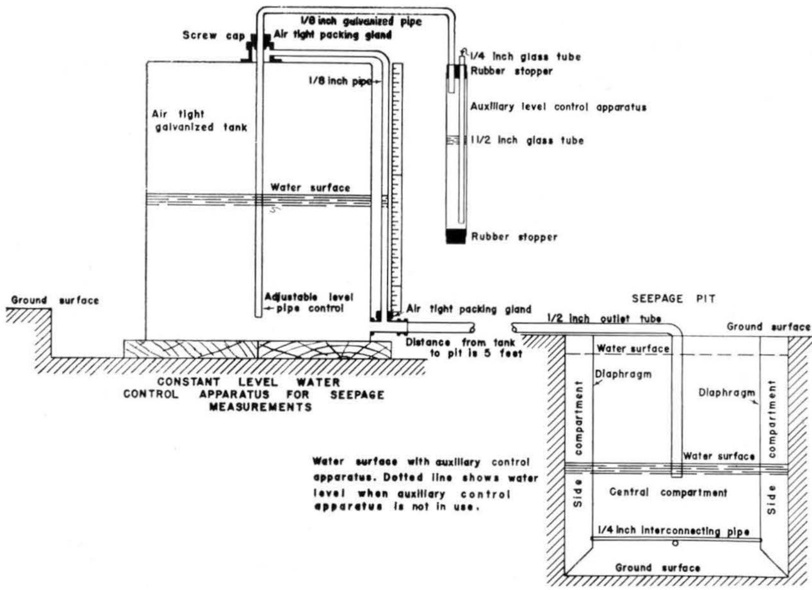

Figure 7.—Section of constant level-control apparatus installed on one of the trenches at Davis, California.

year. In this series the seepage from the ends, sides, and bottom was measured by means of the apparatus previously described. Measurements of the seepage were made when the depths in the trench were 2 feet, 1.5 feet, 1.0 foot, 0.5 foot and 1 inch or 0.083 foot. The surface of the water in all parts of the trench was kept at the same elevation by means of the control apparatus except when the depth was 1 inch. For this condition the level was maintained by regulating the flow into the different parts of the trench by means of valves. The automatic control apparatus failed to function satisfactorily because of leaks in the hose connections caused by the high vacuum required. The tests on effect of depth were made about 9 months after those first reported and some sloughing of the walls had occurred. The trench was kept covered during periods when not in use and before this series of observations on seepage was started, it was seasoned by being kept filled with water for about a day. The first observations were at the 2-foot depth, and at the conclusion of these tests the water level was lowered by successive half-foot steps, until a depth of 0.5 foot was reached. After completing the observations at the 0.5-foot depth, the depths were increased by half-foot steps until the 2-foot depth was again reached. Finally the water was lowered until only an inch remained and the last series of observations was made at this depth. Each series of observations lasted from 3 to 6 hours. The entire series covered the period from January 11 to 16 inclusive.

The results of the observations are set out in table 5, which shows total seepage for 24 hours from each part of the trench as determined from the mean rate during the time of each test, and also shows the unit rates for the

TABLE 5. *Rates of seepage from compartments of Davis, Califorina, Trench No. 2, with various equal depths of water.*

		Seepage per 24 hours											
Depth	Category	Bottom		Sides		West end		East end		Total ends		Total trench	
Ft.		Cu.ft.	Cu.ft. per sq.ft.	Cu.ft.	Cu.ft. per sq.ft.	Cu.ft.	Cu.ft. per sq.ft.	Cu.ft.	Cu.ft. per sq.ft.	Cu.ft.	Cu.ft. per sq.ft.	Cu.ft.	Cu.ft. per sq.ft.
2.00	Down	28.07		29.56		40.54		107.71		148.25		205.88	
	Up	25.76		32.98		48.97		70.03		119.00		177.74	
	Mean	26.92	6.73	31.27	3.91	44.76	3.44	88.87	6.82	133.62	5.14	191.81	5.04
1.50	Down	21.91		23.79		34.13		54.92		89.05		134.75	
	Up	21.04		23.07		41.57		28.95		70.53		114.64	
	Mean	21.48	5.37	23.43	3.91	37.86	3.60	41.94	3.99	79.79	3.80	124.70	4.02
1.00	Down	15.04		10.77		2.55		26.68		29.23		55.04	
	Up	14.32		9.57		8.84		26.32		35.16		59.05	
	Mean	14.68	3.67	10.17	2.54	5.70	0.71	26.46	3.31	32.20	2.01	57.04	2.38
0.50	Down	8.15		2.84		4.20		5.11		9.31		20.30	
	Up	—		—		—		—		—		—	
	Mean	—	2.04	—	1.42	—	0.76	—	0.93	—	0.85	—	1.19
0.083	Down	4.22		0.45		1.80		1.00		2.80		7.47	
	Up	—		—		—		—		—		—	
	Mean	—	1.06	—	1.35	—	0.53	—	0.29	—	0.41	—	0.67

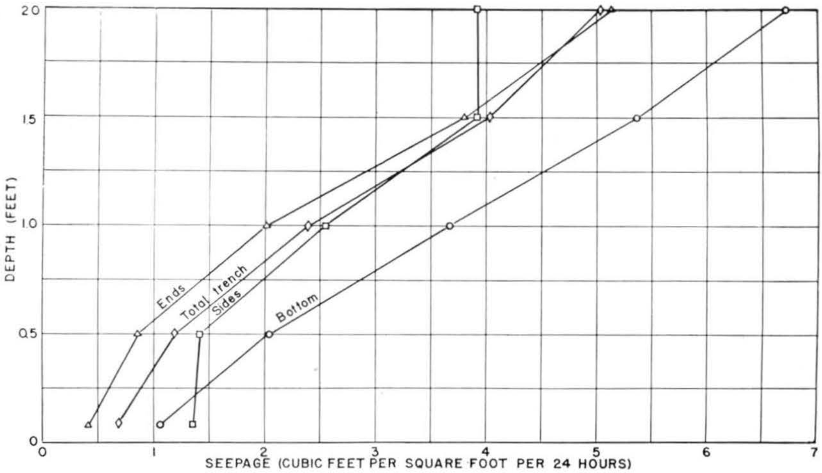

Figure 8.—Effect of depth of water on seepage in different parts of trench at Davis, California.

different parts of the trench. The data are shown plotted in figure 8. During these tests the rate of seepage from the bottom of the trench exceeded that from the sides and the ends at each depth by a significant amount which was contrary to the findings previously reported. It also exceeded the average rate for the whole trench. Since the bottom rate did not change materially and the rate from the sides decreased, it was evident that the changes in the trench that caused the reversal in the bottom and side-seepage rates must have occurred in the sides. Some weathering, no doubt, took place during the 9-month period that the trench was not filled with water, even though it was covered all the time, but why this should reduce the seepage from the sides was not apparent. Actually, the rate of seepage from the sides should have been less than that from the bottom because the average head causing the seepage was less than that on the bottom. The true head causing the seepage, represented by the difference between pressure of the water in the compartment and the pressure of the water in the soil, was not measured. When the seepage was compared with the average depth, the side seepage was shown to exceed that from the bottom.

At the 2.0-foot depth the seepage rate per square foot from the sides was identical with that at 1.5 feet although the total seepage was greater. The increase was in proportion to the increase in area and consequently the rate per unit area remained the same. The hourly rates during the observations at the 2.0-foot depth showed the same trend. If leakage took place into the side compartments from the bottom or ends, the amount must have been small because the water in all compartments was maintained at approximately the same level by the Mariotte control apparatus. Temperature may have had some influence because it was cold enough at night to freeze the gage glasses and the hoses on several occasions during the period these observations were being made. The surface of the ground probably froze also, but this fact was not reported. In any event, the ground to an appreciable depth must have been cold. Such a condition would have reduced the seepage in this area materially and it would also reduce the seepage from the end areas. The effect here, however, would be obscured to some extent by the seepage from the portion of the bottom of the trench included which would not be affected by the temperature.

EFFECTS OF UNEQUAL DEPTHS OF WATER ON SEEPAGE

A series of observations was made on the seepage from the compartments of Trench No. 2 when the depth in the central compartment was 1.5 feet and 2.0 feet in all other compartments. These tests were made in January immediately following those at various depths. The results are set out in table 6. Although the total amounts from the different parts of the trench varied according to the differences in area, the rates per square foot of wetted surface were in surprisingly close agreement. This fact, however, is not significant because the average head on the sides, bottom, ends and whole trench was not the same as that on the bottom compartment. Furthermore, this comparison does not take into account the fact that the pressure in the soil changed when the depth in the center compartment was decreased. The rates for the different parts of the trench, based on the hourly observations. were also consistent.

These data may be compared with those obtained when the water in all compartments of the trench was either 1.5 or 2.0 feet deep. In making the comparison, the observations made when the depths were being increased were used because they were all in a 2-day period at the end of the series during which the effect of time on the observations would be a minimum. The results are shown in table 7.

It will be observed that the seepage from the bottom decreased and that that from the sides and ends increased when the depth of water in the center compartment was 0.5 less than in the ends and sides. The seepage from the whole trench was slightly greater when the depth in the central compartment was 1.5 feet than when the depth in all parts was 2.0 feet. This was due to the fact that the area of the bottom compartment was small compared with the ends and sides, and consequently, the effect of the decrease in rate of seepage from the bottom compartment was overshadowed by the increase from the sides and ends even though small, because of the larger area.

TABLE 6. *Rates of seepage from Davis, California, Trench No. 2 with water 1.5 feet deep in central compartment and 2.0 feet deep in all other compartments.*

Period	Seepage in cubic feet per 24 hours							
	Bottom		Sides		Ends		Total trench	
	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.
1st hr.	18.02	4.50	31.76	3.97	122.52	4.71	172.30	4.53
2nd hr.	18.18	4.55	39.88	4.99	126.78	4.88	184.84	4.86
3rd hr.	17.69	4.42	37.48	4.69	129.33	4.98	184.50	4.86
Mean	17.96	4.49	36.37	4.55	126.21	4.86	180.54	4.75

TABLE 7. *A comparison of the rates of seepage from Davis, California, Trench No. 2 when depth in central compartment was 1.5 feet and in all other compartments 2.0 feet, with those when the depths in all the compartments were 1.5 feet and 2.0 feet.*

Portion of trench	Seepage in cubic feet per 24 hours					
	Depth 1.5 ft. in all compartments		Depth 2.0 ft. in all compartments		Depth 1.5 ft. in central compartment. Depth 2.0 ft. in all others	
	Total[1]	Per sq.ft.[1]	Total[1]	Per sq.ft.[1]	Total	Per sq.ft.
Bottom	21.04	5.21	25.76	6.29	17.96	4.49
Sides	23.07	3.84	32.98	4.12	36.37	4.55
Ends	70.53	3.36	119.00	4.58	126.21	4.86
Total trench	114.64	3.70	177.74	4.68	180.54	4.75

[1]Based on the results of the tests on effect of depth which were made when the depths were being increased, (table 5.)

These comparisons show that the rate of seepage through a given limited area of soil surface submerged to a given depth will be decreased by increasing the depth of water on immediately adjacent areas and will be increased by decreasing the depth on those areas. This relation is based on the fact that the seepage flow from the greater head will exert in the soil at the zone of contact between submerged areas, a superior pressure which will enlarge its avenue of escape and constrict that from the area of less submergence. For this reason a few holes in a canal lining may cause leakage out of proportion to the area involved.

The observations on the seepage from pits and trenches are inconclusive in regard to the belief that side seepage exceeds bottom seepage. They generally show that the seepage rate increases with the depth of water in the canal; but the most important fact brought out is that the seepage rate is variable and may increase or decrease without apparent cause so far as the present knowledge of seepage is concerned.

Pool Experiments

Seepage losses from lined canals and those in very tight soils are generally small. In such canals, losses can be most accurately determined by measuring the drop in pools made by damming both ends of a section of the canal. There is a question whether the seepage from a pool is the same as that from flowing water, since experiments in water-spreading areas have shown that velocities as low as 0.05 foot per second increased the rate of percolation (6). It is believed, however, that the errors resulting from this cause are small in comparison with those that would be introduced if an attempt were made to determine small seepage losses by means of current-meter measurements.

In making the tests, sections of canals were chosen where checks were available to form pools or where no difficulty would be experienced in damming off a section of the canal. The lengths of the sections of the canals chosen were such that the pools formed would have a reasonably uniform depth throughout. The drop in the water surface was measured with gages located at the ends of the pools and sometimes at intermediate points also. Readings were taken twice daily or oftener, depending on the rate of loss. Leaks through headgates and checks were reduced as much as possible and those remaining were mostly small. They were measured volumetrically or by means of triangular weirs. The evaporation loss was measured by means of a special hook gage in a circular pan 8 inches in diameter suspended in the water. The air and water temperatures also were observed. Since little or no rain fell during the time observations were being taken, corrections seldom had to be applied to the evaporation or drop in the water surface in the pools for this reason.

Canal cross-sections were obtained from original design data, but were checked in the field. Lengths of sections were taken from original survey notes and maps or were measured in the field when other sources of information were not available. The data on cross-sections were plotted and from these plots and the lengths of the sections, the water surface areas and the wetted areas were computed. With this information and the net drop in the water surface of the pool, it was possible to compute the seepage loss in cubic feet per square foot of wetted area. The loss in cubic feet per mile and percentage per mile was not determined because these values depend on the quantity being carried by the canal.

EAST CONTRA COSTA IRRIGATION DISTRICT CANALS AND LATERALS

Most of the observations on seepage from canals by the pool method were made on the basins and laterals of the East Contra Costa Irrigation District at Brentwood, California. The main canal consists of six concrete-lined basins at successively higher elevations and extending from Indian Slough about 5½ miles west to the foothills. Laterals are laid out to the north and south from the basins. Water is raised from Indian Slough into Basin No. 1 by a pumping plant containing four horizontal centrifugal pumps with a combined capacity of 117 cubic feet per second. A second plant lifts the water required for the upper laterals from Basin No. 1 to Basin No. 2 and so on to Pumping Plant No. 7, which lifts the water required directly into a lateral. The basins have bottom widths ranging from 7.50 feet for Basin No. 1 to 5.17 feet for Basin No. 6. The side slopes were 1½ to 1, and the grade toward the west was 0.0002. The depth of the basins ranged from 4.50 feet to 6.40 feet. Since the slope of the land is adverse to the canal, each basin starts on a fill and ends in excavation. One of the basins is shown in figure 9.

Figure 9.—Portion of pool in East Contra Costa main canal basin, Brentwood, California.

Construction work on the project was started in 1912 and the basins were completed in 1913. They were lined throughout with a 3-inch layer of concrete covered with ½ inch of gunite. Although frequent expansion joints were provided, the lining showed extensive small cracks in the fall of 1922, when the seepage measurements were made. (See figure 10.) This was probably due to the expansion and contraction of the soil, which ranges from heavy clay loam to adobe.

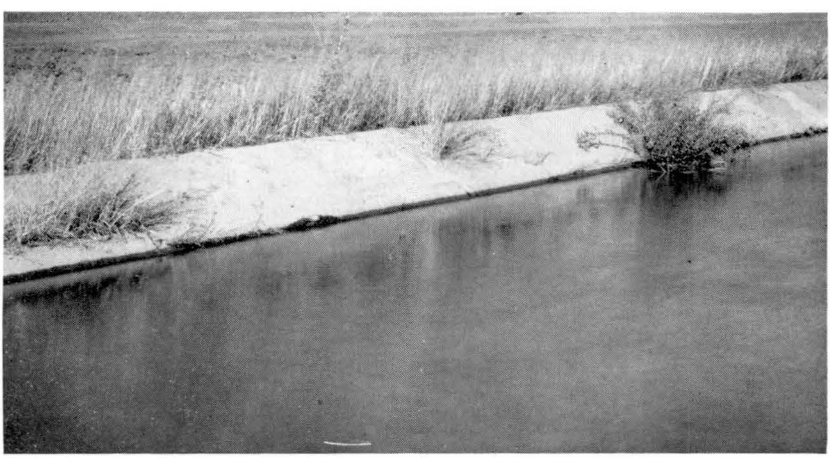

Figure 10.—Cracks in concrete lining of East Contra Costa main canal basin, Brentwood, California.

The observations on the seepage from the basins were conducted during periods when the basins were full of water and no diversions to the laterals were being made. There was a small amount of leakage from each basin into the transformer cooling coils which was measured volumetrically, and there was also leakage of an undetermined amount through the gate valves in the pump discharge lines. These leaks could not be measured, but from the sound they made they were judged to be small. They were also, to some extent, compensating because water from this source leaked into the west end of each basin and out the east end. There were no appreciable leaks through the headgates leading to the laterals.

The results of the seepage observations on the basins are given in table 8. With few exceptions the seepage losses found were quite small and where they were higher than expected there was the possibility that water was turned into a lateral for a short period without being reported. From the predominance of the low rates, it is reasonable to assume that the actual seepage was definitely less than 0.1 cubic foot per square foot per 24 hours. Within the range of heads and temperatures prevailing during these tests there was no indication that the seepage loss decreased with the depths in the basins or that it decreased with the temperature. The unavoidable inaccuracies in the data apparently overshadowed the effect of these factors.

East Contra Costa Laterals

Observations on the seepage from pools in selected laterals of the East Contra Costa Irrigation District were made during October 1923. The following laterals were chosen for the tests:

Lateral No 3 South in clay, untreated
Lateral No. 5 South in clay, treated by puddling by sheep
Lateral No. 2 North in clay, untreated
Lateral No. 6 North in heavy clay, puddled
Lateral No. 3 North in fine sand, first use of lateral
Lateral No. 6 South in clay, concrete lined
Lateral No. 7 North in clay, concrete lined

The tests on laterals No. 3 South, No. 5 South, No. 2 North and No. 6 North were made to determine the effect of puddling on the seepage loss. Two of the laterals had previously been puddled by the trampling of sheep and two had received no special treatment. The laterals were excavated in soils ranging from clay loam to heavy clay, all of which cracked badly when dried. Test borings showed no hardpan substratum. Laterals numbered 3 South, 5 South and 2 North were excavated in clay loam, the difference in the laterals being that No. 2 North was constructed in 1922, No. 3 South was constructed in 1913, and No. 5 South had previously been puddled. Lateral No. 6 North was excavated in the heaviest clay and had also been puddled. After several years of use the banks of the laterals had changed considerably

TABLE 8. *Summary of results of pool measurements of canal seepage.*

Series	Period		Interval	Length of pool	Mean depth	Area[1]		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
	From	To				Water surface	Wetted surface			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
			Days	Feet	Feet	Acres	Acres	Acres	Feet	Cu. Ft.	Cu. Ft.
East Contra Costa Irrigation District, Basin No. 1—concrete lined.											
1	Sept. 2	Sept. 4	1.538	2132	4.33	1.122	1.258	0.094	0.014	0.080	0.071
2	Nov. 13	Nov. 14	.672		3.46	.997	1.108	.060	.007	.053	.048
3	Nov. 14	Nov. 15	.677		3.98	1.071	1.100	.111	.007	.104	.101
4	Nov. 24	Dec. 4	10.054		1.32	.657	.700	.053	.007	.046	.043
										Mean	0.066
East Contra Costa Irrigation District, Basin No. 2—concrete lined.											
1	Sept. 2	Sept. 4	1.535	7076	5.11-3.65[2]	3.518	3.902	.085	.020	.065	.059
2	Nov. 13	Nov. 14	.663		5.16-3.57	3.508	3.963	.063	.013	.050	.044
3	Nov. 14	Nov. 15	.677		5.46-3.86	3.591	4.062	.047	.013	.034	.030
4	Nov. 15	Nov. 24	9.226		5.21-3.62	3.467	3.914	.049	.013	.036	.032
5	Nov. 24	Dec. 4	10.050		4.74-3.16	3.234	3.635	.047	.007	.040	.036
										Mean	0.040
East Contra Costa Irrigation District, Basin No. 3—concrete lined.											
1	Sept. 2	Sept. 4	2.024	3604	4.24	1.694	1.911	.141	.047	.094	.083
2	Nov. 13	Nov. 14	.654		3.98	1.632	1.834	.073	.007	.066	.059
3	Nov. 14	Nov. 15	.677		4.45	1.753	1.982	.100	.007	.093	.082
4	Nov. 15	Nov. 24	9.249		4.18	1.686	1.902	.052	.007	.045	.040
5	Nov. 24	Dec. 24	10.038		3.41	1.489	1.666	.105	.007	.098	.088
										Mean	0.070

[1] For basins No. 1 to 6, area of water surface and wetted surface are given in acres. For the others, the average width of water surface and length of wetted perimeter are shown in feet.

[2] Pool in two parts separated by road culvert.

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Period From	Period To	Interval	Length of pool	Mean depth	Area[1] Water surface	Area[1] Wetted surface	Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage Per sq. ft. water surface per 24 hr.	Seepage Per sq. ft. wetted surface per 24 hr.
			Days	Feet	Feet	Acres	Acres	Feet	Feet	Cu. Ft	Cu. Ft.
East Contra Costa Irrigation District, Basin No. 4—concrete lined.											
1	Sept. 2	Sept. 4	2.025	4662	4.13	2.084	2.358	0.103	0.076	0.027	0.024
2	Nov. 13	Nov. 14	.648		3.68	1.927	2.181	.089	.027	.062	.055
3	Nov. 14	Nov. 15	.680		4.47	2.193	2.488	.088	.024	.064	.056
4	Nov. 15	Nov. 24	9.268		4.18	2.091	2.375	.058	.025	.033	.029
										Mean	0.041
East Contra Costa Irrigation District, Basin No. 5—concrete lined.											
1	Sept. 2	Sept. 4	2.036	3255	3.58	1.289	1.453	.295	.084	.211	.187
2	Nov. 13	Nov. 14	.638		2.45	1.021	1.099	.122	.050	.072	.067
3	Nov. 14	Nov. 15	.678		4.10	1.395	1.580	.292	.039	.253	.223
4	Nov. 15	Nov. 24	9.294		3.60	1.279	1.442	.088	.041	.047	.042
5	Nov. 24	Dec. 4	10.034		2.50	1.027	1.142	.136	.007	.129	.116
										Mean	0.127
East Contra Costa Irrigation District, Basin No. 6—concrete lined.											
1	Sept. 2	Sept. 4	2.036	7999	4.38-3.08[2]	3.036	3.447	.197	.039	.158	.139
2	Nov. 13	Nov. 14	.633		3.95-2.65	2.779	3.141	.103	.011	.092	.081
3	Nov. 14	Nov. 15	.679		4.40-3.10	3.054	3.490	.228	.011	.217	.190
4	Nov. 15	Nov. 24	9.334		4.11-2.81	2.884	3.265	.061	.011	.050	.044
5	Nov. 24	Dec. 4	10.034		3.36-2.06	2.474	2.776	.091	.007	.084	.075
										Mean	0.106

[1] For basins No. 1 to 6, area of water surface and wetted surface are given in acres. For the others, the average width of water surface and length of wetted perimeter are shown in feet.

[2] Pool in two parts.

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
East Contra Costa Irrigation District, Lateral No. 3 South—clay, untreated.												
1	10/3/23	5:19P	8:49A	930	1418	2.102	9.55	10.61	1.445	0.021	1.424	1.282
	10/4/23	8:49A	10:48A	119		1.578	7.82	8.56	1.385	.021	1.364	1.244
	10/4/23	10:48A	1:42P	174		1.438	7.34	8.04	1.360	.021	1.339	1.212
	10/4/23	1:42P	4:53P	191		1.262	6.76	7.36	1.395	.021	1.374	1.262
2	10/5/23	2:49P	4:07P	78	1418	2.601	11.25	12.56	1.315	.021	1.294	1.157
	10/5/23	4:07P	5:08P	61		2.537	11.05	12.33	1.360	.021	1.339	1.200
	10/5/23	5:08P	7:58P	170		2.428	10.65	11.86	1.355	.021	1.334	1.196
	10/5/23	7:58P	8:10A	732		1.999	9.22	10.21	1.375	.021	1.354	1.223
	10/6/23	8:10A	2.25P	375		1.468	7.44	8.14	1.400	.021	1.379	1.261
	10/6/23	2:25P	5:31P	186		1.190	6.52	7.08	1.460	.021	1.439	1.325
3	10/8/23	9:05A	11:03A	118	1418	2.676	11.52	12.86	.957	.021	.936	.838
	10/8/23	11:03A	3:50P	287		2.536	11.05	12.32	1.010	.021	.989	.886
	10/8/23	3:50P	8:00P	250		2.347	10.37	11.57	1.005	.021	.984	.883
4	10/9/23	9:51A	11:23A	92	1418	2.438	10.69	11.92	1.005	.021	.984	.883
	10/9/23	11:23A	1:42P	139		2.358	10.43	11.62	1.003	.021	.982	.881
	10/9/23	1:42P	4:57P	195		2.235	10.00	11.14	1.085	.021	1.064	.957
	10/9/23	4:57P	8:24A	927		1.801	8.55	9.42	1.125	.021	1.104	1.004
5	10/10/23	11:25A	3:27P	242	1418	2.377	10.49	11.70	1.115	.021	1.094	.983
	10/10/23	3:27P	5:47P	140		2.223	9.88	11.08	1.225	.021	1.204	1.081
	10/10/23	5:47P	8:09A	862		1.794	8.53	9.38	1.230	.021	1.209	1.098
	10/11/23	8:09A	5:40P	571		1.187	6.52	7.08	1.195	.021	1.174	1.079
East Contra Costa Irrigation District, Lateral No. 5 South—clay, puddled.												
2	10/24/23	12:26P	1:37P	71	1620	1.440	10.04	10.19	9.12	.01	9.11	8.98
	10/24/23	1:37P	2:40P	63		.942	8.08	8.13	7.66	.01	7.65	7.61
3	10/25/23	10:15A	10:45A	30	1620	1.779	11.52	11.76	7.46	.01	7.45	7.30
	10/25/23	10:45A	11:16A	31		1.631	10.87	11.07	7.66	.01	7.65	7.50
	10/25/23	11:16A	12:01P	45		1.439	10.04	10.18	6.99	.01	6.98	6.89
	10/25/23	12:01P	2:01P	120		1.088	8.30	8.37	5.73	.01	5.72	5.68

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
East Contra Costa Irrigation District, Lateral No. 2 North—clay, untreated.												
2	10/4/23	5:31P	7:38P	127	1327	2.832	15.07	16.38	4.95	0.02	4.93	4.53
	10/4/23	7:38P	8:40P	62		2.515	13.87	15.07	4.51	.02	4.49	4.13
	10/4/23	8:40P	8:41A	721		1.561	10.68	11.24	3.44	.02	3.42	3.25
	10/5/23	8:41A	11:25A	164		.574	7.19	7.25	2.22	.02	2.20	2.18
3	10/5/23	2:38P	3:40P	62	1327	2.861	15.20	16.50	5.07	.02	5.05	4.65
	10/5/23	3:40P	4:40P	60		2.658	14.37	15.65	4.60	.02	4.58	4.20
	10/5/23	4:40P	5:40P	60		2.471	13.75	14.89	4.47	.02	4.45	4.11
	10/5/23	5:40P	8:27P	167		2.141	12.66	13.57	4.06	.02	4.04	3.77
	10/5/23	8:27P	8:39A	732		1.656	9.50	9.89	2.76	.02	2.74	2.63
4	10/6/23	2:11P	3:11P	60	1327	2.203	12.88	13.81	4.03	.02	4.01	3.74
	10/6/23	3:11P	4:19P	68		2.030	12.29	13.12	3.70	.02	3.68	3.45
	10/6/23	4:19P	5:21P	62		1.869	11.74	12.47	3.45	.02	3.43	3.23
	10/6/23	5:21P	8:01P	169		1.610	10.84	11.41	3.16	.02	3.14	2.98
5	10/8/23	10:27A	11:31A	64	1327	2.305	13.20	14.23	5.55	.02	5.53	5.13
	10/8/23	11:31A	1:15P	104		2.028	12.29	13.12	4.08	.02	4.06	3.80
	10/8/23	1:15P	4:11P	176		1.670	11.05	11.65	3.46	.02	3.44	3.26
	10/8/23	4:11P	8:28P	257		1.213	9.53	9.95	2.76	.02	2.74	2.63
East Contra Costa Irrigation District, Lateral No. 6 North—heavy clay, puddled.												
1	10/4/23	11:20A	2:14P	174	1291	2.493	12.14	13.07	0.335	.021	0.314	0.292
	10/4/23	2:14P	5:51P	217		2.446	12.00	12.89	.286	.021	.265	.247
	10/4/23	5:51P	9:09A	918		2.353	11.70	12.53	.231	.021	.210	.196
	10/5/23	9:09A	11:40A	151		2.270	11.42	12.25	.207	.021	.186	.174
	10/5/23	11:40A	6:03P	383		2.231	11.30	12.07	.210	.021	.189	.177
	10/5/23	6:03P	9:05A	902		2.156	11.05	11.79	.144	.003[1]	.147	.137
	10/6/23	9:05A	6:38P	573		2.080	10.80	11.49	.157	.021	.136	.128
	10/6/23	6:38P	9:39A	901		2.004	10.55	11.22	.146	.003[1]	.149	.140
	10/7/23	9:39A	8:20A	1361		1.902	10.23	10.85	.118	.021	.097	.091
	10/8/23	8:20A	5:49P	569		1.825	9.97	10.55	.109	.021	.088	.083
	10/8/23	5:49P	12:14P	1105		1.766	9.78	10.32	.100	.021	.079	.075
	10/9/23	12:14P	10:03A	1309		1.684	9.52	10.03	.100	.021	.079	.075
	10/10/23	10:03A	12:58P	1615		1.580	9.17	9.64	.094	.021	.073	.069
	10/11/23	12:58P	9:15A	1217		1.477	8.84	9.23	.108	.021	.087	.083
	10/12/23	9:15A	9:56A	2921		1.331	8.37	8.70	.104	.021	.083	.080
	10/14/23	9:56A	1:56P	8880		.929	7.03	7.20	.098	.021	.077	.075

[1] Gain.

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
East Contra Costa Irrigation District, Lateral No. 3 North—fine sand, first use of lateral.												
3	10/6/23	10:06.8A	10:22.2A	15.4	323.5	2.846	14.60	16.09	30.02	0.02	30.00	27.20
	10/6/23	10:22.2A	10:37.2A	15.0		2.540	13.48	14.90	28.47	.02	28.45	25.70
	10/6/23	10:37.2A	10:52.2A	15.0		2.247	12.47	13.70	27.46	.02	27.44	25.00
	10/6/23	10:52.2A	11:05.5A	13.3		1.994	11.60	12.62	23.77	.02	23.75	21.80
	10/6/23	11:05.5A	11:20.5A	15.0		1.762	10.82	11.69	23.39	.02	23.37	21.70
	10/6/23	11:20.5A	11:34.9A	14.4		1.533	10.02	10.79	20.20	.02	20.18	18.70
	10/6/23	11:34.9A	11:50.1A	15.2		1.336	9.30	9.98	17.23	.02	17.21	16.00
	10/6/23	11:50.1A	12:05.2P	15.1		1.165	8.72	9.31	14.07	0.02	14.05	13.17
	10/6/23	12:05.2P	12:20.3P	15.1		1.033	8.25	8.75	11.49	.02	11.47	10.82
	10/6/23	12:20.3P	12:35.1P	14.8		.919	7.84	8.30	10.82	.02	10.80	10.20
	10/6/23	12:35.1P	12:50.5P	15.4		.813	7.45	7.85	9.01	.02	8.99	8.54
	10/6/23	12:50.5P	1:05.4P	14.9		.718	7.11	7.46	8.80	.02	8.78	8.36
7	10/10/23	1:04.1P	1:20.2P	16.1	323.5	2.162	12.18	13.32	17.83	.02	17.81	16.37
	10/10/23	1:20.2P	1:35.4P	15.2		1.981	11.57	12.55	15.24	.02	15.22	14.00
	10/10/23	1:35.4P	1:49.8P	14.4		1.835	11.04	11.96	12.58	.02	12.56	11.60
	10/10/23	1:49.8P	2:05.5P	15.7		1.715	10.64	11.50	10.10	.02	10.08	9.34
	10/10/23	2:05.5P	2:21.2P	15.7		1.613	10.30	11.10	8.44	.02	8.42	7.81
	10/10/23	2:21.2P	2:35.8P	14.6		1.530	10.02	10.79	7.02	.02	7.00	6.50
	10/10/23	2:35.8P	2:50.8P	15.0		1.460	9.77	10.48	6.63	.02	6.61	6.16
	10/10/23	2:50.8P	4:17.3P	86.5		1.261	9.06	9.68	5.44	.02	5.42	5.07
	10/10/23	4:17.3P	4:32.5P	15.2		1.076	8.38	8.93	4.51	.02	4.49	4.22
	10/10/23	4:32.5P	4:47.8P	15.3		1.026	8.23	8.75	4.78	.02	4.76	4.47
	10/10/23	4:47.8P	5:02.5P	14.7		.979	8.04	8.53	4.04	.02	4.02	3.78
	10/10/23	5:02.5P	5:17.5P	15.0		.934	7.88	8.35	4.50	.02	4.48	4.23
Several yards of clay added to water at inlet when refilling pool.												
8	10/12/23	10:03.0A	10:19.0A	16.0	323.5	2.819	14.50	16.02	22.79	.02	22.77	20.60
	10/12/23	10:19.0A	10:33.4A	14.4		2.584	13.64	15.08	21.15	.02	21.13	19.10
	10/12/23	10:33.4A	10:49.5A	16.1		2.371	12.89	14.22	18.63	.02	18.61	16.90
	10/12/23	10:49.5A	11:04.6A	15.1		2.179	12.25	13.42	16.60	.02	16.58	15.13
	10/12/23	11:04.6A	11:19.4A	14.8		2.020	11.69	12.74	13.73	.02	13.71	12.61
	10/12/23	11:19.4A	11:35.2A	15.8		1.886	11.25	12.18	11.14	.02	11.12	10.26
	10/12/23	11:35.2A	11:50.5A	15.3		1.777	10.85	11.75	8.74	.02	8.72	8.05
	10/12/23	11:50.5A	12:05.4P	14.9		1.690	10.57	11.41	7.34	.02	7.32	6.79
	10/12/23	12:05.4P	12:20.5P	15.1		1.615	10.33	11.14	6.90	.02	6.88	6.38
	10/12/23	12:20.5P	12:35.7P	15.2		1.547	10.08	10.85	6.34	.02	6.32	5.86

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
	10/12/23	12:35.7P	12:50.6P	14.9	323.5	1.485	9.86	10.57	5.42	0.02	5.40	5.04
	10/12/23	12:50.6P	1:46.5P	55.9		1.358	9.40	10.08	5.04	.02	5.02	4.68
	10/12/23	1:46.5P	2:02.0P	15.5		1.238	8.96	9.58	3.68	.02	3.66	3.43
	10/12/23	2:02.0P	2:32.4P	30.4		1.175	8.75	9.34	3.94	.02	3.92	3.67
	10/12/23	2:32.4P	3:03.2P	30.8		1.090	8.44	9.00	3.94	.02	3.92	3.68
	10/12/23	3:03.2P	3:32.3P	29.1		1.015	8.16	8.68	3.44	.02	3.42	3.21
	10/12/23	3:32.3P	4:02.4P	30.1		.945	7.94	8.41	3.43	.02	3.41	3.22
	10/12/23	4:02.4P	4:32.5P	30.1		.876	7.67	8.10	3.24	.02	3.22	3.05
	10/12/23	4:32.5P	5:03.1P	30.6		.809	7.45	7.85	3.07	.02	3.05	2.90
East Contra Costa Irrigation District, Lateral No. 6 South—in clay, concrete lined.												
1	10/23/23	3:53P	5:20P	87	850	2.324	11.81	13.06	3.02	.01	3.01	2.72
	10/23/23	5:20P	8:44P	924		1.441	8.80	9.57	2.46	.01	2.45	2.25
	10/24/23	8:44P	12:11P	207		.538	5.73	6.00	1.50	.01	1.49	1.42
2	10/24/23	2:25P	3:25P	60	850	2.556	12.60	13.98	2.90	.01	2.89	2.61
	10/24/23	3:25P	4:25P	60		2.434	12.16	13.52	3.05	.01	3.04	2.74
	10/24/23	4:25P	5:25P	60		2.310	11.76	13.00	2.82	.01	2.81	2.54
	10/24/23	5:25P	8:35P	190		2.072	10.94	12.06	2.75	.01	2.74	2.48
	10/24/23	8:35P	8:17A	702		1.338	8.46	9.16	2.25	.01	2.24	2.07
	10/25/23	8:17A	9:43A	86		.735	6.41	6.79	1.67	.01	1.66	1.56
	10/25/23	9:43A	11:45A	122		.618	6.00	6.31	1.54	.01	1.53	1.45
3	10/25/23	2:28P	3:26P	58	850	2.564	12.61	14.00	3.46	.01	3.45	3.11
	10/25/23	3:26P	4:26P	60		2.434	12.16	13.50	2.55	.01	2.54	2.30
	10/25/23	4:26P	5:26P	60		2.331	11.83	13.08	2.49	.01	2.48	2.24
	10/25/23	5:26P	7:41P	135		2.171	11.28	12.45	2.33	.01	2.32	2.10
	10/25/23	7:41P	8:10A	749		1.560	9.19	10.01	1.93	.01	1.92	1.76
	10/25/23	8:10A	2:24P	374		.885	6.91	7.37	1.32	.01	1.31	1.23
	10/25/23	2:24P	5:14P	170		.658	6.12	6.47	.92	.01	.91	.86
East Contra Costa Irrigation District, Lateral No. 7 North—in clay, concrete lined.												
1	10/23/23	11:54A	2:32P	158	288.5	2.063	10.54	11.75	0.456	.014	.442	.396
	10/23/23	2:32P	5:02P	150		2.025	10.40	11.60	.322	.014	.308	.276
	10/23/23	5:02P	8:18A	916		1.913	10.05	11.16	.303	.014	.289	.259
	10/24/23	8:18A	11:59A	221		1.800	9.67	10.75	.218	.014	.204	.184
	10/24/23	11:59A	4:49P	290		1.758	9.53	10.58	.238	.014	.224	.202
	10/24/23	4:49P	8:46A	957		1.670	9.30	10.26	.195	.014	.181	.164
	10/25/23	8:46A	11:32A	166		1.599	9.03	9.99	.144	.014	.130	.118
	10/25/23	11:32A	5:01P	329		1.574	8.97	9.90	.140	.014	.126	.114

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
East Contra Costa Irrigation District, Lateral No. 7 North—in clay, concrete lined (continued).												
1	10/25/23	5:01P	9:08A	967	288.5	1.514	8.78	9.67	0.128	0.014	0.114	0.104
	10/26/23	9:08A	2:12P	304		1.455	8.61	9.45	.137	.014	.123	.112
	10/26/23	2:12P	4:59P	167		1.434	8.54	9.38	.100	.014	.086	.078
	10/26/23	4:59P	8:24A	925		1.392	8.40	9.22	.116	.014	.102	.093
	10/27/23	8:24A	8:15P	711		1.329	8.21	8.99	.106	.014	.092	.084
	10/27/23	8:15P	10:31A	856		1.272	8.05	8.78	.103	.014	.089	.082
Turlock Irrigation District, Lateral No. 5½—concrete lined.												
1	9/6/23	11:05A	6:06P	421	3090	2.25	10.65	12.08	1.456	1.302	0.154	0.136
	9/6/23	6:06P	9:03A	907		1.92	9.70	10.91	.429	.377	.052	.046
2	9/7/23	3:22P	6:25P	183	3090	2.72	12.00	13.71	.556	.425	.131	.115
	9/7/23	6:25P	9:10A	885		2.52	11.43	13.02	.538	.447	.091	.080
Fresno Irrigation District, Houghton Lateral—sandy loam, untreated.												
1	8/22/23	1:45P	5:25P	220	9150	1.83	22.3	23.1	0.255	0.023[1]	0.278	0.270
	8/22/23	5:25P	9:30A	965		1.74	21.8	22.5	.206	.018[1]	.224	.217
	8/23/23	9:30A	4:05P	395		1.64	21.0	21.7	.276	.014[1]	.290	.281
	8/23/23	4:05P	8:34A	989		1.55	20.4	21.0	.154	.019[1]	.173	.168
	8/24/23	8:34A	5:20P	526		1.47	19.8	20.6	.153	.034[1]	.187	.180
	8/24/23	5:20P	9:19A	959		1.40	19.5	20.2	.137	.026[1]	.163	.157
North Pool, Farm Pumping Plant Lateral, Gilcrest, Colorado—sand, untreated.												
2	3/13/44	1:06P	1:11P	5	211.1	1.750	6.70	7.59	23.04		23.04	20.31
	3/13/44	1:11P	1:16P	5		1.675	6.56	7.38	20.16		20.16	17.92
	3/13/44	1:16P	1:21P	5		1.605	6.42	7.17	20.16		20.16	18.05
	3/13/44	1:21P	1:26P	5		1.535	6.28	6.97	20.16		20.16	18.15
	3/13/44	1:26P	1:31P	5		1.465	6.13	6.78	20.16		20.16	18.22
	3/13/44	1:31P	1:36P	5		1.400	6.00	6.60	17.28		17.28	15.70
	3/13/44	1:36P	1:41P	5		1.335	5.86	6.41	20.16		20.16	18.41
	3/13/44	1:41P	1:46P	5		1.275	5.72	6.24	14.40		14.40	13.20
	3/13/44	1:46P	1:51P	5		1.220	5.60	6.09	17.28		17.28	15.90
	3/13/44	1:51P	1:56P	5		1.160	5.46	5.92	17.28		17.28	15.95
	3/13/44	1:56P	2:01P	5		1.100	5.34	5.76	17.28		17.28	16.00
	3/13/44	2:01P	2:06P	5		1.045	5.21	5.61	14.40		14.40	13.37
	3/13/44	2:06P	2:11P	5		.990	5.08	5.45	17.28		17.28	16.10

[1] Gain

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period From	Period To	Interval	Length of pool	Mean depth	Average Width water surface	Average Length wetted perimeter	Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage Per sq. ft. water surface per 24 hr.	Seepage Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
South Pool, Farm Pumping Plant Lateral, Gilcrest, Colorado—sand, untreated.												
2	3/13/44	1:06P	1:11P	5	182.5	1.242	6.72	7.34	10.08		10.08	9.89
	3/13/44	1:11P	1:16P	5		1.205	6.64	7.22	11.52		11.52	10.59
	3/13/44	1:16P	1:21P	5		1.164	6.53	7.08	12.10		12.10	11.16
	3/13/44	1:21P	1:26P	5		1.124	6.42	6.94	10.94		10.94	10.11
	3/13/44	1:26P	1:31P	5		1.084	6.31	6.81	12.10		12.10	11.20
	3/13/44	1:31P	1:36P	5		1.042	6.20	6.67	12.10		12.10	11.25
	3/13/44	1:36P	1:41P	5		1.007	6.10	6.56	8.06		8.06	7.50
	3/13/44	1:41P	1:46P	5		.973	6.00	6.43	11.52		11.52	10.75
	3/13/44	1:46P	1:51P	5		.937	5.88	6.30	9.22		9.22	8.60
	3/13/44	1:51P	1:56P	5		.901	5.77	6.16	11.52		11.52	10.79
	3/13/44	1:56P	2:01P	5		.864	5.66	6.04	10.08		10.08	9.45
	3/13/44	2:01P	2:06P	5		.826	5.56	5.91	11.52		11.52	10.84
	3/13/44	2:06P	2:11P	5		.791	5.45	5.78	8.64		8.64	8.14
College East Farm, Fort Collins, Colorado, N-S Lateral—earth, untreated.												
2	3/26/45	2:13P	2:43P	30	355	1.331	4.67	5.56	8.74		8.74	7.34
	3/26/45	2:43P	3:13P	30		1.161	3.89	4.56	7.68		7.68	6.55
	3/26/45	3:13P	3:43P	30		1.004	3.00	3.44	7.30		7.30	6.37
	3/26/45	3:43P	4:13P	30		.858	2.39	2.64	6.72		6.72	6.08
3	3/26/45	4:58P	5:28P	30	355	1.336	4.67	5.56	7.20		7.20	6.05
	3/26/45	5:28P	5:58P	30		1.191	3.95	4.64	6.82		6.82	5.82
	3/26/45	5:58P	7:06P	30		.971	2.88	3.24	6.31		6.31	5.61
College East Farm, Fort Collins, Colorado, E-W Lateral—earth, untreated.												
3	3/27/45	3:30P	3:50P	20	141	0.938	3.08	3.75	7.35		7.35	6.04
	3/27/45	3:50P	4:10P	20		.840	2.91	3.50	6.34		6.34	5.27
	3/27/45	4:10P	4:30P	20		.760	2.77	3.30	4.90		4.90	4.11
4	3/27/45	4:50P	5:10P	20	141	.970	3.15	3.85	6.48		6.48	5.30
	3/27/45	5:10P	5:30P	20		.880	2.93	3.65	6.48		6.48	5.20
	3/27/45	5:30P	5:50P	20		.798	2.79	3.35	5.62		5.62	4.68
College East Farm, Fort Collins, Colorado, E-W Lateral—2-inch concrete lining.												
1	9/13/45	2:09P	5:30P	201	218.4	0.98	2.54	3.45	0.322	0.009	0.313	0.231
	9/13/45	5:30P	8:44A	914		.74	2.40	3.10	.414	.006	.408	.316
	9/14/45	8:44A	12:50P	246		.65	2.19	2.70	.469	.008	.461	.374

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period		Interval	Length of pool	Mean depth	Average		Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage	
		From	To				Width water surface	Length wetted perimeter			Per sq. ft. water surface per 24 hr.	Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
College East Farm, Fort Collins, Colorado, E-W Lateral—2½-inch concrete lining												
1	9/13/45	2:12P	5:32P	200	296.1	0.98	2.52	3.47	0.713	0.009	0.704	0.512
	9/13/45	5:32P	8:46A	914		.72	2.27	2.84	.674	.006	.668	.534
	9/14/45	8:46A	12:55P	249		.45	1.94	2.22	.607	.008	.599	.523
College East Farm, Fort Collins, Colorado, E-W Lateral—3-inch concrete lining.												
1	9/13/45	2:15P	5:34P	199	307.2	0.86	2.42	3.16	0.768	0.009	0.759	0.582
	9/13/45	5:34P	8:52A	918		.55	2.19	2.69	.496	.006	.490	.399
	9/14/45	8:52A	12:59P	247		.46	1.94	2.38	.385	.008	.377	.307
College East Farm, Fort Collins, Colorado, N-S Lateral—2-inch oil lining.												
1	9/13/45	2:19P	3:07P	48	136.1	.88	4.31	4.89	2.22	0.01	2.21	1.95
	9/13/45	3:07P	3:35P	28		.83	4.16	4.71	2.01	.01	2.00	1.77
	9/13/45	3:35P	4:08P	33		.78	4.03	4.55	2.27	.01	2.26	2.00
	9/13/45	4:08P	4:40P	32		.74	3.93	4.40	1.75	.01	1.74	1.55
	9/13/45	4:40P	5:06P	26		.70	3.82	4.27	1.83	.01	1.82	1.63
	9/13/45	5:06P	5:36P	30		.68	3.74	4.16	1.58	.01	1.57	1.41
College East Farm, Fort Collins, Colorado, N-S Lateral—3-inch bentonite lining.												
1	9/13/45	2:21P	3:08P	47	63.0	.61	3.98	4.30	2.36	0.01	2.35	2.18
	9/13/45	3:08P	3:37P	29		.55	3.69	4.05	2.14	.01	2.13	1.94
	9/13/45	3:37P	4:09P	32		.51	3.58	3.85	2.43	.01	2.42	2.25
	9/13/45	4:09P	4:41P	32		.45	3.40	3.64	2.30	.01	2.29	2.14
	9/13/45	4:41P	5:07P	26		.41	3.22	3.43	1.83	.01	1.82	1.71
	9/13/45	5:07P	5:38P	31		.37	3.11	3.32	2.04	.01	2.03	1.90
College East Farm, Fort Collins, Colorado, E-W Lateral—2-inch concrete lining.												
2	6/11/46	10:52A	12:52P	120	218.4	0.986	2.53	3.35	0.384	0.016	0.368	0.278
	6/11/46	12:52P	2:52P	120		.956	2.50	3.28	.336	.016	.320	.244
	6/11/46	2:52P	4:52P	120		.931	2.48	3.23	.288	.016	.272	.209
	6/11/46	4:52P	7:52P	180		.907	2.45	3.17	.192	.016	.176	.136
	6/11/46	7:52P	8:52A	780		.802	2.33	2.93	.340	.026	.314	.250
	6/12/46	8:52A	10:52A	120		.697	2.20	2.67	.312	.031	.281	.231

TABLE 8. Continued—*Summary of results of pool measurements of canal seepage.*

Series	Date	Period From	Period To	Interval	Length of pool	Mean depth	Average Width water surface	Average Length wetted perimeter	Mean drop in water surface per 24 hr.	Evaporation plus leakage per 24 hr.	Seepage Per sq. ft. water surface per 24 hr.	Seepage Per sq. ft. wetted surface per 24 hr.
				Minutes	Feet	Feet	Feet	Feet	Feet	Feet	Cu. Ft.	Cu. Ft.
College East Farm, Fort Collins, Colorado, E-W Lateral—2½-inch concrete lining.												
2	6/11/46	10:55A	12:55P	120	296.1	0.867	2.40	3.17	0.780	0.016	0.764	0.578
	6/11/46	12:55P	2:55P	120		.810	2.35	3.05	.576	.016	.560	.432
	6/11/46	2:55P	4:55P	120		.765	2.29	2.94	.528	.016	.512	.399
	6/11/46	4:55P	7:55P	180		.716	2.23	2.83	.424	.016	.408	.321
	6/11/46	7:55P	8:55A	780		.553	2.05	2.46	.506	.026	.480	.400
	6/12/46	8:55A	10:55A	120		.398	1.84	2.05	.456	.031	.425	.381
College East Farm, Fort Collins, Colorado, E-W Lateral—3-inch concrete lining.												
2	6/11/46	10:59A	12:59P	120	307.2	0.870	2.46	3.20	0.432	0.016	0.416	0.320
	6/11/46	12:59P	2:59P	120		.840	2.42	3.13	.288	.016	.272	.210
	6/11/46	2:59P	4:59P	120		.815	2.40	3.07	.300	.016	.284	.222
	6/11/46	4:59P	7:59P	180		.790	2.37	3.02	.192	.016	.176	.138
	6/11/46	7:59P	8:59A	780		.694	2.26	2.80	.310	.026	.284	.229
	6/12/46	8:59A	10:59A	120		.599	2.13	2.57	.264	.031	.233	.193
College East Farm, Fort Collins, Colorado, N-S Lateral—2-inch oil lining.												
2	6/11/46	11:03A	12:03P	60	136.1	0.588	3.50	3.76	4.11	0.02	4.09	3.81
	6/11/46	12:03P	1:03P	60		.436	3.01	3.19	3.22	.02	3.20	3.02
	6/11/46	1:03P	2:03P	60		.310	2.61	2.72	2.83	.02	2.81	2.70
3	6/11/46	3:17P	4:17P	60		0.432	2.99	3.16	2.59	0.02	2.57	2.43
	6/11/46	4:17P	5:17P	60		.328	2.63	2.76	2.40	.02	2.38	2.27
College East Farm, Fort Collins, Colorado, N-S Lateral—3-inch bentonite lining.												
2	6/11/46	11:05A	12:05P	60	63.0	0.461	3.19	3.39	3.41	0.02	3.39	3.19
	6/11/46	12:05P	1:05P	60		.318	2.46	2.58	3.43	.02	3.41	3.25
3	6/11/46	3:18P	4:18P	60		0.354	2.67	2.80	2.62	0.02	2.60	2.48
	6/11/46	4:18P	5:18P	60		.242	2.08	2.16	2.81	.02	2.79	2.69

Figure 11.—Pool in Lateral No. 2 North, East Contra Costa Irrigation District, Brentwood, California. Condition of banks is also shown.

from the original cross-sections and were no longer uniform. (See figure 11.) The cross-sections taken indicated, however, a similarity of section with bottom widths of from 5 to 6 feet and side slopes of about 1½ to 1, except Lateral 3 South which had a bottom width of 3½ feet.

The portions of the laterals tested in each case extended from the headgate in the main canal to the check or dam at the lower end of the pool at the distance given in table 8. There were no diversions from the lateral during the time the observations were being made and all gates were inspected to see that there were no leaks. Where the seepage loss was small, observations were made on a single filling of the pool, but where the losses were high several series of observations were made to disclose whether the seepage rate decreased after the soil had become thoroughly wetted. The results are set out in table 8.

The outstanding result of the observations was the wide differences in the seepage rate from the different laterals. It is apparent also that puddling had little effect on the seepage rate from Lateral 5 South because the loss was greater than that from either Lateral No. 3 South or No. 2 North, which were in similar soil and untreated. This phenomenon was attributed to the fact that these soils cracked badly when dried and consequently the effect of puddling was temporary. Furthermore, these cracks persisted in the subsoil and resulted in a high percentage of voids. While the project lands were being brought under irrigation, streams of several second-feet would frequently disappear into holes in a field. All the water was taken up by the voids in the subsoil and the flow would continue until holes were plugged by dynamiting. No doubt the cracks in the lateral bed did not seal completely when water was turned in, and as a result the cracks led to the more porous soil below, which provided an easy passage for the water.

Lateral No. 6 North was excavated in a heavy clay soil which contained sand and gravel. A very low rate of loss was found for the lateral. In fact, the rate was less than that from some concrete-lined channels. This lateral had previously been puddled, but a conclusion that the low rate was caused by puddling is debatable in view of the small effect from this treatment previously noted. In order for puddling to be effective in soils of this character, the laterals would probably have to be kept wet continuously.

Lateral No. 3 North, see figure 12, was excavated in fine blow sand in 1922 and at the time observations were made, had never been used. This soil is of considerable depth and does not contain hardpan. The pool was formed by a sand dam in each end of the section chosen and was filled by opening the headgate in the pipeline under the bed of the lateral at the north end of the pool.

Because of the very high rate of seepage, the drop in the pool was read at approximately 15-minute intervals. Observations were continued over a 7-day period by refilling the pool whenever most of the water had disappeared. A summary of the results of several representative series is given in table 8. Between series 7 and 8 an attempt was made to seal the pores in the sand by dumping several yards of clay into the pool at the inlet while the pool was being filled. This method did not prove to be effective because the clay did not have enough time to become thoroughly mixed with the water and consequently most of it was deposited within 50 feet of the inlet. The observations of series 8, which were made the next day, indicate that the seepage was reduced slightly but much more clay would have to be added to be really effective. There was a definite reduction in seepage with depth and also with the number of times the pool had been refilled.

Figure 12.—Pool in lateral No. 3 North, East Contra Costa Irrigation District, Brentwood, California.

Observations were also made on two concrete-lined laterals of the East Contra Costa Irrigation District. Lateral No. 6 South is on one of the ridges extending southward from the main canal on the east side of Marsh Creek. The soil here is lighter textured than in the surrounding areas. The concrete lining of the lateral, placed in 1918, is 2 inches thick and was in good condition at the time of the tests, but was distinctly porous. Lateral 7 North was excavated in a heavy clay or adobe soil and was lined in 1915 with 2 inches of concrete which was in fair condition in 1923 when the seepage tests were made. Lateral 6 South had a bed width of 4 feet, side slopes of approximately 1½ to 1 and a depth of 3 feet. Lateral 7 North had the same general shape but was only 2.5 feet deep. The seepage loss from Lateral No. 6 South (table 8), was found to be unusually high for this type of lining. Observations were made on 3 consecutive days and the pool was refilled each day, but there was no appreciable reduction in the seepage rate as the underlying soil became saturated. It is believed that the porous concrete used in the lining and the characteristics of the soil in which the lateral was excavated were the principal reasons for the high rate of seepage which reached a maximum of over 3 cubic feet per square foot of wetted surface per 24 hours at the highest stage of the lateral. These observations show a definite relation between seepage and depth.

Although the lining of Lateral No. 7 North did not seem to be superior to that in Lateral No. 6 South, the seepage was approximately one-tenth as much (table 8), and this difference was probably due largely to the heavier soil in which Lateral No. 7 North was excavated. However, the seepage loss was much greater than that from the basins of the main canal. The tests on Lateral No. 7 North covered a 5-day period and although the seepage rate decreased with the lowering of the water surface, it is not clear whether the decrease was due to the reduction in head or the swelling of the soil under the concrete.

TURLOCK IRRIGATION DISTRICT LATERAL

Two series of observations were made on Lateral No. 5½ of the Turlock Irrigation District, Turlock, California, in October 1923. The portion of the lateral under test was lined the year previously with 2 inches of concrete laid without expansion joints. At the time of the tests the lining was in perfect condition. (See figure 13.) The soil in which the lateral was formed is very sandy. When the lining was constructed, the size of the lateral was reduced by pushing in the sides and excavating a new channel in the filled material after it had been compacted by soaking with water. Considerable leakage occurred through gates and checks during the tests. These losses were measured but since they exceeded the seepage (table 8), erratic results

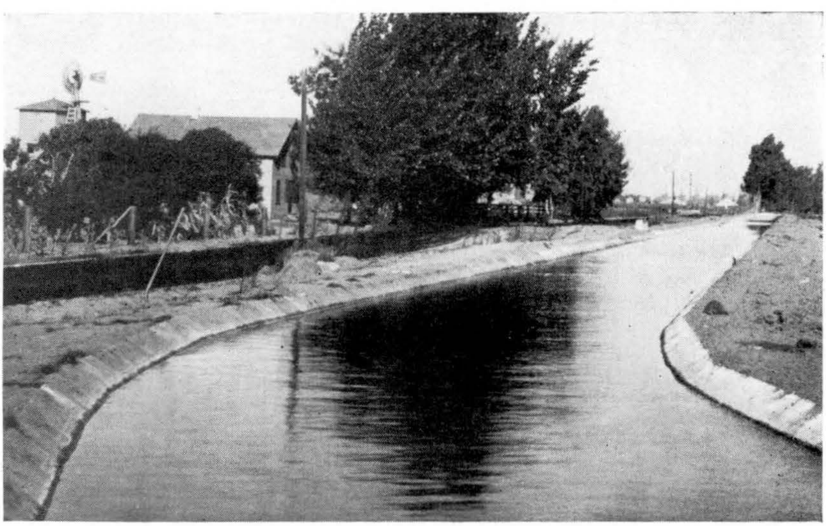

Figure 13.—Pool in Lateral 5½, Turlock Irrigation District, Turlock, California.

were obtained in the seepage measurements. For this reason too great weight should not be given to the results, but they are believed to be fairly reliable because they are comparable to those obtained from the basins of the Brentwood Main Canal.

After more than 20 years of service the lining of this lateral was still in good condition according to a report received from the Chief Engineer of the District. The only trouble experienced was the erosion of the banks resulting from recent use of the adjoining land as a pasture for cattle. Owing to the sandy soil the banks had been tramped down to the extent that a portion of the lining was unsupported and consequently there was a slight leakage visible through cracks in this portion. The plan was to rebuild the banks with earth to support the lining and to fence the lateral to keep the cattle out.

FRESNO IRRIGATION DISTRICT CANAL

After completing a series of current-meter observations to determine the seepage from the Houghton Canal of the Fresno Irrigation District at Fresno, California (page 88 and table 9), a pool was formed in the canal to check the measurements by noting the drop in the water surface. Figure 14 shows a portion of the pool. The east end of the pool was formed by the check at the High Ditch diversion and the west end by the check at West Lawn Avenue. The bottom width of the section under test was about 14 feet and the sides were eroded to approximately natural slopes. Because of the grade of the canal the upper end of the pool was quite shallow. The canal is in sandy soil and in some places the bed is in hardpan. Some difficulty was experienced in stopping the leaks through the headgates and checks and some inflow

Figure 14.—Pool in Houghton Canal at Fresno, California.

occurred at the upper check. The inflow exceeded the losses through the gates, which accounted for the gain from the source. (See table 8.) Observations were made twice daily through a period of 3 days. The seepage losses found were quite consistent and in general decreased with the depth, but they were slightly greater than those obtained previously from the continuous record of inflow and outflow based on current-meter measurements.

GAGE CANAL

The current-meter measurements of seepage on the Gage Canal, at Riverside, California (page 79 and table 9), were checked by noting the drop in a pool comprising the terminal basin and a portion of the lower end of the canal. Here, as elsewhere along the canal, the concrete lining was 3/4-inch thick. The soil is a heavy black clay or adobe. During the 7-day period of the test there were no diversions for irrigation but there was a small amount drawn out for domestic purposes. This withdrawal was measured once during the period and was assumed to be constant. The total area of the canal and basin was 2½ acres and the depth of water ranged from 0.4 foot at the upper end of the pool to 3.11 feet at the lower end. Because of the irregular shape of the pool these seepage measurements are not reported in table 8. The average seepage loss for the period was 0.028 cubic foot per square foot of wetted surface per 24 hours. For the two series of current-meter measurements the losses for the entire canal were respectively 0.037 and 0.047 cubic foot per square foot per 24 hours. Although the percentage difference was considerable, the actual losses agreed as well as could be expected in view of their small magnitude.

At the time the tests were made the lining was 30 years old, and it is now (1947) 50 years since the lining was laid. According to a recent report from the company's engineer the old lining is still in use and annual repairs, consisting mainly of patching places where roots or erosion have damaged the lining, require from 3 to 5 sacks of cement per mile of canal. This is an outstanding example of the durability of concrete linings.

COLORADO FARM LATERALS

Observations were made in March 1944. on the outlet channel for a farm pumping plant near Gilcrest, Colorado, to determine the losses. Two pools were constructed in the channel by building an earth dam about midway between two checks and then pumping the basins full of water. Since the ditch was in very sandy soil and had not been in use since the previous year, the first filling of the pools was allowed to soak away so as to wet the ground thoroughly. The pools were then refilled and observations on the rate of drop were taken at 5-minute intervals until less than 1-foot depth of water remained. Because of the sandy soil, the ditch had lost its original shape and was badly eroded near the checks. The average bed width was 3 feet and the average width at the high-water line was about 6.5 feet. The water in the pools was from 1 to 3 feet deep. There were no leaks and since the seepage rate was very high, no correction was made for evaporation.

The results of the observations are given in table 8. Both pools showed a high rate of loss and that from the north pool, which was deeper, consistently exceeded that from the south pool. However, the rate from each pool throughout the period of observations was apparently independent of the depth. For this reason it must be assumed that the higher rate from the north pool was due to a difference in the soil rather than the depth. The losses here measured are comparable to those from Lateral No. 3 North at Brentwood, California, which is also in very sandy soil.

Several series of observations were made in 1945 and 1946 to determine the seepage losses from a lateral on the College East Farm at Fort Collins, Colorado, before and after it was lined. The lateral was excavated in earth classified as Fort Collins loam, a fairly heavy soil. A portion of the lateral is shown in figure 15. Two pools were formed in the lateral by building an earth dam at each end of the section to be tested. The pools were filled with city water through a fire hose attached to a nearby hydrant. The drop in the water surface was measured on staff gages reading to 0.01 foot, located at the ends of the pools. The seepage observations were made in March, and since the lateral had not been used since the previous September, the sections were saturated by allowing the first fillings of the pools to seep away before measurements of the losses were begun. The results of the seepage measurements are given in table 8. The losses in both pools were nearly the same and amounted to about 5 cubic feet per square foot of wetted perimeter per 24 hours.

Figure 15—Farm lateral, College East Farm, Fort Collins, Colorado, before being lined.

After completion of this series of seepage measurements, the lateral was filled with earth and compacted with a sheepsfoot roller. A new lateral was excavated in the filled material to the dimensions and grade required for the lined sections. These lined sections consisted of 300 feet each of 2, 2½ and 3-inch concrete linings, 65 feet of 3-inch bentonite and earth lining, and 136 feet of 2-inch oil and earth lining. The concrete-lined sections were approximately semi-circular in shape with the top 30° of the arc on each side tangent to the curve. (See figure 16.) All the concrete-lined sections

underlying other portions of the lateral. When the seepage observations were repeated a year later, the losses showed the same tendency. (See table

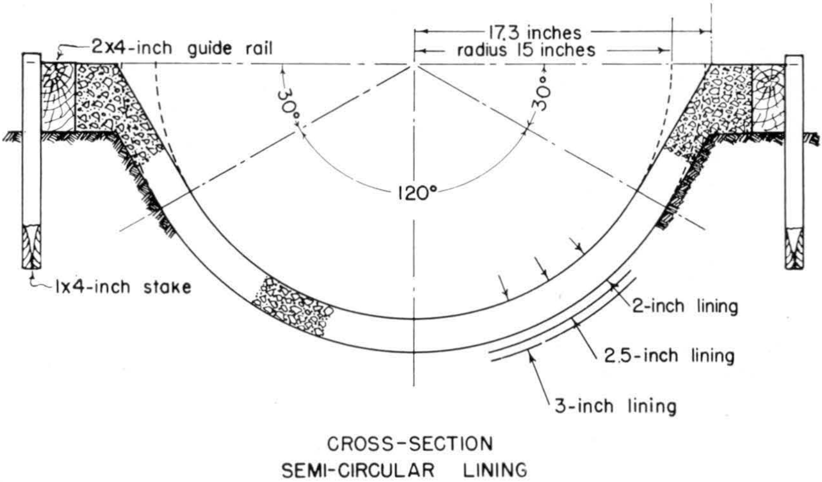

Figure 16.—Design of semi-circular concrete lining for farm lateral, College East Farm, Fort Collins, Colorado.

were built with a radius of 15 inches. The lining was placed without forms by means of a rotating trowel attached to a pipe on the lateral axis. A 1-to-5 mixture of Portland cement and screened gravel was used. Figure 17 shows the finished lining. The oil and bentonite linings were made trapezoidal in shape, with a 2-foot bottom and 1½ to 1 side slopes. About 10 pounds of bentonite were used per square yard of the bentonite lining, and 1¾ gallons of oil per square yard of the oil lining. The soil for these mixtures was taken from the material excavated from the lateral. The material for the oil

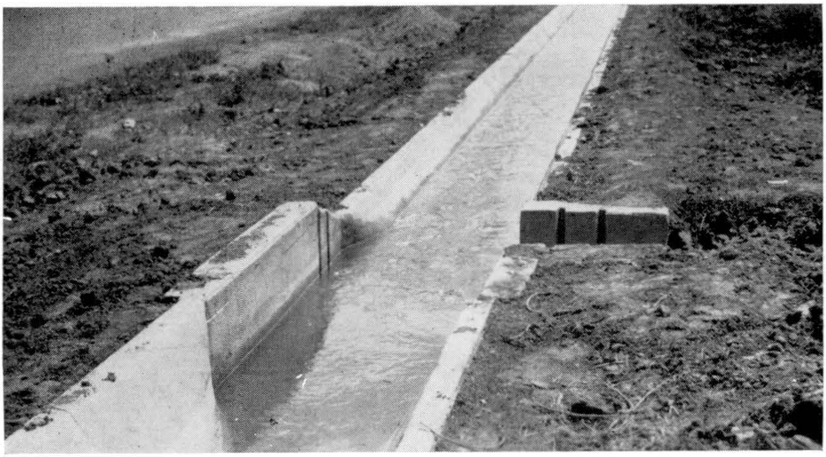

Figure 17.—Finished section of concrete lining of farm lateral, College East Farm, Fort Collins, Colorado.

Figure 18.—Oil lining of farm lateral, College East Farm, Fort Collins, Colorado.

and the bentonite linings was mixed in a concrete mixer and after it had been shoveled into place, it was compacted by hand as shown in figure 18.

These linings were applied during June 1945 and they were tested for seepage in September 1945 and again in June 1946. Pools were made in the concrete-lined sections by putting boards in the grooves in the turnout structures and filling the space between the boards with earth. Earth dams were built in the oil and bentonite-lined sections between boards, which were cut to fit the sections. The pools were filled with water from the ditch that supplied the lateral. A point gage (figure 19) was used to measure the drop in the water surface. All sections were allowed to become saturated before observations on the seepage losses were commenced.

Figure 19.—Point gage being used to measure drop in water surface of pool in concrete-lined section of farm lateral, College East Farm, Fort Collins, Colorado.

The results of the observations are set out in table 8. All types of lining were effective in reducing seepage and the concrete lining produced the greatest reduction. The effectiveness of the concrete linings did not increase with the thickness of the lining; in fact, the 2-inch lining was apparently superior to the $2\frac{1}{2}$-inch lining and equal to the 3-inch lining. Variations in the soil underlying the concrete do not account for the differences because the soil on which the 2-inch lining was placed contains more sand than that underlying other portions of the lateral. When the seepage observations were repeated a year later, the losses showed the same tendency. (See table

8.) At that time, however, all the losses through the concrete linings had decreased, whereas a definite increase in seepage was noted in the oil and bentonite linings. Although the concrete lining was carefully placed, under controlled conditions, nevertheless, the losses were higher than those shown for some of the other linings reported in table 8.

SUMMARY OF RESULTS OF SEEPAGE MEASUREMENTS

A study of the results of these tests (table 8), shows that the losses measured by the pool method are more consistent than those determined by current-meter gagings. (See table 9.) In fact, it would be impossible to detect losses as small as those from good concrete-lined canals by the current-meter method. These data show also, that some unlined canals lose less water than some lined canals, but where the losses from concrete-lined canals were high, the concrete was porous or in poor condition. In some cases, however, the reason for the high losses was not apparent. Another important fact disclosed by these tests is that the soil classification alone is not a satisfactory guide in estimating seepage losses. It would seem that the only safe procedure is to measure the actual losses, and to do this the pool method is most satisfactory wherever the conditions are such that it can be used.

Inflow-Outflow Measurements

Seepage determinations were made by means of current-meter measurements on both lined and unlined irrigation canals and laterals in various types of soils in California. A description is given of each canal or lateral tested.

After the canal in which the seepage was to be measured had been chosen, sections were located which fulfilled as many as possible of the requirements previously outlined. (See page 11.) If conditions seemed to warrant, water-stage recorders were installed at each end of the section; where conditions apparently did not justify installation at first, subsequent developments usually indicated the desirability of the register record. From the register records or the estimated velocity in the canal, the proper time interval between the gagings at the upper and lower stations was computed. Several gage readings were taken with a plumb-bob hook gage before the gaging was started to determine whether the stage was changing rapidly. Where recorders were installed the record showed what was occurring. As soon as the stage was fairly constant the depths of water at the gaging station were measured. Soundings were made at 5-foot intervals in large canals, 2-foot intervals in medium-sized canals, and 1 or ½-foot intervals in small canals. A thin scale was used in measuring the depths in earth channels and an engineer's level was used in lined sections. In the latter case the depths were determined from the profile across the canal and the elevation of the water surface.

After the depths had been determined, the velocity measurements were made with the current meter, a Price cup meter and a Hoff propeller meter being used. Most of the measurements were made by vertical integration or the 2-and-8-tenths method, but the multiple-point method was also employed. A stop watch was used to take the time. The gage height was noted at the beginning and end of the gaging and if necessary at intermediate points to make it possible to compute the true area of the section for the period while the test was being made.

Two or more measurements were usually made at each gaging station, different meters or different methods being used. After the proper time interval, as previously determined, the same procedure was followed at the lower station.

If there were any diversions or leaks in the section of the canal under test they were measured during the interval between the gagings or immediately after the gagings at the lower station. Large diversions were measured with the current meter, others were measured with rectangular or triangular weirs and if very small, with a calibrated can. The evaporation loss from the water surface of the canal was measured in a circular pan, 8 inches in diameter, suspended in the water. A special hook-gage reading to 0.001 foot was used to measure the loss. The temperatures of the air and the water were also taken.

The length of the section of canal under test was ascertained from the original survey notes, from maps, or by measurement in the field. Cross-sections of the canal were taken at both ends of the section and at intermediate points where changes occurred. The area of the water surface and the wetted area were computed from plots of these data. When unlined canals included lined sections, flumes, or lined tunnels, these were excluded.

To determine the seepage losses from the spot measurements with current meters, the average outflow (corrected for diversions, leaks and evaporation), was subtracted from the inflow. The loss per mile[7] and per square foot of wetted surface were computed

[7] The true rate of loss per mile is:

$$i = 1 - \left[\frac{Q_2}{Q_1} \right]^{1/n}$$

where i = rate of loss per mile
n = number of miles
Q_2 = outflow
Q_1 = inflow

However, if there are diversions even this law does not hold; therefore the data presented were computed on the basis of a straight line variation.

from the net loss and the length and wetted area of the canal. When register records were available the total inflow and the outflow were computed from the gage-height records, and the discharge curves for the upper and lower stations plotted from the current-meter measurements and the corresponding gage heights. The total loss due to diversions, leaks and evaporation during the period was added to the outflow. If the gage height at the end of the period differed from that at the beginning, a correction was made for channel storage. The net difference was the total volume of water lost during the period.

LINDSAY-STRATHMORE CANAL

Lindsay-Strathmore Lowline Canal, Lindsay, California, was lined with gunite in 1915. No change was made in the canal section, the gunite to a thickness of ½ inch being shot directly on the old surface. No expansion

Figure 20.—Lindsay-Strathmore Lowline canal with gunite lining when tested for seepage losses.

joints were provided, and except for the shrinkage cracks, the lining was in excellent condition, as shown in figure 20. The canal section is irregular and on the average the bottom is about 6 feet wide and the side slopes 1¼ to 1. The depth is about 2½ feet. The canal line follows the base of the Sierra Nevada foothills and throughout its length it is excavated in Porterville clay loam adobe, a heavy soil which cracks badly when dry and puddles easily when wet. No hardpan was found, but the bedrock lies very near the surface in some places.

The tests on the canal were made during the period from September 22 to September 26, 1922. At that time the canal contained considerable moss although it was being treated with copper sulphate at several points. This moss interfered with the meter in making measurements and also caused considerable fluctuation at the lower station owing to the clogging of the screens used in removing the moss from the water at the pumping station which raises the water to the bench flume. An attempt was made to reduce the fluctuation by gradually changing the screens so that at no time were all the screens free or considerably clogged, but on account of the difficulty in regulating the screens this procedure was not entirely successful.

The discharge measurements were all made with a small Price meter for determining the velocity and an engineer's level and hook gage for determining the area. Vertical integration, 2-and-8-tenths depth and multiple-point were methods used in measuring the velocity. In all, 15 gagings were made; 9 by the vertical integration method, 4 by the 2-and-8-tenths and 2 by the multiple-point method as shown in table 9. Gagings numbered 14 and 14′, 17 and 17′, and 18 and 18′ were check measurements, the first two being a comparison of two measurements by the same method, while the others are comparisons of different methods. Gagings 14 and 14′, both vertical integrations, differed from each other by less than 1/5 of 1 percent, while 17 and 17′, and 18 and 18′, differed from each other by about 1½ percent in each case and in both cases the vertical integration gave a smaller discharge than the 2-and-8-tenths method. Eight seepage determinations were made and of these four showed gains and four showed losses, but the total gains slightly exceeded the losses, as shown by the mean value in table 9. A gain in this section of canal hardly seemed possible and the results can only be explained by the limitations of the method of measurement.

Alta Irrigation District

Alta Main Canal

Alta Irrigation District Main Canal, Dinuba, California, was constructed in 1882. The section chosen for making the seepage measurements was built 100 feet wide on the bottom and was designed for a water depth of 5 feet and a capacity of 1,200 cubic feet per second. Since that time the section had changed considerably as a result of erosion and the trampling of stock. The fall was about 18 inches to the mile. The portion of the canal measured extends from the gaging station below the headgate to the highway at Dunegans Gap. From the headgate for about one-quarter of a mile the canal runs through a deep cut in the bottom land along Kings River in soil classified as Hanford fine sandy loam. As the canal climbs to the higher ground it cuts for a short distance through Holland coarse sandy loam and from there on skirts the foothills, climbing rapidly up the slope, which is quite steep in this region. The soil here is Porterville clay adobe, locally known as "dry bog" land. It is a heavy soil free from hardpan, but close to bedrock at some points. In the river bottom the canal cuts through occasional gravel beds, but when the water was out of the canal the leakage through the headgate did not seep away readily. Figure 21 shows a section of the canal.

Two short sections of concrete linings occurred in the portion of the canal measured, which were excluded in making the computations for seepage, but which provided nearly ideal sections for making the current-

Table 9. *Seepage Measurements (Inflow-Outflow Method).*

Test number	Upper station: Gaging number	Upper station: Location	Upper station: Discharge	Upper station: Method	Average Temperature, F: Air	Average Temperature, F: Water	Length of section	Canal depth	Width of water surface	Length of wetted perimeter	Velocity in canal per sec.	Diversions	Leaks	Evaporation
			c.f.s.		deg.		mi.	ft.	ft.	ft.	ft.	c.f.s.	c.f.s.	c.f.s.
Lindsay-Strathmore, concrete lined, clay loam adobe soil.														
1	9	Head	41.44	Int.	82	65	5.378	2.3	11.3	13.0	2.1	—	—	.043
2	11	Head	40.55	.2 & .8	82	65	5.378	2.4	11.4	13.2	2.0	—	—	.044
3	13	Head	41.45	Int.	82	65	5.378	2.4	11.4	13.2	2.0	—	—	.044
4	13	Head	41.45	Int.	82	65	5.378	2.4	11.4	13.2	2.0	—	—	.044
5	15	Head	41.33	Int.	82	65	5.378	2.4	11.4	13.1	2.0	—	—	.044
6	17	Head	41.81	.2 & .8	82	65	5.378	2.4	11.4	13.1	2.0	—	—	.044
7	17′	Head	41.26	Int.	82	65	5.378	2.4	11.4	13.1	2.0	—	—	.044
8	19	Head	40.65	M.P.	82	65	5.378	2.4	11.4	13.2	1.9	—	—	.044
		Mean 1	41.24											
Alta Main, earth section, clay adobe and some fine sandy loam.														
9	21	Head	116.27	Int.	71	66[1]	1.346	1.7	80.2	81.1	1.0	3.05	—	.117
10	23	Head	122.56	.6	71	66	1.346	1.7	80.3	81.1	1.0	2.68	—	.117
11	25	First lining	118.17	.6	71	66	.261	1.7	83.5	84.4	0.9	—	—	.024
12	27	Head	122.13	.6	71	66	.484	0.8	95.0	96.4	1.6	—	—	.045
13	27	Head	122.13	.6	71	66	.935	1.7	83.3	84.3	1.0	1.97	—	.076
14	27	Head	122.13	.6	71	66	1.346	1.7	80.3	81.1	1.0	1.97	—	.117
15	28	First lining	117.79	.6	71	66	.451	2.0	77.5	78.3	0.9	1.97	—	.034
16	28	First lining	117.79	.6	71	66	.862	1.9	76.1	77.0	1.0	1.97	—	.075
17	29	Blow Campbell Lat.	111.77	.6	71	66	.411	2.0	70.9	71.7	1.0	—	—	.039
18	31	Head	100.34	Int.	71	66	.484	0.7	94.8	96.1	1.5	—	—	.045
19	31	Head	100.34	Int.	71	66	.745	1.3	88.8	89.8	1.0	—	—	.067
20	31	Head	100.34	Int.	71	66	.935	1.6	82.9	83.8	0.9	1.88	—	.075
21	31	Head	100.34	Int.	71	66	.935	1.6	82.9	83.8	0.8	1.88	—	.075
22	31	Head	100.34	Int.	71	66	1.346	1.6	79.8	80.6	0.9	1.88	—	.116
23	32	First lining	92.83	Int.	71	66	.261	1.6	83.2	84.1	0.8	—	—	.024
24	32	First lining	92.83	Int.	71	66	.451	1.9	77.1	77.9	0.7	1.88	—	.034
25	32	First lining	92.83	Int.	71	66	.451	1.9	77.1	77.9	0.7	1.88	—	.034
26	32	First lining	92.83	Int.	71	66	.862	1.8	75.7	76.5	0.8	1.88	—	.074
27	34	Above Campbell Lat.	96.37	Int.	71	66	.190	2.5	71.0	71.4	0.6	1.88	—	.011
28	34	Above Campbell Lat.	96.37	Int.	71	66	.190	2.5	71.0	71.4	0.6	1.88	—	.011
29	34	Above Campbell Lat.	96.37	Int.	71	66	.601	2.0	72.3	72.9	0.7	1.88	—	.050
30	35	Below Campbell Lat.	91.04	Int.	71	66	.411	1.9	70.7	71.4	0.7	—	—	.039
31	36	Below Campbell Lat.	94.22	.2 & .8	71	66	.411	1.9	70.7	71.4	0.8	—	—	.039
		Mean 1	115.32											
		Mean 2	111.24											
		Mean 3	105.31											
		Mean 4	105.50											
		Mean 5	107.60											
		Mean 6	101.15											
		Mean 7	99.01											
		Mean 8	96.37											
Alta Main, earth section, first half mile fine sandy loam; rest clay adobe.														
32	159	Head	896.79	.2 & .8	77	55	1.286	4.1	92.3	94.1	2.8	9.37	—	.215[2]
33	162	Head	715.24	.2 & .8	77	55	1.286	3.7	90.0	91.6	2.6	8.69	—	.209
34	171	Head	620.78	.2 & .8	77	55	1.261	3.4	88.6	90.1	2.4	10.37	—	.204
		Mean 9	620.78											
			706.4	Period 4 p.m. 6/17/23 to 6 p.m. 6/18/23										
Alta East Branch, earth section, sandy loam.														
35	165	Orosi Lateral	39.01	.2 & .8	77	66	.658	2.3	25.8	26.9	.80	—	—	.021
36	167	Orosi Lateral	39.45	.2 & .8	77	66	.658	2.2	25.7	26.8	.83	—	—	.021
37	169	Orosi Lateral	61.09	.2 & .8	77	66	.658	2.8	27.4	28.8	.97	.65	—	.023
38	174	Orosi Lateral	50.71	.2 & .8	77	66	.658	2.6	26.8	28.0	.88	1.35	—	.022
39	177	Orosi Lateral	57.58	Int.	77	66	.658	2.7	27.1	28.5	1.0	1.84	—	.023
40	180	Orosi Lateral	72.20	Int.	77	66	.658	3.0	28.0	29.6	1.0	.80	—	.023
		Mean 1	53.34											

(1) Evaporation tank temperatures. Test numbers 9 to 31, incl.
(2) East Branch record. Test numbers 32 to 34, incl.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Lower station: Gaging number	Lower station: Location	Lower station: Discharge	Lower station: Method	Lower discharge, evaporation, diversions & leaks	Seepage loss: In section	Seepage loss: Per mile	Seepage loss: Per mile	Seepage loss: Per sq. ft. wetted area in 24 hr.	Classification number
			c.f.s.		c.f.s.	c.f.s.	c.f.s.	%	cu.ft.	
Lindsay-Strathmore, concrete lined, clay loam adobe soil.										
1	10	Lower	41.21	Int.	41.25	.19	.0353	.085	.044	1
2	12	Lower	41.03	.2 & .8	41.07	–.52	–.0967	–.238	–.120	1
3	14	Lower	41.63	Int.	41.67	–.22	–.0409	–.099	–.051	1
4	14′	Lower	41.55	Int.	41.59	–.14	–.0260	–.063	–.032	1
5	16	Lower	41.92	Int.	41.96	–.63	–.1171	–.283	–.146	1
6	18	Lower	41.56	.2 & .8	41.60	.21	.0390	.093	.049	1
7	18′	Lower	40.94	Int.	40.98	.28	.0521	.126	.065	1
8	20	Lower	39.31	M.P.	39.35	.30	.0558	.137	.069	1
							Mean 1	–.0299	–.0153	
Alta Main, earth section, clay adobe and some fine sandy loam.										
9	22	Second lining.	105.46	Int.	108.63	7.64	5.68	4.88	1.15	1
10	24	Second lining.	108.33	.6	111.13	11.43	8.49	6.93	1.71	1
11	26	Above Campbell Lat.	111.68	.6	111.70	6.47	24.79	20.98	4.81	4
12	28	First lining.	117.79	.6	117.83	4.30	8.88	7.25	1.51	2
13	29	Below Campbell Lat.	111.77	.6	113.82	8.31	8.89	7.27	1.73	5
14	30	Second lining.	106.34	.6	108.43	13.70	10.18	8.34	2.05	1
15	29	Below Campbell Lat.	111.77	.6	113.77	4.02	8.91	7.56	1.86	6
16	30	Second lining.	106.34	.6	108.39	9.40	10.90	9.25	2.32	3
17	30	Second iining.	106.34	.6	106.38	5.39	13.11	11.73	2.99	7
18	32	First lining.	92.83	Int.	92.87	7.47	15.43	15.38	2.63	2
19	34	Above Campbell Lat.	96.37	Int.	96.44	3.90	5.23	5.21	.95	
20	35	Below Campbell Lat.	91.04	Int.	93.00	7.34	7.85	7.82	1.53	5
21	36	Below Campbell Lat.	94.22	.2 & .8	96.18	4.16	4.45	4.44	.87	5
22	37	Second lining.	87.13	Int.	89.13	11.21	8.33	8.30	1.69	1
23	34	Above Campbell Lat.	96.37	Int.	96.39	–3.56	–13.64	–14.69	–2.65	4
24	35	Below Campbell Lat.	91.04	Int.	92.95	–0.12	–0.27	–0.291	–0.057	6
25	36	Below Campbell Lat.	94.22	.2 & .8	96.13	–3.30	–7.32	–7.89	–1.54	6
26	37	Second lining.	87.13	Int.	89.08	3.75	4.35	4.69	0.93	3
27	35	Below Campbell Lat.	91.04	Int.	92.93	3.44	18.11	18.79	4.15	8
28	36	Below Campbell Lat.	94.22	.2 & .8	96.11	0.26	1.37	1.42	0.31	8
29	37	Second lining.	87.13	Int.	89.06	7.31	12.16	12.62	2.73	
30	37	Second lining.	87.13	Int.	87.17	3.87	9.42	10.35	2.16	7
31	37	Second lining	87.13	Int.	87.17	7.05	17.15	18.20	3.93	7
							Mean 1	7.08	1.65	
							Mean 2	10.93	2.07	
							Mean 3	7.24	1.63	
							Mean 4	5.28	1.08	
							Mean 5	6.56	1.38	
							Mean 6	.43	.09	
							Mean 7	13.36	3.04	
							Mean 8	10.11	2.23	
Alta Main, earth section, first half mile fine sandy loam; rest clay adobe.										
32	160	H. W. Bridge	930.25	.2 & .8	939.84	–43.05	–33.48	–3.73	–5.82	
33	163	H. W. Bridge	736.41	.2 & .8	745.31	–30.07	–23.38	–3.27	–4.18	
34	172	Second lining.	583.74	.2 & .8	594.31	26.47	20.99	3.38	3.81	9
							Mean 9	3.38	3.81	
			Period 4 p.m. 6/17/23 to 6 p.m. 6/18/23					–.088	–.077	
Alta East Branch, earth section, sandy loam.										
35	166	Sand Creek.	38.94	.2 & .8	38.96	.05	.076	.195	.046	1
36	168	Sand Creek.	40.10	.2 & .8	40.12	–.67	–1.018	–2.58	–.621	1
37	170	Sand Creek.	61.01	.2 & .8	61.68	–.59	–.897	–1.47	–.510	1
38	175	Sand Creek.	50.89	.2 & .8	52.26	–1.55	–2.356	–4.65	–1.38	1
39	178	Sand Creek.	58.16	Int.	60.02	–2.44	–3.708	–6.44	–2.13	1
40	181	Sand Creek.	71.39	Int.	72.21	–.01	–.152	–.210	–.084	1
							Mean 1	–2.517	–.782	

Note: The negative signs indicate gains.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Upper station: Gaging number	Upper station: Location	Upper station: Discharge	Upper station: Method	Average Temperature, F: Air	Average Temperature, F: Water	Length of section	Canal depth	Width of water surface	Length of wetted perimeter	Velocity in canal per sec.	Diversions	Leaks	Evaporation
			c.f.s.		deg.		mi.	ft.	ft.	ft.	ft.	c.f.s.	c.f.s.	c.f.s.
		Gage, concrete, loams and sandy loams of various types.												
41														
42														
43	40	Upper Weir	25.72	Int.	67	53	11.68	1.9	8.4	10.4	1.9		.490[1]	.032
44	41	Upper Weir	26.13	Int.	67	53	11.68	1.9	8.4	10.4	1.9		.490	.032
45	44	Upper Weir	26.61	.2 & .8	67	53								
46	45	Upper Weir	26.64	.2 & .8	67	53								
47	48	Upper Weir	25.21	.2 & .8	67	53	11.68							
48	49	Upper Weir	24.72	.2 & .8	67	53	11.68							
49	52	Upper Weir	23.64	.2 & .8	67	53	11.68	1.9	8.4	10.4	1.7		.490	.032
50	53	Upper Weir	23.24	.2 & .8	67	53	11.68	1.9	8.4	10.4	1.7		.490	.032
51	56	Upper Weir	24.72	.2 & .8	67	53	11.68	2.0	8.5	10.5	1.7		.490	.033
52	57	Upper Weir	23.77	.2 & .8	67	53	11.68	2.0	8.5	10.5	1.7		.490	.033
53							11.68						.490	
54							11.68							
		Mean 1	24.54											
			24.9	Period from 2 p.m. 12/28/22 to 7 a.m. 12/29/22										
		Imperial West Side Main, earth section, sand, fine sand, very fine sand and clay and silty clay of several types.												
55	62	West drain	280.27	.2 & .8	65	52	14.69	5.5	32.8	37.1	1.8	2.49	.03	.368
56	65	West drain	276.05	.2 & .8	65	52	14.69	5.5	32.8	37.1		3.12	.48	.368
57	68	West drain	274.82	.2 & .8	65	52	14.69	5.5	32.8	37.1	1.8	5.68		.368
58	68	West drain	274.82	.2 & .8	65	52	14.69	5.5	32.8	37.1	1.8	5.68		.368
59	72	West drain	271.05	M.P.	65	52	14.69	5.5	32.8	37.1	1.8	4.61	.64	.368
60	75	West drain	280.88	.2 & .8	65	52	14.69	5.5	32.8	37.1	1.8	7.50	.10	.368
61	78	West drain	273.87	M.P.	65	52	14.69	5.5	32.8	37.1	1.8	7.64		.368
		Mean 1	275.95											
			277.5	Period from midnight 1/22/23 to midnight 1/25/23										
		Imperial Fillaree, earth section, fine sand of several types and silty clay.												
62	81	Head	6.15	Int.	65	52	3.17	.9	6.6	7.0	1.4			.016
63	83	Head	5.94	Int.	65	52	1.37	.9	7.3	7.8	1.1			.008
64	83	Head	5.94	Int.	65	52	3.17	.9	6.6	7.0	1.4			.016
65	84	First bridge	5.46	Int.	65	52	1.80	.8	5.9	6.2	1.7			.008
66	85′	Head	6.02	Int.	65	52	1.37	.9	7.3	7.8	1.1			.008
67	85′	Head	6.02	Int.	65	52	3.17	.9	6.6	7.0	1.4			.016
68	86	First bridge	5.34	Int.	65	52	1.80	.8	5.9	6.2	1.7			.008
		Mean 1	6.04											
		Mean 2	5.98											
		Mean 3	5.40											

(1) From record of Dec. 28, 1922. Test numbers 43, 44, and 49 to 52, incl.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Lower station				Lower discharge, evaporation, diversions & leaks	Seepage loss				Classification number
	Gaging number	Location	Discharge	Method		In section	Per mile	Per mile	Per sq. ft. wetted area in 24 hr.	
			c.f.s.		c.f.s.	c.f.s.	c.f.s.	%	cu.ft.	
		Gage, concrete, loams and sandy loams of various types.								
41	38	Lower Weir	18.23	Int.						
42	39	Lower Weir	18.14	Int.						
43	42	Lower Weir	24.66	Int.	25.18	.54	.046	.179	.072	1
44	43	Lower Weir	24.77	Int.	25.29	.84	.072	.276	.113	1
45	46	Below tunnel	26.70	.2 & .8						
46	47	Below tunnel	27.16	.2 & .8						
47	50									
48	51									
49	54	Lower Weir	22.91	.2 & .8	23.43	.21	.018	.076	.028	1
50	55	Lower Weir	22.45	.2 & .8	22.97	.27	.023	.099	.036	1
51	58	Lower Weir	23.90	.2 & .8	24.42	.30	.026	.105	.041	1
52	59	Lower Weir	23.74	.2 & .8	24.26	−.49	−.042	−.177	−.065	1
53	60	Lower Weir	26.27	.2 & .8						
54	61	Lower Weir	23.97	.2 & .8						
							Mean	.097	.037	
		Period from 2 p.m. 12/28/22 to 7 a.m. 12/29/22					.030	.120	.047	
		Imperial West Side Main, earth section, sand, fine sand, very fine sand and clay and silty clay of several types.								
55	64	Thompson's Crossing	269.24	.2 & .8	272.13	8.14	.554	.198	.244	1
56	67									
57	70	Thompson's Crossing	269.26	.2 & .8	275.31	−0.49	−.033	−.012	−.015	1
58	71	Thompson's Crossing	265.55	.2 & .8	271.60	3.22	.219	.080	.097	1
59	74	Thompson's Crossing	268.28	M.P.	273.90	−2.85	−.194	−.072	−.086	1
60	77	Thompson's Crossing	269.18	.2 & .8	277.15	3.73	.254	.090	.112	1
61	80	Thompson's Crossing	263.26	M.P.	271.27	2.60	.177	.065	.078	1
							Mean	.059	.072	
		Period from midnight 1/22/23 to midnight 1/25/23					.309	.111	.136	
		Imperial Fillaree, earth section, fine sand of several types and silty clay.								
62	82	Second bridge	5.38	Int.	5.40	0.75	.237	3.85	.554	1
63	84	First bridge	5.46	Int.	5.47	0.47	.343	5.77	.719	2
64	85	Second bridge	5.58	Int.	5.60	0.34	.107	1.80	.250	1
65	85	Second bridge	5.58	Int.	5.59	−0.13	−.072	−1.32	−.190	3
66	86	First bridge	5.34	Int.	5.35	0.67	.489	8.12	1.03	2
67	87	Second bridge	5.39	Int.	5.41	0.61	.192	3.19	.449	1
68	87	Second bridge	5.39	Int.	5.40	−.06	−.033	−0.62	−.087	3
							Mean 1	2.96	.418	
							Mean 2	6.96	.873	
							Mean 3	−.97	−.138	

Note: The negative signs indicate gains.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

	Upper station				Average Temperature, F									
Test number	Gaging number	Location	Discharge	Method	Air	Water	Length of section	Canal depth	Width of water surface	Length of wetted perimeter	Velocity in canal per sec.	Diversions	Leaks	Evaporation
			c.f.s.			deg.	mi.	ft.	ft.	ft.	ft.	c.f.s.	c.f.s.	c.f.s.
Merced Yosemite LeGrand, earth section; loams, gravelly loams, clay loams and clay adobe.														
69	88	Flume No. 1	25.69	Int.			6.801	1.4	20.3	21.2	.88		.026	.145[1]
70	89	Flume No. 1	26.57	Int.			6.801	1.5	20.3	21.4	.90		.026	.145
71	88	Flume No. 1	25.69	Int.			9.977	1.4	19.7	21.2	.84		.045	.211
72	89	Flume No. 1	26.57	Int.			9.977	1.5	20.0	21.4	.89		.045	.214
73	90	Flume No. 2	20.47	Int.			3.176	1.3	19.1	20.6	.80		.019	.068
74	91	Flume No. 2	21.56	Int.			3.176	1.4	19.4	21.1	.84		.019	.069
75	94	Flume No. 1	44.63	Int.			6.801	2.1	22.1	23.5	1.1		.039	.158
76	95	Flume No. 1	45.45	Int.			6.801	2.1	22.2	23.6	1.1		.039	.159
77	94	Flume No. 1	44.63	Int.			9.327	2.1	21.9	23.6	1.1		.070	.219
78	95	Flume No. 1	45.45	Int.			9.327	2.1	22.0	23.7	1.1		.070	.220
79	96	Flume No. 2	43.18	Int.			2.525	2.1	21.8	23.5	1.1		.031	.063
80	97	Flume No. 2	44.22	Int.			2.525	2.1	22.0	23.6	1.1		.031	.063
81	100	Flume No. 1	67.57	.2 & .8			6.801	2.5	23.4	25.1	1.3		.054	.167
82	101	Flume No. 1	68.61	.2 & .8			6.801	2.6	23.5	25.3	1.3		.054	.168
83	100	Flume No. 1	67.57	.2 & .8			9.327	2.6	23.3	25.4	1.2		.085	.233
84	101	Flume No. 1	68.61	.2 & .8			9.327	2.6	23.1	25.3	1.3		.085	.231
85	102	Flume No. 2	58.82	.2 & .8			2.525	2.4	22.2	24.8	1.2		.031	.064
86	103	Flume No. 2	60.36	.2 & .8			2.525	2.5	22.2	24.8	1.2		.031	.064
87	106	Flume No. 1	56.42	Int.			9.327	2.3	22.5	24.5	1.2		.081	.225
88	107	Flume No. 1	57.68	Int.			9.327	2.3	22.5	24.5	1.2		.081	.225
89	109	Flume No. 1	48.68	Int.			9.327	2.2	22.1	23.9	1.1		.075	.221
90	110	Flume No. 1	50.08	Int.			9.327	2.2	22.1	24.0	1.1		.075	.221
91	113	Flume No. 2	41.57	Int.										
92	114	Flume No. 2	41.90	Int.										
93	151	Twin Bridge	101.37	Int.	82	68	9.512	3.2	24.6	27.4	1.5		.283	.257
94	151	Twin Bridge	101.37	Int.	82	68	9.512	3.2	24.6	27.4	1.5		.283	.257
95	154	Twin Bridge	93.45	.2 & 8[2]	82	68	9.512	3.1	24.3	27.0	1.4		.197	.254
96	155	Twin Bridge	94.76	Int.[2]	82	68	9.512	3.1	24.4	27.0	1.4		.197	.255
		Mean 1	26.13											
		Mean 2	54.89											
		Mean 3	56.56											
		Mean 4	51.64											
		Mean 5	97.74											
			95.25	Period from 11 p.m. 6/8/23 to 6 a.m. 6/12/23										
Merced, Burchell Lateral; earth section; loam and clay adobe.														
97	157	Burchell	13.39	Int.	82[3]	68[3]	1.822	1.9	12.0	13.0	.75			.023[3]
		Mean 1	13.39											

(1) From June record. Tests 69 to 90, incl.

(2) Measurements made on different days. Tests 95 and 96.

(3) Yosemite LeGrand records.

Gagings number: 39, 41, 43, 45, 47, 49, 53, 55, 57, 59, 65, 70, 89, 91, 93, 95, 97, 99, 101, 103, 105, 107, 110, 112, 114, 116, 118, 120, 122, 126, 128, 146 and 148 were made with a Hoff propeller meter. All others were made with Price cup meters.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Lower station: Gaging number	Lower station: Location	Lower station: Discharge	Lower station: Method	Lower discharge, evaporation, diversions & leaks	Seepage loss: In section	Seepage loss: Per mile	Seepage loss: Per mile	Seepage loss: Per sq. ft. wetted area in 24 hr.	Classification number
			c.f.s.		c.f.s.	c.f.s.	c.f.s.	%	cu.ft.	
Merced Yosemite LeGrand, earth section; loams, gravelly loams, clay loams and clay adobe.										
69	90	Flume No. 2	20.47	Int.	20.64	5.05	.743	2.89	.573	1
70	91	Flume No. 2	21.56	Int.	21.73	4.84	.712	2.68	.544	1
71	92	Bear Creek	16.25	Int.	16.51	9.18	.920	3.58	.710	
72	93	Bear Creek	21.30	Int.	21.56	5.01	.502	1.89	.384	
73	92	Bear Creek	16.25	Int.	16.34	4.13	1.300	6.35	1.032	
74	93	Bear Creek	21.30	Int.	21.39	0.17	.054	.250	.042	
75	96	Flume No. 2	43.18	Int.	43.38	1.25	.184	.412	.128	3
76	97	Flume No. 2	44.22	Int.	44.42	1.03	.151	.332	.105	3
77	98	Tunnel	44.52	Int.	44.81	–.18	–.019	–.043	–.013	2
78	99	Tunnel	45.56	Int.	45.85	–.40	–.043	–.095	–.030	2
79	98	Tunnel	44.52	Int.	44.61	–1.43	–.566	–1.31	–.394	4
80	99	Tunnel	45.56	Int.	45.65	–1.43	–.566	–1.28	–.392	4
81	102	Flume No. 2	58.82	.2 & .8	59.04	8.53	1.254	1.86	.817	3
82	103	Flume No. 2	60.36	.2 & .8	60.58	8.03	1.181	1.72	.764	3
83	104	Tunnel	59.64	.2 & .8	59.96	7.61	.816	1.21	.526	2
84	105	Tunnel	58.56	.2 & .8	58.88	9.73	1.043	1.52	.674	2
85	104	Tunnel	59.64	.2 & .8	59.74	–0.92	–.364	–.619	–.240	4
86	105	Tunnel	58.56	.2 & .8	58.66	1.70	.673	1.12	.444	4
87	108	Tunnel	50.67	Int.	50.98	5.44	.583	1.03	.389	2
88	108	Tunnel	50.67	Int.	50.98	6.70	.718	1.24	.479	2
89	111	Tunnel	44.84	Int.	45.14	3.54	.380	.781	.260	2
90	112	Tunnel	45.04	Int.	45.34	4.74	.508	1.01	.346	2
91										
92										
93	152	Tunnel	89.47	Int.	90.01	11.36	1.194	1.18	.713	5
94	153	Tunnel	90.88	.2 & .8	91.42	9.95	1.046	1.03	.624	5
95	156	Tunnel	86.87	Int.	87.32	6.13	.644	.689	.390	5
96	156	Tunnel	86.87	Int.	87.32	7.44	.782	.825	.474	5
							Mean 1	2.78	.559	
							Mean 2	.91	.335	
							Mean 3	1.22	.465	
							Mean 4	–.398	–.139	
							Mean 5	.938	.551	
		Period from 11 p.m. 6/8/23 to 6 a.m. 6/12/23						.687	.39	
Merced, Burchell Lateral; earth section; loam and clay adobe.										
97	158	Santa Fe R. R.	13.26	Int.	13.28	.11	.060	.450	.076	
							Mean 1	.450	.076	

Note: The negative signs indicate gains.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Upper station: Gaging number	Upper station: Location	Upper station: Discharge	Upper station: Method[1]	Average Temperature, F: Air	Average Temperature, F: Water	Length of section	Canal depth	Width of water surface	Length of wetted perimeter	Velocity in canal per sec.	Diversions	Leaks	Evaporation
			c.f.s.		deg.		mi.	ft.	ft.	ft.	ft.	c.f.s.	c.f.s.	c.f.s.
Anderson-Cottonwood Main; earth section; gravelly loam of different types.														
98	115	Station No. 1	128.82	Int.	72	64	1.932	3.7	33.7	35.0	1.3	1.43	.68	.083
99	116	Station No. 1	124.41	Int.	72	64	1.932	3.7	33.7	35.0	1.2	1.43	.68	.083
100	119	Station No. 1	141.28	.2 & .8	72	64	1.932	4.0	34.3	35.8	1.3	2.83	.51	.084
101	120	Station No. 1	143.35	.2 & .8	72	64	1.932	4.0	34.3	35.8	1.3	2.83	.51	.084
102	123	Station No. 1	146.24	M.P.	72	64	1.932	4.1	34.7	36.3	1.3	5.19	.77	.085
		Mean (1)	136.82											
Anderson-Cottonwood Main; earth section; loam and gravelly loam of different types.														
103	125	Station No. 3	68.74	Int.	72	64	4.914	3.1	22.9	24.8	1.1	17.40	1.55	.143
104	126	Station No. 3	64.77	Int.	72	64	4.914	3.1	22.9	24.8	1.0	17.40	1.55	.143
105	129	Station No. 3	67.16	.2 & .8	72	64	4.914	3.0	22.7	24.5	1.1	19.18	1.66	.142
106	130	Station No. 3	68.89	Int.	72	64	4.914	3.0	22.7	24.5	1.1	19.18	1.66	.142
107	133	Station No. 3	75.50	.2 & .8	72	64	4.914	3.2	23.2	25.1	1.1	15.86	1.79	.145
108	134	Station No. 3	76.76	Int.	72	64	4.914	3.2	23.2	25.1	1.1	15.86	1.70	.145
		Mean (1)	70.30											
Anderson-Cottonwood Lateral 9; earth section; gravelly loam.														
109	137	Green's Bridge	8.10	Int.	72	64	.606	.9	8.5	9.0	1.2		.05	.007
110	138	Green's Bridge	8.54	Int.	72	64	.606	.9	8.5	9.0	1.3		.05	.007
		Mean (1)	8.32											
Orland Lateral No. 8; earth section; gravelly, sandy loam.														
111	141	Lined section	13.75	Int.	88	77	.696	1.8	10.4	11.4	0.89			.013
112	143	Lined section	14.05	.2 & .8	88	77	.696	1.8	10.4	11.4	0.90			.013
		Mean (1)	13.90											
Orland Lateral No. 101; earth section; gravelly, sandy loam.														
113	145	Check	7.63	Int.	88	77	.831	1.2	7.3	8.2	1.0			.011
114	146	Check	7.90	Int.	88	77	.831	1.2	7.3	8.2	1.1			.011
		Mean (1)	7.81											
Orland Lateral No. 211; concrete; gravelly, sandy loam.														
115		Upper Weir	1.71	Weir[1]	88	77	1.629	.5	5.6	5.8	.63		.008	.016
116		Upper Weir	1.71	Weir	88	77	1.202	.4	5.2	5.3	.94			.011
117		Middle Weir	1.67	Weir	88	77	.427	.7	6.4	6.8	.38		.008	.005
118		Upper Weir	1.68	Weir	88	77	1.629	.5	5.6	5.8	.62		.008	.016
119		Upper Weir	1.68	Weir	88	77	1.202	.4	5.2	5.3	.92			.011
120		Middle Weir	1.62	Weir	88	77	.427	.7	6.4	6.8	.37		.008	.005
121		Upper Weir	1.68	Weir	88	77	1.629	.5	5.6	5.8	.62		.008	.016
122		Upper Weir	1.68	Weir	88	77	1.202	.4	5.2	5.3	.92			.011
123		Middle Weir	1.60	Weir	88	77	.427	.7	6.4	6.8	.37		.008	.005
		Mean (1)	1.69											
		Mean (2)	1.69											
		Mean (3)	1.63											
Orland Highline; concrete; loam.														
124	149	End lining.	131.24	.2 & .8	88	77	2.650	2.8	24.9	26.8	2.2		.33	.17
		Mean (1)	131.24											

(1) For tests 115 to 123, inclusive, Cipoletti weirs were used to measure the flow.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Lower station				Lower discharge, evaporation, diversions & leaks	Seepage loss				Classification number
	Gaging number	Location	Discharge	Method		In section	Per mile	Per mile	Per sq. ft. wetted area in 24 hr.	
			c.f.s.		c.f.s.	c.f.s.	c.f.s.	%	cu.ft.	
Anderson-Cottonwood Main; earth section; gravelly loam of different types.										
98	117	Station No. 2	124.34	Int.	126.53	2.29	1.185	.920	.554	1
99	118	Station No. 2	122.19	Int.	124.38	.03	.016	.013	.007	1
100	121	Station No. 2	134.75	.2 & .8	138.17	3.11	1.610	1.14	.736	1
101	122	Station No. 2	136.57	.2 & .8	139.99	3.36	1.739	1.21	.795	1
102	124	Station No. 2	138.53	M.P.	144.57	1.67	.864	.591	.389	1
						Mean (1)		.791	.498	
Anderson-Cottonwood Main; earth section; loam and gravelly loam of different types.										
103	127	Station No. 4	48.57	Int.	67.66	1.08	.220	.320	.145	1
104	128	Station No. 4	50.49	Int.	69.58	–4.81	–.979	–1.51	–.646	1
105	131	Station No. 4	46.32	.2 & .8	67.30	–.14	–.028	–.042	–.019	1
106	132	Station No. 4	46.93	Int.	67.91	0.98	.199	.289	.133	1
107	135	Station No. 4	54.61	.2 & .8	72.31	3.19	.649	.860	.423	1
108	136	Station No. 4	54.69	Int.	72.39	4.37	.889	1.16	.579	1
						Mean (1)		.225	.104	
Anderson-Cottonwood Lateral 9; earth section; gravelly loam.										
109	139	Wheeler's Bridge	6.28	Int.	6.34	1.76	2.904	35.9	5.28	1
110	140	Wheeler's Bridge	6.70	Int.	6.76	1.78	2.937	34.4	5.34	1
						Mean (1)		35.1	5.31	
Orland Lateral No. 8; earth section; gravelly, sandy loam.										
111	142	Bridge	9.84	Int.	9.85	3.90	5.60	40.7	8.04	1
112	144	Bridge	9.94	.2 & .8	9.95	4.10	5.89	41.9	8.45	1
						Mean (1)		41.3	8.24	
Orland Lateral No. 101; earth section; gravelly, sandy loam.										
113	147	Lower Check	5.50	Int.	5.51	2.12	2.55	33.4	5.09	1
114	148	Lower Check	5.89	Int.	5.90	2.09	2.52	31.5	5.03	1
						Mean (1)		32.5	5.06	
Orland Lateral No. 211; concrete; gravelly, sandy loam.										
115		Lower Weir	1.62	Weir[1]	1.64	.07	.043	2.51	.121	1
116		Middle Weir	1.67	Weir	1.68	.03	.025	1.46	.077	2
117		Lower Weir	1.62	Weir	1.63	.04	.094	5.63	.226	3
118		Lower Weir	1.58	Weir	1.60	.08	.049	2.92	.138	1
119		Middle Weir	1.62	Weir	1.63	.05	.042	2.50	.130	2
120		Lower Weir	1.58	Weir	1.59	.03	.070	4.32	.168	3
121		Lower Weir	1.58	Weir	1.60	.08	.049	2.92	.138	1
122		Middle Weir	1.60	Weir	1.61	.07	.058	3.45	.179	2
123		Lower Weir	1.58	Weir	1.59	.01	.023	1.44	.055	3
						Mean (1)		2.78	.133	
						Mean (2)		2.47	.129	
						Mean (3)		3.82	.150	
Orland Highline; concrete; loam.										
124	150	Chute.	128.26	.2 & .8	128.71	2.53	.955	.728	.583	1
						Mean (1)		.728	.583	

([1]) For tests 115 to 123, inclusive, Cipoletti weirs were used to measure the flow.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Upper station: Gaging number	Upper station: Location	Upper station: Discharge	Upper station: Method	Average Temperature, F: Air	Average Temperature, F: Water	Length of section	Canal depth	Width of water surface	Length of wetted perimeter	Velocity in canal per sec.	Diversions	Leaks	Evaporation
			c.f.s.		deg.		mi.	ft.	ft.	ft.	ft.	c.f.s.	c.f.s.	c.f.s.
Fresno Houghton; earth section; sand and sandy loam.														
125	185	Grant Ave.	83.34	.2 & .8	89	78	1.742	2.4	31.8	32.9	2.24	12.95	.04	.099
126	189	Grant Ave.	56.04	.2 & .8	89	78	1.742	1.9	30.6	32.0	2.67	7.89	.02	.095
127	194	Grant Ave.	76.28	.2 & .8	89	78	1.742	2.3	31.5	32.7	2.26	14.48	–.05	.098
		Mean (1)	71.89											
			64.7		Period midnight to 10 p.m. 8/14/23.									
Fresno Briggs Ditch; earth section; loam and sandy loam of different types.														
128	196	Head	34.27	.2 & .8	89	78	4.351	1.8	13.2	14.3	1.54		.12	.102
129	197	Head	34.34	Int.	89	78	4.351	1.8	13.2	14.3	1.56		.12	.102
130	201	Head	34.17	.2 & .8	89	78	4.351	1.8	13.2	14.3	1.56		.15	.102
131	202	Head	34.17	Int.	89	78	4.351	1.8	13.2	14.3	1.55		.15	.102
132	201	Head	34.17	.2 & .8	89	78	1.389	1.8	12.0	13.1	1.70		.02	.030
133	202	Head	34.17	Int.	89	78	1.389	1.8	12.0	13.1	1.71		.02	.030
134	205	Golden Dawn	30.15	.2 & .8	89	78	2.962	1.9	14.0	15.0	1.35		.13	.074
135	206	Golden Dawn	30.67	Int.	89	78	2.962	1.9	14.0	15.0	1.35		.13	.074
		Mean (1)	34.24											
		Mean (2)	34.17											
		Mean (3)	30.41											
			34.24		Period from 8 a.m. 8/18/23 to 10 a.m. 8/19/23									
Turlock Highline; earth section; loam, sandy loam and clay loam of different types.														
136	207	Head	137.94	.2 & .8	90	78	12.23	4.0	29.1	31.9	1.38			.402
137	208	Head	135.27	Int.	90	78	12.23	4.0	29.1	31.9	1.37			.402
		Mean (1)	136.60											
Turlock Lateral No. 17-B; earth section; sand and sandy loam of different types.														
138	212	Head	17.03	.2 & .8	90	78	4.22	1.7	12.3	13.1	.72	1.48	.05	.059
139	213	Head	17.62	Int.	90	78	4.22	1.7	12.3	13.1	.74	1.48	.05	.059
140	216	Head	17.26	.2 & .8	90	78	4.22	1.7	12.3	13.1	.76		.18	.059
141	217	Head	17.82	Int.	90	78	4.22	1.7	12.3	13.1	.78		.18	.059
142	216	Head	17.26	.2 & .8	90	78	1.79	1.3	11.8	12.4	1.30		.02	.024
143	217	Head	17.82	Int.	90	78	1.79	1.3	11.8	12.4	1.36		.02	.024
144	221	Section 22	14.85	.2 & .8	90	78	2.43	2.1	12.7	13.8	.55		.16	.035
145	222	Section 22	15.86	Int.	90	78	2.43	2.1	12.7	13.8	.57		.16	.035
		Mean (1)	17.43											
		Mean (2)	17.54											
		Mean (3)	15.36											
			17.14		Period from 2 p.m. 9/2/23 to 6 a.m. 9/4/23									
Sutter-Butte Main; earth section; loam.														
146	223	Cox Spillway	210.58	.2 & .8	68	62	6.553	5.1	40.8	43.6	1.1	27.65[1]	0.71	.090
147	226	Cox Spillway	206.91	.2 & .8	68	62	6.553	5.2	41.0	43.8	1.1	24.26	0.89	.090
148	227	Cox Spillway	209.39	Int.	68	62	6.553	5.2	41.0	43.8	1.1	24.29	0.89	.090
149	231	Cox Spillway	178.48	.2 & .8	68	62	6.553	2.7	33.1	34.5	2.2	10.48		.073
150	232	Cox Spillway	176.42	Int.	68	62	6.553	2.7	33.1	34.5		10.48		
		Mean (1)	208.96											
			207.4		Period from 4 a.m. 9/21/23 to midnight 9/22/23.									
Sutter-Butte Green lateral; earth section; clay adobe.														
151	235	Flume	5.94	.2 & .8	68	62	3.036	1.41	16.3	16.8	.22	.68	.12	.017
152	236	Flume	6.14	Int.	68	62	3.036	1.41	16.3	16.8	.22	.68	.12	.017
153	239	Flume	5.15	.2 & .8	68	62	3.036	1.42	16.4	16.9	.27		.29	.017
154	240	Flume	5.77	Int.	68	62	3.036	1.42	16.4	16.9	.28		.29	.017
155	243	Flume	5.63	.2 & .8	68	62	3.036	1.55	17.5	18.0	.25		.42	.018
156	244	Flume	5.76	Int.	68	62	3.036	1.55	17.5	18.0	.26		.42	.018
		Mean (1)	5.73											

([1]) One small diversion not measured, omitted.

TABLE 9. Continued—*Seepage Measurements (Inflow-Outflow Method).*

Test number	Lower station				Lower discharge, evaporation, diversions & leaks	Seepage loss				Classification number
	Gaging number	Location	Discharge	Method		In section	Per mile	Per mile	Per sq. ft. wetted area in 24 hr.	
			c.f.s.		c.f.s.	c.f.s.	c.f.s.	%	cu.ft.	
		Fresno Houghton; earth section; sand and sandy loam.								
125	186	West Lawn Ave.	72.07	.2 & .8	85.16	-1.82	-1.045	-1.25	-.520	1
126	190	West Lawn Ave.	50.22	.2 & .8	58.23	-2.19	-1.257	-2.24	-.643	1
127	195	West Lawn Ave.	61.83	.2 & .8	76.36	-.08	-.046	-.060	-.023	1
						Mean (1)		-1.09	-.394	
		Period midnight to 10 p.m. 8/14/23.						.462	.150	
		Fresno Briggs Ditch; earth section; loam and sandy loam of different types.								
128	198	Jensen Ave.	28.96	.2 & .8	29.18	5.09	1.170	3.41	1.34	1
129	199	Jensen Ave.	29.58	Int.	29.80	4.54	1.043	3.04	1.19	1
130	203	Jensen Ave.	29.85	.2 & .8	30.10	4.07	.935	2.74	1.07	1
131	204	Jensen Ave.	29.46	Int.	29.71	4.46	1.025	3.00	1.17	1
132	205	Golden Dawn	30.15	.2 & .8	30.20	3.97	2.86	8.37	3.57	2
133	206	Golden Dawn	30.67	Int.	30.72	3.45	2.48	7.26	3.10	2
134	203	Jensen Ave	29.85	.2 & .8	30.05	0.10	.034	.113	.037	3
135	204	Jensen Ave.	29.46	Int.	29.66	1.01	.341	1.11	.372	3
						Mean (1)		3.05	1.194	
						Mean (2)		7.81	3.33	
						Mean (3)		.617	.204	
		Period from 8 a.m. 8/18/23 to 10 a.m. 8/19/23						3.07	1.19	
		Turlock Highline; earth section; loam, sandy loam and clay loam of different types.								
136	209	East Ave. Bridge	130.31	.2 & .8	130.71	7.23	.591	.428	.303	1
137	210	East Ave. Bridge	130.64	Int.	131.04	4.23	.346	.256	.177	1
						Mean (1)		.343	.239	
		Turlock Lateral No. 17-B; earth section; sand and sandy loam of different types.								
138	214	Section 17-20	7.06	.2 & .8	8.65	8.38	1.986	11.66	2.48	1
139	215	Section 17-20	7.27	Int.	8.86	8.76	2.076	11.78	2.59	1
140	219	Section 17-20	8.21	.2 & .8	8.45	8.81	2.088	12.10	2.61	1
141	220	Section 17-20	8.21	Int.	8.45	9.37	2.220	12.46	2.77	1
142	221	Section 22	14.85	.2 & .8	14.89	2.37	1.324	7.67	1.75	2
143	222	Section 22	15.86	Int.	15.90	1.92	1.073	6.02	1.42	2
144	219	Section 17-20	8.21	.2 & .8	8.41	6.44	2.65	17.84	3.14	3
145	220	Section 17-20	8.21	Int.	8.41	7.45	3.06	19.29	3.63	3
						Mean (1)		12.00	2.61	
						Mean (2)		6.83	1.58	
						Mean (3)		18.59	3.38	
		Period from 2 p.m. 9/2/23 to 6 a.m. 9/4/23						11.96	2.56	
		Sutter-Butte Main; earth section; loam.								
146	225	Pumping Station	162.80	.2 & .8	191.25	19.33	2.95	1.40	1.11	1
147	229	Pumping Station	166.70	.2 & .8	191.94	14.97	2.28	1.10	.85	1
148	230	Pumping Station	168.33	Int.	193.60	15.79	2.41	1.15	.90	1
149	234	Pumping Station	152.21	.2 & .8	162.76	15.72	2.40	1.34	1.14	
150										
						Mean (1)		1.22	.95	
		Period from 4 a.m. 9/21/23 to midnight 9/22/23						1.11	.86	
		Sutter-Butte Green lateral; earth section; clay adobe.								
151	237	Venturi Flume	1.54	.2 & .8	2.36	3.58	1.179	19.8	1.15	1
152	238	Venturi Flume	1.51	Int.	2.33	3.81	1.255	20.4	1.22	1
153	241	Venturi Flume	4.12	.2 & .8	4.43	0.72	.237	4.60	.23	1
154	242	Venturi Flume	4.08	Int.	4.39	1.38	.455	7.89	.44	1
155	245	Venturi Flume	4.27	.2 & .8	4.71	0.92	.303	5.38	.27	1
156	246	Venturi Flume	4.31	Int.	4.75	1.01	.333	5.78	.30	1
						Mean (1)		10.94	.595	

Note: The negative signs indicate gains.

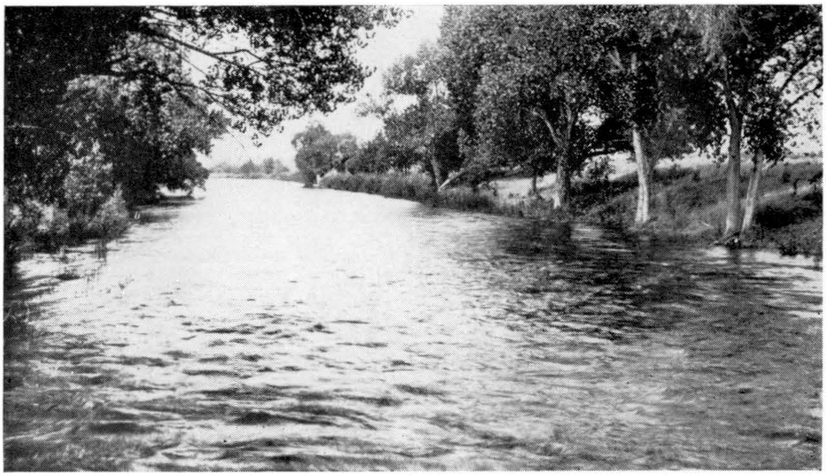

Figure 21.--Alta Main Canal, showing portion of section tested, Dinuba, California.

meter measurements and fortunately divided the canal into two parts approximately according to soil class. Other divisions of the section were made to determine the effect of the velocity, nature of material and cross-section on the seepage. In all, 26 determinations were made, divided into two groups, the first in October 1922, when the canal was carrying about 100 cubic feet per second, and the second in June 1923, when it was carrying from 600 to 900 cubic feet per second. These measurements are summarized according to section and discharge in table 9. Group 1 is for the entire section and is the mean of four tests; groups 2 and 3 are for the upper and lower sections respectively, and are each the means of two tests. These tests indicate that the loss was slightly greater in the sandy section than in adobe section, but since the loss in the whole distance was less than either, the results were not conclusive in this respect. Groups 4 to 8 show about the same losses with somewhat greater variation and no indication that the type of soil had any influence. The difference in the current-meter measurements due to the changes in velocity and cross-section and the difficulty in determining the area accurately at some of the stations in these groups, on account of the soft mud in the bottom, evidently had a greater influence than the type of soil. Group 9 is for the whole section tested and differs from Group 1 only in that the discharge is much greater.

As should be expected, the loss in percentage per mile was less and the rate per unit area was greater than for the smaller discharge. Tests numbered 32 and 33, which might have been included in this group, both showed a gain and have been discarded because there is some doubt as to the accuracy of the area determination in gaging numbers 160 and 163, which were made from the Dunegans Gap bridge during the maximum stage of the canal. Because of the high velocity and the roughness of the bottom at this point, the depth measurements were too great. A continuous record of the gage height of the canal was taken for this period and the total seepage was determined for the period from 4:00 p.m. June 17, 1923, to 6:00 p.m. June 18, 1923, but on account of the error in the two measurements previously noted, the result showed a gain. (See table 9.)

Alta East Branch

The portion of the East Branch Canal tested (figure 22) extends from the headgate of the Orosi School House lateral to the Sand Creek crossing. It was constructed after the main canal, probably about 1890. The bottom width ranged from 12 to 20 feet and the side slopes were about 1½ to 1. The grade of the canal is very flat. The soil, Hanford sandy loam, is uniform throughout the portion of the canal tested. Hardpan occurs but usually at considerable depth, although at some points the canal excavation is partly in hardpan. This region was the flood channel of Sand Creek and on account of the sandy soil large losses were assumed to occur from this section of the canal. The tests, however, with one exception showed a gain. (See table 9.) Six tests were made and in order to reduce the chance of systematic error in the current-meter measurements due to local conditions, the locations of both the upper and lower sections were changed, as was also the time of making the measurements. The results, however, still showd a gain. No waste water was entering the canal. The gaging stations were similar in cross-section and although the average velocities were low, they were very nearly identical. However, the length of the section tested was very short, so that slight errors in the current-meter measurements might completely overshadow the seepage losses. This is probably what occurred.

Figure 22.—East Branch Alta canal showing section under test for seepage losses.

In an attempt to find an explanation for the apparent gain, several profiles of the ground surface, water table, and hardpan on lines perpendicular to the axis of the canal were obtained. These profiles, shown in figures 23, 24 and 25, indicate that at these points at least, the canal was losing some water. The loss was probably small, however, because even when the hardpan was continuous there were no signs of high ground water or

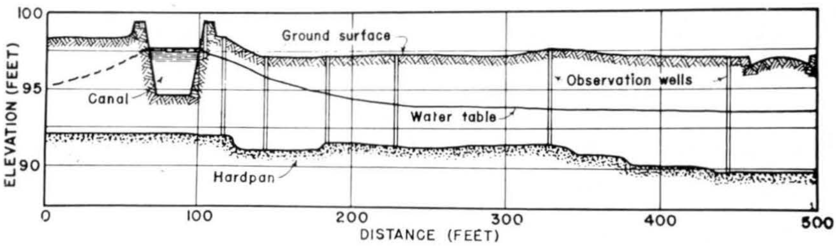

Figure 23.—Ground-water profile perpendicular to Alta East Branch Canal at School House Lateral.

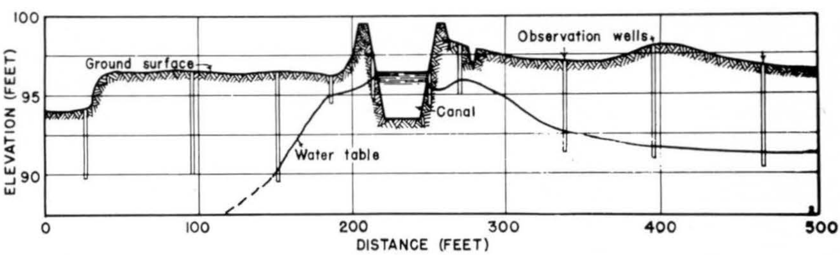

Figure 24.—Ground-water profile perpendicular to Alta East Branch Canal at Orosi Highway.

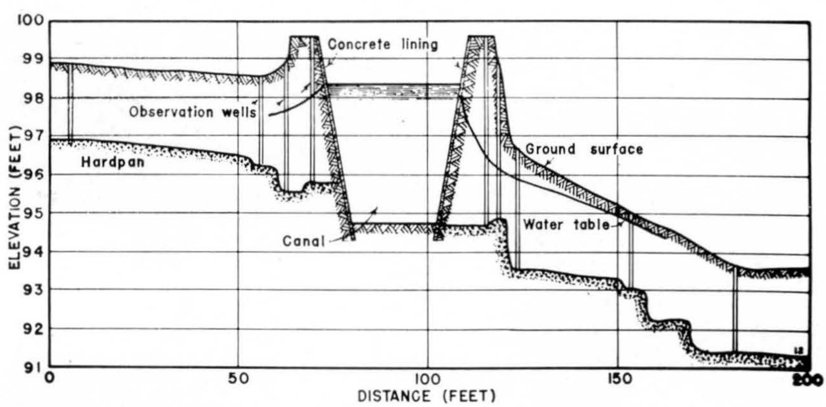

Figure 25.—Ground-water profile perpendicular to concrete-lined section of Alta East Branch Canal at Brann's Crossing.

alkali. Measurement of the true loss by seepage from a pool located in the canal could not be attempted on account of the necessity of keeping the canal in use.

GAGE CANAL

Gage Canal, Riverside, California, was built in the period from 1885 to 1888. The lining was started almost immediately thereafter and completed in 1890. A section of the canal is shown in figure 26. The lining consisted of a $\frac{3}{4}$-inch plaster coat of cement mortar which was applied to the canal section without reinforcement or expansion joints and although the lining was over 30 years old at the time of the tests, except for some buckling and minor failures, it was still in fair condition. The lining is now (1947) over 50 years old and is still in service. The size of the canal varied according to the quantity of water to be carried. Starting with a bottom width of 10 feet at the head it decreased to a width of 5 feet at the lower end. The side slope, $\frac{3}{4}$ to 1, was constant and the grade was about $2\frac{1}{2}$ feet per mile.

The portion of the canal tested, extending from the portal of the first tunnel to the second check below the lower weir, passes through a variety of soils distributed as follows:

$1\frac{3}{4}$ miles Hanford sandy loam; $1\frac{3}{4}$ miles Sierra loam; 5 miles Placentia loam; $\frac{1}{2}$ mile Madera sandy loam; $2\frac{1}{2}$ miles Ramona loam; and $\frac{1}{4}$ mile rock. Hardpan occurs only in the Madera sandy loam, but the various types of loams are all underlain by heavier soils and in the case of the Sierra loam, bedrock occurs close to the surface.

After leaving the valley of the Santa Ana River, from which it draws its water, the canal enters a series of tunnels beginning at the upper end of the portion of the canal under test and continues until it reaches the mesa above

Figure 26.—Concrete lining of Gage Canal at Riverside, California.

Riverside. These tunnels, totalling over a mile in length, which are lined with 6 inches of concrete instead of ¾-inch as elsewhere, were included in making the seepage determination. The canal crosses several deep arroyos by means of wooden flumes. They were excluded from the seepage determinations.

In making the seepage measurements, the original plan was to use the weirs located at each end of the section tested, but a careful inspection showed that the bulkhead of the lower weir was leaking considerably, so this plan was abandoned. Since gage-height recording instruments were located at these points and since the weirs would furnish ideal controls for determining the gage heights, the gage records were kept at these points while the discharge measurements were made at points more favorably located for current-meter work. The length of the section was measured between the points at which the current-meter measurements were made. Many diversions occurred in this portion of the canal, but fortunately at the time the tests were being made (December 19, 1922 to December 29, 1922), no water was being used for irrigation and the only water taken out was for domestic use. This use was assumed to be constant and when measured, was found to amount to 0.490 cubic foot per second.

Since a continuous record of the gage height was available, both the total seepage and the seepage during the separate tests were determined. The results are set out in table 9. Although the results of the individual tests showed some variation, the means agreed reasonably well with the results obtained from the continuous record and both indicated a very small loss.

IMPERIAL IRRIGATION DISTRICT

West Side Main

West Side Main of the Imperial Irrigation District, Imperial, California, was built between 1900 and 1905. It is the highline canal on the west side of the valley. The portion of the canal tested extends from the West Side drain to the District's gaging station at Thompson's Crossing, a distance of 14.69 miles. The original section of the canal has been completely changed by the deposition of silt and the cleaning by dragline and other equipment. This work went on almost continuously. Figure 27 shows a section of the canal just after one side had been cleaned.

The canal passes through a variety of soil types divided approximately as follows:

¾ mile Rositas very fine sand; 4½ miles Rositas sand; ¼ mile Carrizo sand; ½ mile Superstition fine sand; 1 mile Meloland fine sand; 3 miles Holtville silty clay; 1½ miles Holtville clay; and 3 miles Imperial clay. These are all alluvial and lacustrine soils and are usually found in horizontal layers except at Thompson's Crossing where the older layers have been sharply tilted and then covered with horizontal deposits. The sandy soils are underlain with layers of sand except the Meloland fine sand which has a subsoil of silty clay loam, silty clay and clay. The subsoil of the Holtville

Figure 27.—Partially cleaned section West Side Main, Imperial Irrigation District, Imperial, California.

clay and Holtville silty clay is of lighter texture, but the Imperial clay is a uniformly hard soil, heavily impregnated with alkali and impervious to water. These soils represent the material in which the canal was excavated, but at the time of the tests the bed of the canal was probably made up almost entirely of different materials, principally silt deposited from the water in the canal.

There was little surface indication of seepage from the canal even where the gorge of the New River cut close to the canal during the flood from the Colorado River in 1906. The profile of the ground water (figure 28), made near Thompson's Crossing where the river is within 200 feet of the canal, shows how the ground water slopes under these conditions. No water appeared on the surface of the cliff at this point.

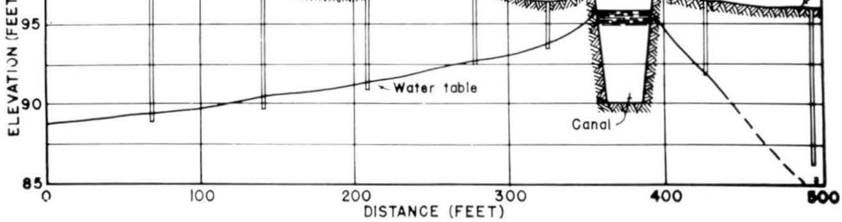

Figure 28.—Ground-water profile perpendicular to Imperial West Side Main at Thompson's Crossing.

In making the seepage determinations, six tests were completed and one abandoned on account of trouble with sand in the bearings of the meter. The results of these tests varied considerably, but since the loss was small even in the long section of the canal under test, this was to be expected because of the possibility of error in the current-meter measurements. The results are shown in table 9, numbers 55-61.

During the seepage measurements, a continuous record of the gage height at each end of the section was obtained. This record, with the discharge data of the separate tests, made possible the determination of the total seepage loss for any period covered by the tests. The results for the 4-day period from January 22 to January 25, 1923, are given in table 9. These values are nearly twice as great as the mean values for the separate tests, but being based on the continuous record, should be given greater weight.

Fillaree Lateral

Fillaree Lateral diverts water from the West Side Main and was built during the cotton boom of World War I. It heads at the Fern check and irrigates land above the West Side Main. The bottom of the lateral was from 2 to 4 feet wide. The portion of the lateral under test extends from the Fern check to the bridge 3.17 miles below the head. Another bridge 1.37 miles below the headgate furnished an intermediate gaging station for determining the losses in a shorter section of the lateral. The soils cut by the lateral are similar to those on the main canal, viz., Holtville silty clay and Rositas, Meloland and Superstition fine sand. Sand and clay soils are about equally divided. The lateral was cleaned with a Ruth dredge in 1922, but was completely lined with silt at the time the seepage tests were made.

Although the soils are similar to those in the main canal, the losses were greater in the lateral. The results of the seven seepage determinations, divided into three groups, are shown in table 9, numbers 62-68. Group 1, the mean of three tests, shows the loss in the entire section, and groups 2 and 3, each the mean of two tests, show the loss in the upper and lower part of the section respectively. The results indicate that the upper section, which is largely in sand, lost more than the lower section, which is largely in clay and apparently gained water. As this is the highline canal there was no possibility of a gain. Since the amount was small it was probably due to the inaccuracies in the current-meter measurements.

MERCED IRRIGATION DISTRICT

Yosemite-LeGrand Canal

Yosemite-LeGrand Canal, Merced, California, extending from Lake Yosemite to LeGrand, was completed in 1923. The canal had a bottom width of 16 feet, side slopes 1½ to 1, and grade of 0.00015. At a depth of 5.8 feet on the assumption that $n = .030$, the capacity was 200 cubic feet per second. The portion of the canal tested is in the section between Lake Yosemite and the Bear Creek Crossing, but on account of the changing conditions several changes were made in the actual points at which the measurements were made. Most of the measurements at the upper end of the canal were made at the lower end of Flume No. 1, but because of the higher velocity in the flume with increasing discharge, and the attempt during the last four tests, to find locations which would make the upper and lower stations more nearly alike, the upper station was moved to the highway bridge above the flume. At first the lower station was located at the wasteway at the Bear Creek Crossing, but it was found difficult to measure the flow at that point owing to the very high velocity and the fact that the water was not always discharged there. For this reason the station was moved to the mouth of the tunnel above the wasteway. Flume No. 2, at a distance of 6.801 miles below Flume No. 1, was used as an intermediate gaging station.

The canal runs through a great variety of soils, all of which are quite heavy. From the highway bridge to Flume No. 2 it runs through 1/2 mile of Alamo clay adobe, 2 1/2 miles of San Joaquin loam and 3 miles of Redding gravelly loam. From Flume No. 2 to Bear Creek it runs through 1 1/4 miles of Redding gravelly loam, 1/2 mile of Alamo clay adobe, and 1/2 mile of Altamont loams and clay loams. The Alamo clay adobe is a heavy soil from 2 to 4 feet deep resting on a ferruginous hardpan. The San Joaquin loam has a similar substratum and is at about the same depth. Redding gravelly loam is a shallower soil with a heavy clay subsoil which overlies a dense iron-clay hardpan. The Altamont loams and clay loams are shallow soils resting directly on the parent rocks. Coarse gravel was frequently encountered in the canal excavations and at some points small boulder beds occurred. Through these porous areas considerable seepage might be expected, but the tests did not bear this out, even though the canal was being used for the first time when these tests were being made.

The tests on the canal were made at two different periods, the first from April 25 to May 1, 1923, and the second from June 8 to 11, 1923. Twenty-eight tests in all were attempted, but of these only 26 were completed and four of these have been disregarded because the measurements in the Bear Creek wasteway are believed to be in error. (See table 9, numbers 69-96.) These tests were divided into five groups depending on the discharge and the portion of the canal involved. The mean seepage values for the different groups are set out in table 9. Groups 1 and 3 were for the same section of the canal, but the discharge was about twice as great in group 3. The results show that the loss was greater for the smaller discharge both in percentage per mile and cubic feet per unit area in 24 hours. This was probably due to the fact that the tests of group 1 were made very soon after the water was turned into the canal. Groups 2 and 5 were also for similar sections and variable quantities, but the greater quantity showed a larger loss both in percentage per mile and cubic feet per unit area in 24 hours. The percentage per mile was only very slightly greater, as should be expected. Group 4, which was for the section from Flume No. 2 to the lower portal of the tunnel, showed a slight gain probably caused by errors in the current-meter measurements due to dissimilarity of section and difference in velocity.

During the tests of the second period, or Group 5, a continuous record of the gage heights was obtained from recording instruments installed at the upper and lower stations. The seepage loss computed from this record and the discharge measurements agree reasonably well with the mean of the results of the individual tests of group 5.

The surface indications of seepage from the canal were few, although there had been no chance to build up a silt lining. A few leaks occurred at the bottom of fills and during the first-period tests some water also leaked in through the upper banks. This, however, disappeared before the second period.

Burchell Lateral

Burchell lateral, built in 1923, is the highline canal carrying water to the booster station north of LeGrand. The portion tested extends from station 297+50 to station 393+69. The bottom width was 5 feet, the side slopes were 1½ to 1, and the grade was 0.0002. At a depth of 3.8 feet with a value of $n=0.030$, the capacity was 45 cubic feet per second. The lateral was excavated in Madera loam and Montezuma clay adobe in approximately equal amounts. The Madera loam has a hardpan substratum whereas the Montezuma clay adobe, a much heavier soil, has a cemented calcareous subsoil. A single test was made; before others could be attempted, conditions had so changed that comparable results could not be obtained. The result of this one test (No. 97), indicated a small loss, but it could not be given much weight without supporting tests. A sinkage measurement, made on a pool in a small lateral east of Planada in Montezuma clay adobe, indicated a loss in cubic feet per square foot per 24 hours, nearly twice as great as that from the Burchell lateral. In either case the loss was no more than should be expected from an average concrete lining and consequently it is hardly to be expected that the exact amount of the loss could be detected by current-meter measurements.

ANDERSON-COTTONWOOD IRRIGATION DISTRICT

Main Canal

The main canal of the Anderson-Cottonwood Irrigation District, Redding, California, was built about 1915. It is 30 miles long and extends from Redding to the south side of Cottonwood Creek. The bottom width of the canal depends on the capacity required, and in the sections under test, the range was from 12 to 26 feet. The side slopes were 1½ to 1 and the grade was 0.000145. At the upper end it was designed for a water depth of 6 feet, a freeboard of 1½ feet and a capacity of 365 cubic feet per second. The water carried was usually quite clear and no appreciable silting had occurred.

Two portions of the main canal were chosen for testing: the first, between Clear Creek and Spring Gulch, extends from station 521 to station 623 and the second, between Anderson and Cottonwood, which includes Panorama Point, extends from station 872+27 to station 1131+73. These sections were both chosen because the seepage loss was reported to be heavy although the soil in each case was rather dense. The upper section is cut largely through Anderson gravelly loam, but some Redding gravelly loam was also encountered. The second section is mostly in Redding loam and

Figure 29.—Anderson-Cottonwood canal, Redding, California, showing canal section and type of soil.

gravelly loam, with only a small amount of Anderson gravelly loam. The Redding loam and gravelly loam are red loam soils with a heavy subsoil resting on a dense hardpan at variable depths. The Anderson gravelly loam is a shallow soil resting directly on a hardpan substratum. These soils contain considerable gravel and at some points fair-sized boulders were encountered. Figure 29 shows a section of the canal and the type of soil in which it was excavated.

The seepage tests on the Main Canal were made in May 1923. On the upper section, between Clear Creek and Spring Gulch, stations 1 and 2, five tests were made as shown by test numbers 98 to 102 in table 9. The results are quite consistent and the mean value, although indicating considerable seepage, is not as large as might be expected from the waterlogged condition of the land in this region. Figure 30 shows a profile of the ground water here. On the lower section, between Anderson and Cottonwood (stations 3 and 4), six tests were made as shown by numbers 103 to 108 in table 9. The results varied considerably, but this was partly due to the

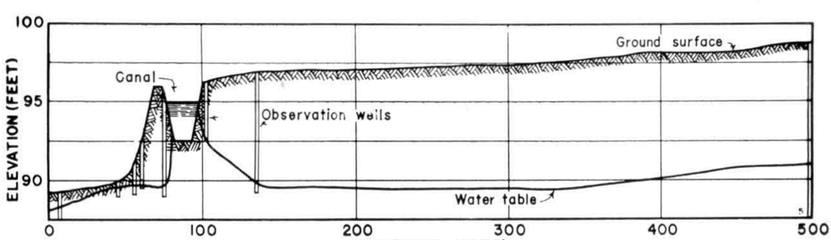

Figure 30.—Ground-water profile perpendicular to Anderson-Cottonwood Main Canal at crossing on road west from Cottonwood.

fact that one of the meters used was out of adjustment and after its use was discontinued more consistent results were obtained. Some variation is probably due also to the large diversions which occurred in this section. The mean of all the tests indicated a much smaller seepage loss than occurred in the upper section and as far as the surface indications were concerned, this was to be expected. The only waterlogging that was observed occurred in the area adjacent to Panorama Point where the sidehill construction of the canal had materially weakened the lower bank.

Lateral No. 9

Lateral No. 9 of the Anderson-Cottonwood Irrigation District diverts water from the Main canal a short distance below Wood Gulch and irrigates the land between the canal and the Sacramento River. The lateral is irregular in section, ranging in bottom width from 2 to 7 feet. The portion tested for seepage losses extends from Green's house to the point where the lateral crosses Wheeler's road. This section of the lateral was excavated entirely in Anderson gravelly loam. There were surface indications of seepage from

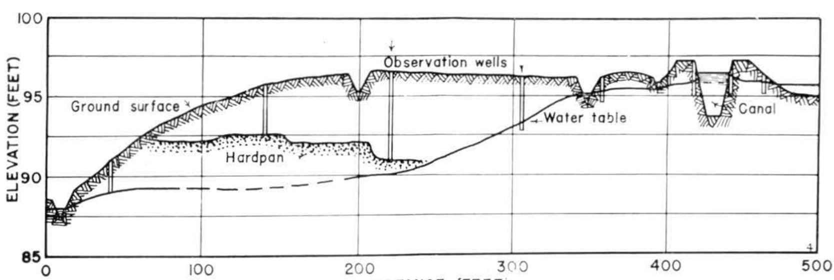

Figure 31.—Ground-water profile perpendicular to Anderson-Cottonwood Main Canal at Green's place.

the lateral, but as it parallels the Main canal, both were probably contributing causes. The profile shown in figure 31, which is across a dry portion of lateral 9 in this region, shows that the Main canal was leaking, but the results of the seepage tests on the lateral (numbers 109-110 in table 9), show that a large loss was occurring here also. Only two tests were made, but a different gaging station was used in each case. For the first test high-velocity gaging stations were used and for second, low-velocity stations. As shown in the table the results were very consistent.

ORLAND PROJECT

Highline Canal

Construction of the Highline Canal was started in 1908 after the Orland project had been taken over by the United States Bureau of Reclamation. A small amount of development had occurred previous to that time, but most of the canals were built or enlarged by the Bureau of Reclamation. The Highline Canal was built to replace the old main canal. It extends from

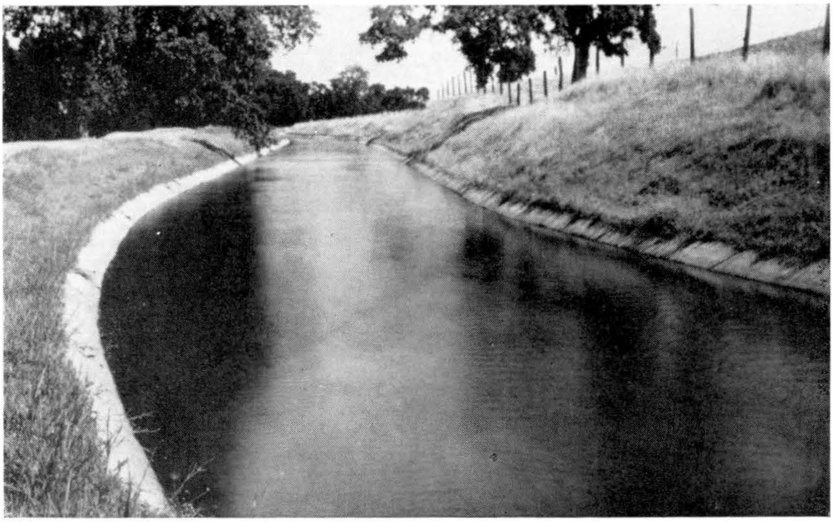

Figure 32.—Concrete-lined Highline Canal, Orland Project, Orland, California.

Stony Creek to the Hambright siphon where it rejoins the old canal. The portion under test extends from approximately the beginning of the concrete-lined section below Hall's check to the chute at Hambright Creek. The bottom width of the canal ranged from 14 to 20 feet, the side slopes from 1½ to 1 to 2 to 1, and the grade from 0.00035 to 0.0006. This portion of the canal is outside the area covered by the soil map of this region and although definite information was not available, the soil is believed to be San Joaquin loam. This is reddish loam with a heavier substratum which rests at various depths on a nearly impervious red gravelly hardpan. The lining of this section, which is mostly 2 inches thick, was applied at different times, and was all apparently in good condition. (See figure 32.) Some seepage was visible on the lower side of the canal, and although but one test was made on account of the limited time available, the results (table 9, test number 124), showed a rather high rate of seepage from this type of lining, but still small in comparison with the seepage from unlined canals on the project.

Lateral No. 8

The portion of Lateral 8 under test was unlined. It extended from the lower end of the lined section to the bridge, 0.696 miles below this point. The bottom width was 5 feet and the side slopes 1½ to 1. The lateral was excavated in Sacramento gravelly sandy loam, an open soil of considerable depth in which large seepage losses occurred. Two seepage tests were made, as set out in table 9 (test numbers 111 and 112). They both showed very large losses and as was to be expected, checked very closely.

Lateral No. 101

Lateral 101 is on the north side of Stony Creek and heads about ½ mile below North Diversion. The bottom width was 3 feet and the side slopes 1½ to 1. All of the lateral was lined except the portion under test, which was located between two lined sections. The soil in this area is Sacramento gravelly sandy loam, the same as found at Lateral 8, and the seepage losses were almost as high. (See test numbers 113 and 114, table 9.) The results checked very closely. Measurements made by representatives of the Bureau of Reclamation on a longer portion of this canal and with a greater discharge, showed a loss about 30 percent smaller which, considering the difference of conditions, was entirely possible. Borings made here disclosed a bed of coarse gravel and boulders from 5 to 6 feet beneath the surface which probably provided the drainage channel for the large losses from the lateral.

Lateral No. 211

Lateral 211 runs parallel in a general way to South Canal and the section under test was quite close to the large canal. The soil is Sacramento gravelly sandy loam and on account of the heavy seepage from this type of soil, the lateral was lined throughout with a 1½-inch layer of concrete. The lining was in good condition. Cipolletti weirs were provided for measuring the flow and were so located as to divide the section under test into two parts. The results of the tests are reported in table 9, test numbers 115 to 123. They show some variation, but the mean values for the different groups check fairly well and indicate that the losses were about the same throughout the section. Compared with the results of the seepage tests on the unlined canals, they indicate that the 1½-inch lining reduced the seepage loss 98 percent. Tests made by representatives of the Bureau of Reclamation on a shorter section of the lateral, but with a discharge 10 times as great, showed about a 50 percent higher seepage.

FRESNO IRRIGATION DISTRICT

Houghton Canal

Houghton Canal, shown in figure 14, is one of the large laterals of the Fresno Irrigation District, Fresno, California. It was built in the early days and has lost its original shape. The section under test, which lies between Grant Avenue and Westlawn Avenue, was from 14 to 22 feet in bottom width and had side slopes approximating natural channels. The soil is Fresno sand and Fresno sandy loam, distributed in about equal proportions along the section. Fresno sand is a deep soil, free of hardpan. Fresno sandy loam is underlain with a silty clay loam which rests on a layer of hardpan through which the canal was excavated at some points. Alkali and high ground water were serious problems in the region, but the seepage tests on the canal given in table 9, numbers 125-127, showed a gain rather than a loss, which indicated that the trouble was due to other causes. A continuous gage-height record was kept during the tests and the seepage computed on the basis of total inflow and outflow indicated a small loss as set out in table 9. The disagreement was probably due to fluctuations in the canal which were

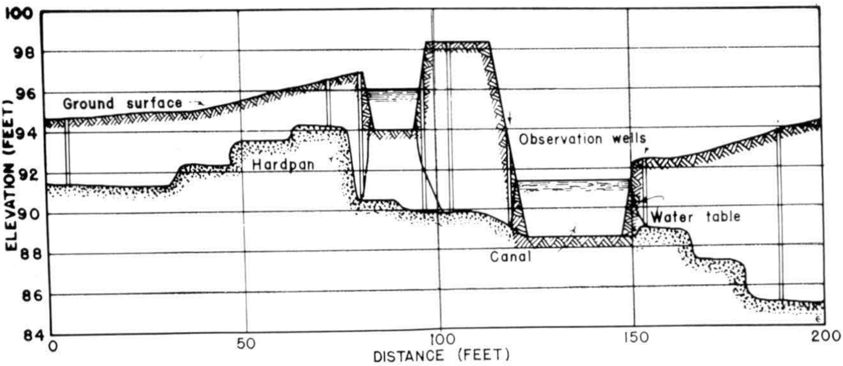

Figure 33.—Ground-water profile perpendicular to Houghton Canal at High Ditch Check.

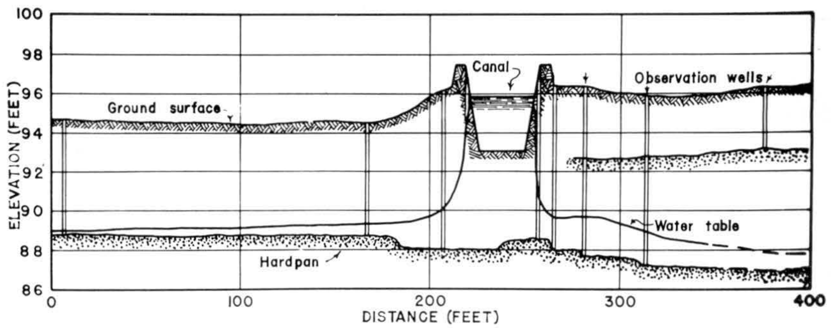

Figure 34.—Ground-water profile perpendicular to Houghton Canal at Westlawn Avenue.

taken into account only by the latter method. To investigate how the seepage was taking place, two profiles of the ground surface, ground water and hardpan as shown in figures 33 and 34 were made at right angles to the canal. The profile near Grant Avenue, figure 33, where the canal was partly excavated in hardpan, shows that little or no loss occurred here but the profile near Westlawn Avenue, figure 34, shows that seepage was occurring in appreciable quantities. Measurements of sinkage made in the canal from a pool extending from below High Check to the check at Westlawn Avenue showed results only slightly higher than those from the continuous record. (See table 8.)

Briggs Ditch

Briggs Ditch was built in 1886 by the farmers as a cooperative enterprise. It diverts water from Fancher Creek, which serves as a lateral at this point and irrigates land to the south in the direction of Fowler. The portion of the canal under test extended from the head to the Jensen Avenue crossing. The cross-section of the canal was irregular. The bottom was from 6 to 8 feet wide, while the depth remained constant at about 2 feet. The canal was excavated in light sandy soil of the following types: Madera fine sandy loam, Madera sand, Madera sandy loam and San Joaquin sandy loam, of which all types are about equally represented. Eight seepage tests were made, as set out in table 9, numbers 128-135. To determine, if possible, the distribution of the seepage losses, the canal was divided into two sections by an intermediate gaging station which separated the tests into three groups. Group 1 was for the entire section under test. and as shown in the table, the results of the individual tests were consistent. Groups 2 and 3 were for the upper and lower sections and although not so consistent, indicated that most of the loss occurred in the upper section of the canal where it runs close to Fancher Creek. These results should not be given too much weight, however, because an appreciable change of stage occurred while these latter tests were being made; but profiles of the ground water in the two sections, as shown in figures 35 and 36, indicate that the loss was greater in the upper section (figure 36).

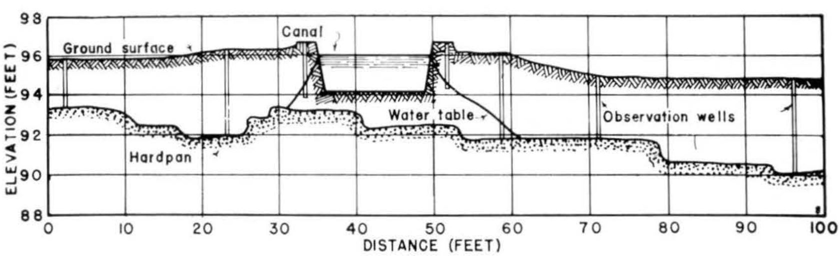

Figure 35.—Ground-water profile perpendicular to Briggs Ditch at Jensen Avenue.

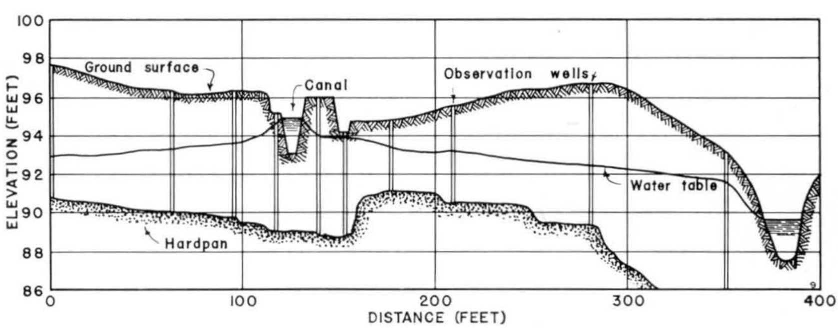

Figure 36.—Ground-water profile perpendicular to Briggs Ditch at Temperance Avenue.

TURLOCK IRRIGATION DISTRICT

Highline Canal

The Highline Canal of the Turlock Irrigation District, Turlock, California, was built about 10 years after the main canal, which was completed in 1901. It irrigates the high ground on the east side of the district and extends from the main canal 3 miles east of Hickman to the Merced River. The portion under test extends from the main canal to the highway crossing about ½ mile below the headgate of Cross Ditch No. 1. The bottom of the canal was from 14 to 22 feet wide and the water was from 2 to 5 feet deep. The canal traverses a variety of soils. For the first 5 miles, the soils are equally divided between Madera loams, Altamont loams and clay loams. For the next 7 miles, approximately ⅔ of the distance is through Madera and San Joaquin sandy loams undifferentiated, and ⅓ through San Joaquin sandy loam. The Altamont loams and clay loams are usually 6 feet or more in depth, but grow heavier in texture and lighter in color as the depth increases. Madera loams are underlain with hardpan at a depth of 6 feet or less as are also the San Joaquin sandy loams undifferentiated. Madera and San Joaquin sandy loams are shallow soils with hardpan from 12 to 24 inches below the surface. With hardpan so close to the surface, it was only natural that some of it should be encountered in excavating the canal. The only indications of seepage from the canal were at the bottom of fills across small drainage channels. Borings made near the canal indicated that the seepage loss was small. The results of the seepage tests (table 9, numbers 136 and 137), showed that the loss was only about twice as great as from a good concrete-lined canal. Only two tests were made on account of the interference with irrigation necessary during the tests, but no change occurred in the stage of the canal so the results are probably quite close to the truth.

Lateral 17-B

Lateral 17-B is a part of the original irrigation system completed in 1901. It diverts water from Upper No. 2 canal where the highway between Keyes and Ceres crosses the canal, and extends about 3 miles west to where it joins Lower No. 2 canal. It is a private ditch and has been indifferently maintained. The bottom width was about 6 feet and the depth varied from 1½ to 2½ feet. The soils through which the canal passes are as follows: 2¾ miles Oakley and Fresno sands undifferentiated and 1½ miles Fresno sandy loam (both phases). Oakley and Fresno sands are light sandy soils of considerable depth, usually underlain with an impervious hardpan of variable thickness. The Fresno sandy loams are of the white ash and brown phases. They have a substratum of calcareous cemented silt at depths ranging from 2 to 6 feet which is sometimes known as hardpan.

The results of the seepage tests on the lateral (table 9, numbers 138-145), showed that heavy losses occurred. In an attempt to isolate the area of maximum seepage the lateral was divided into two sections. The results of these tests indicated that the maximum loss occurred in the lower portion. Too much weight could not be attached, however, to these tests because the

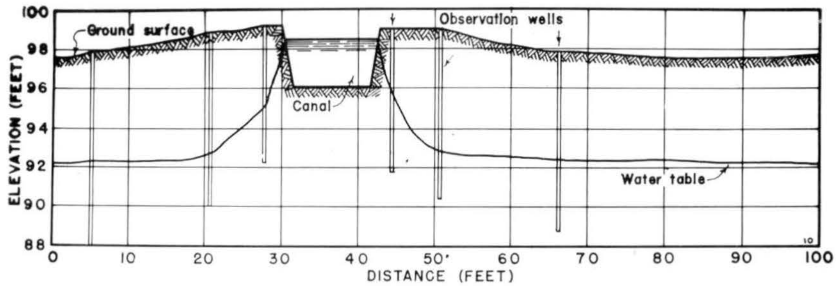

Figure 37.—Ground-water profile perpendicular to Lateral 17-B of Turlock District on Crow's Landing Road.

measurements at the intermediate gaging stations showed considerable variation. Profiles of the ground water made in the two sections indicated that most of the water was lost in the lower portion. Figure 37 shows the profile of the ground water in the lower section. No ground water was found in the upper section.

SUTTER-BUTTE CANAL COMPANY

Sutter-Butte Main

The original Sutter-Butte canal system, Gridley, California, was built in 1904 and 1905, but since that time extensive improvements and extensions have been completed. The portion of the canal under test, which extends from Cox spillway to the highway below the pumping station is, except for the portion below the pumping station, all in the original canal system, but has been considerably improved in recent years. A section of the main canal is shown in figure 38. The bottom width varied from 14 to 38 feet and the

Figure 38.—Portion of Sutter-Butte Canal, Gridley, California.

side slopes of 1½ to 1 were constant. The depth depended on the checks in the canal, which were only a short distance apart. The portion of the canal upon which the seepage measurements were made runs along the high ground near the Feather River through Madera and Gridley loam, undifferentiated. These are light-textured soils with a discontinuous hardpan substratum and in some sections a heavy clay subsoil. The seepage tests were divided into two groups; one with the canal checked up to the normal depth of operation and one with all the checks out. The results given in table 9, numbers 146 to 150, show very little effect from the change of depth, but this may have been due in part to the change of stage which occurred during gaging number 234 and made it impossible to complete test number 150. Profiles

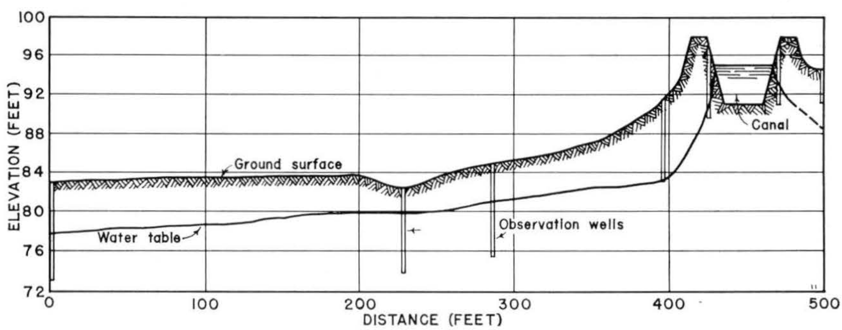

Figure 39.—Ground-water profile perpendicular to Sutter-Butte Canal at Hopkins Ranch.

of the ground water made near Hopkins Bridge, figure 39, at the upper end of the section, and near the Boynton check at the central portion of the section show that some seepage occurred at the upper station while very little occurred in the central portion. Computation of seepage based on the continuous record of the gage height, and the discharge measurements of group 1 checks the results of the individual determinations quite closely.

Green Lateral

This lateral is a part of the original Sutter-Butte system and extends from the end of the Belding Lateral to the highway running west from Gridley. The portion under test lies between station 21+01 and station 181+33. The bottom of the lateral was from 5 to 10 feet wide and the water was between 1 and 2 feet deep. It is in Stockton clay adobe throughout its entire length. This is an impervious clay soil with a calcareous hardpan substratum. The results of the seepage measurements given in table 9, numbers 151-156, were consistently small except for the first two tests which showed a much higher seepage loss than should be expected in this type of soil. This was due, in part, to the fact that a large change of stage occurred while the first two tests were being made. Borings made to determine the location of the ground water indicated that the loss from the lateral was small.

TABLE 10. *Summary of Seepage Measurements (Inflow-Outflow Method).*

Number of tests	Canal system	Canal	Section	Soil class	Lining	Average discharge	Average seepage loss: Per mile	Average seepage loss: Per sq. ft. of wetted area in 24 hr.
						c.f.s.	percent	cu. ft.
1	2	3	4	5	6	7	8	9
8	Lindsay-Strathmore	Lowline	Upper to lower station	Clay loam adobe	Gunite	41.24	-.0299	-.0153
4	Alta Irrigation District	Main	Head to second lining	Fine sandy loam & adobe		115.32	7.08	1.65
1	Alta Irrigation District	Main	Head to second lining	Fine sandy loam & adobe		620.78[1]	3.38[1]	3.81[1]
Continuous record.	Alta Irrigation District	Main	Head to second lining	Fine sandy loam & adobe		706.4	-.088	-0.77
2	Alta Irrigation District	Main	Head to first lining	Fine sandy loam		111.24	10.93	2.07
2	Alta Irrigation District	Main	First lining to second lining	Adobe		105.31	7.24	1.63
2	Alta Irrigation District	Main	First lining to above Campbell Ditch	Adobe		105.50	5.28	1.08
3	Alta Irrigation District	Main	Head to below Campbell Ditch	Loam and adobe		107.60	6.56	1.38
3	Alta Irrigation District	Main	First lining to below Campbell Ditch	Loam and adobe		101.15	.43	.09
2	Alta Irrigation District	Main	Below Campbell Ditch to second lining	Adobe		99.01	13.36	3.04
2	Alta Irrigation District	Main	Above to below Campbell Ditch	Adobe		96.37	10.11	2.23
6	Alta Irrigation District	East Branch	Orosi Lateral to Sand Creek	Sandy loam		53.34	-2.517	-.782
6	Gage	Main	Upper weir to lower weir	Loam and sandy loam	Concrete	24.54	.097	.037
Continuous record.	Gage	Main	Upper weir to lower weir	Loam and sandy loam	Concrete	24.9	.120	.047
6	Imperial Irrigation District	West side main	West drain to Thompson Crossing	Sand and clay		275.95	.059	.072
Continuous record.	Imperial Irrigation District	West side main	West drain to Thompson Crossing	Sand and clay		277.5	.111	.136
3	Imperial Irrigation District	Fillaree lateral	Head to second bridge	Sand and silty clay		6.04	2.96	.418
2	Imperial Irrigation District	Fillaree lateral	Head to first bridge	Silty clay		5.98	6.96	.873
2	Imperial Irrigation District	Fillaree lateral	First to second bridge	Sand		5.40	-.97	-.138
8	Merced Irrigation District	Yosemite LeGrand	Flume No. 1 to tunnel	Loam and adobe		54.89	.91	.335
4	Merced Irrigation District	Yosemite LeGrand	Farm Bridge to tunnel	Loam and adobe		97.74	.938	.551
Continuous record.	Merced Irrigation District	Yosemite LeGrand	Farm Bridge to tunnel	Loam and adobe		95.25	.687	.39
2	Merced Irrigation District	Yosemite LeGrand	Flume No. 1 to Flume No. 2	Loam and adobe		26.13	2.78	.559
4	Merced Irrigation District	Yosemite LeGrand	Flume No. 1 to Flume No. 2	Loam and adobe		56.56	1.22	.465
4	Merced Irrigation District	Yosemite LeGrand	Flume No. 2 to tunnel	Loam and adobe		51.64	-.398	-.139
1	Merced Irrigation District	Burchell Lateral	Burchell to Santa Fe Railroad	Loam and adobe		13.39	.450	.076

TABLE 10. Continued—*Summary of Seepage Measurements (Inflow-Outflow Method).*

Number of tests	Canal system	Canal	Section	Soil class	Lining	Average discharge	Average seepage loss: Per mile	Average seepage loss: Per sq. ft. of wetted area in 24 hr.
						c.f.s.	percent	cu. ft.
1	2	3	4	5	6	7	8	9
5	Anderson-Cottonwood Irrigation District	Main	Station 1 to Station 2	Gravelly loam		136.82	.791	.498
6	Anderson-Cottonwood Irrigation District	Main	Station 3 to Station 4	Loam and gavelly loam		70.30	.225	.104
2	Anderson-Cottonwood Irrigation District	Lateral No. 9	Green's Bridge to Wheeler's Bridge	Gravelly loam		8.32	35.1	5.31
1	Orland Project	Highline	End of lining to chute	Loam	Concrete	131.24	.728	.583
2	Orland Project	Lateral No. 8	Lined section to bridge	Sandy loam		13.90	41.3	8.24
2	Orland Project	Lateral No. 101	Check to lower check	Sandy loam		7.81	32.5	5.06
3	Orland Project	Lateral No. 211	Upper to lower weir	Sandy loam	Concrete	1.69	2.78	.133
3	Orland Project	Lateral No. 211	Upper to middle weir	Sandy loam	Concrete	1.69	2.47	.129
3	Orland Project	Lateral No. 211	Middle to lower weir	Sandy loam	Concrete	1.63	3.82	.150
3	Fresno Irrigation Dist.	Houghton	Grant Ave. to Westlawn Ave.	Sand and sandy loam		71.89	-1.09	-.394
Continuous record.	Fresno Irrigation Dist.	Houghton	Grant Ave. to Westlawn Ave.	Sand and sandy loam		64.7	.462	.150
4	Fresno Irrigation Dist.	Briggs Ditch	Head to Jensen Avenue	Loam and sandy loam		34.24	3.05	1.194
Continuous record.	Fresno Irrigation Dist.	Briggs Ditch	Head to Jensen Avenue	Loam and sandy loam		34.24	3.07	1.19
2	Fresno Irrigation Dist.	Briggs Ditch	Head to Golden Dawn	Loam and sandy loam		34.17	7.81	3.33
2	Fresno Irrigation Dist.	Briggs Ditch	Golden Dawn to Jensen Avenue	Loam and sandy loam		30.41	.617	.204
2	Turlock Irrigation Dist.	Highline	Head to East Avenue Bridge	Loam, sandy loam, clay loam		136.60	.343	.239
4	Turlock Irrigation Dist.	Lateral 17 B	Head to Section 17-20	Sand and sandy loam		17.43	12.00	2.61
Continuous record.	Turlock Irrigation Dist.	Lateral 17 B	Head to Section 17-20	Sand and sandy loam		17.14	11.96	2.56
2	Turlock Irrigation Dist.	Lateral 17 B	Head to Section 22	Sand and sandy loam		17.54	6.83	1.58
2	Turlock Irrigation Dist.	Lateral 17 B	Section 22 to Section 17-20	Sand and sandy loam		15.36	18.59	3.38
3	Sutter-Butte Canal Co.	Main	Cox Spillway to Pumping Station	Loam		208.96	1.22	.95
Continuous record.	Sutter-Butte Canal Co.	Main	Cox Spillway to Pumping Station	Loam		207.4	1.11	.86
1	Sutter-Butte Canal Co.	Main	Cox Spillway to Pumping Station	Loam		178.48	1.34	1.14
6	Sutter-Butte Canal Co.	Green Lateral	Flume to rating flume	Clay adobe		5.73	10.94	.595

[1] Two tests omitted.
Note: The negative signs indicate gains.

SUMMARY OF RESULTS OF SEEPAGE MEASUREMENTS

The results of the current-meter measurements of seepage from the canals given in table 9, and a summary given in table 10, show conclusively that the seepage losses from concrete-lined canals are small, but no less than the losses from some unlined canals. The soils in which the canals are excavated seem to have no consistent relation to the seepage losses. Both high and low rates of loss occurred in clay soils as well as in sandy soils. The seepage is apparently independent of the size of the canal although in percentage per mile, the laterals generally lose more than the main canals. Attempts to isolate the areas where the losses occurred by making measurements at intermediate points, usually gave erratic results. The disparity in the losses from the different sections of the same canal under approximately similar conditions is an indication of the limitations of the current-meter method of determining seepage losses.

The location of the ground water exerts a definite influence on the seepage from canals and in case the ground-water level is high, it may materially reduce the seepage or may even cause a gain. This is one of the reasons that seepage does not vary in direct relation to the depth of water in the canal. Hardpan and layers of impervious material beneath the canal bed also affect the seepage. It is evident from the foregoing analysis that no generalizations can be made regarding seepage from lined and unlined canals or from canals in various types of soil. Apparently the only safe practice in reaching any conclusion regarding seepage from a channel is to make seepage measurements.

CONCLUSIONS

Of the 125,000 miles of canals and laterals used for irrigation in the 17 Western States less than 5,000 miles had been lined by 1939, although the Census records show that 38 percent of all the water diverted for irrigation was lost before it reached the point of delivery to the farm.

Seepage as here used is restricted to the movement of water into or out of irrigation channels through the bed material. The amount of seepage may be measured in cubic feet per square foot of water surface or per square foot of wetted surface, in cubic feet per mile of channel and in percentage of total flow per mile.

The permeability of the material forming the lining of the canal, whether it be natural soil, a deposit of silt or an artificial lining, is under most circumstances the dominant factor in determining the rate of seepage. Theoretically, the head and the temperature affect the seepage rate, but the influence of these factors is frequently overshadowed by the errors and uncertainties in seepage measurements and the changes in pressure in the soil which, according to Darcy's law, have as much influence as the depth of water.

Seepage losses are determined by measuring the inflow and the outflow from a section of the channel, by noting the drop in the water surface of a pool formed by damming off a portion of the channel, by measuring the drop of the water surface in pits and trenches excavated in the canal or in the ground adjacent to it, and by permeameter observations on material in place in the canal or on samples taken from the canal without changing the structure of the material.

The current meter is best adapted for making inflow and outflow measurements in determining seepage, but it is usually not sufficiently accurate to measure the loss from lined canals or canals in soils of low permeability. Under these circumstances, the pool method yields the most satisfactory results. Measurement of the seepage from pits and trenches is satisfactory only for making rough estimates. Permeameters have been used to a limited extent, but the results obtained have been erratic. Further study of permeameters is recommended.

The results of observations with the seepage-cup permeameter on seepage indicate that it is possible to measure seepage in the specific area covered by the cup, but they indicate also that this device is subject to serious limitations.

Two methods have been developed for converting the seepage-cup observations into the equivalent canal seepage. The method based on formulas developed from a theoretical analysis of seepage phenomena according to Darcy's law, is an accurate way of converting the observations into canal seepage. The graphical method is simpler, but it is also less accurate. However, the errors introduced from this source are unimportant in comparison with the unavoidable experimental errors. More experimental data will have to be obtained before the practicability of this device can be determined.

The observations on the seepage from pits and trenches in two different types of soil, which were made in attempting to segregate side and bottom seepage by mathematical analysis of the experimental data, did not yield conclusive results.

The results of the attempt to separate side and bottom seepage by installing metal partitions in a trench were also inconclusive. These tests did disclose, however, that seepage is not a constant quantity. It increases or decreases with changes in conditions that frequently are not apparent to the observer. Furthermore, these tests demonstrated the efficacy of the Mariotte control apparatus in maintaining constant water levels. Although changes in pressure in the soil were disregarded, fairly consistent results were obtained on the effect of head on the seepage from various parts of the trench. These tests showed, also, the effect of having higher heads in some parts of the trench than in others.

Measurements of seepage based on the drop of the water surface in pools in the canal were in general more consistent than those made by measuring the inflow and the outflow with current meters. In fact, very low rates of seepage could be measured accurately by this method if there were no large diversions or leaks in the section under test.

The seepage measurements based on the observations on the drop in the water surface in pools show that high and low rates may occur in both lined and unlined canals. In some cases the losses from canals in sandy soils were less than the losses from canals in heavy soils.

Seepage losses from concrete-lined canals ranged from less than 0.1 to more than 3 cubic feet per square foot per 24 hours. The losses from some canals in heavy clay were also less than 0.1 cubic foot per square foot per 24 hours and the losses from some canals in very sandy soils exceeded 25 cubic feet per square foot per 24 hours.

Seepage measurements made by measuring the inflow and outflow with current meters should be conducted during periods when the stage of the canal is constant or changes in bank and channel storage may seriously affect the results. Where register records are used in conjunction with the current-meter measurements, the stage need not remain constant because the period covered can be chosen so that the stage is the same at the beginning and the end.

The current-meter measurements show that the seepage losses are smaller than generally anticipated and that there is no consistent relation between type of soil and seepage; other factors such as ground-water levels and silt accumulations may overshadow the effect of soil type. There is a wide range of rates of seepage from lined canals, which seems to indicate that the quality of the lining is an important factor.

The wide range of rates of seepage found by pool measurements and inflow and outflow measurements with current meters indicates the desirability of making actual seepage measurements whenever it is necessary to know how much seepage is occurring in a canal.

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