REUSE OF SURFACE RUNOFF FROM FURROW IRRIGATION

By

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INTRODUCTION

The reuse of surface runoff from furrow irrigation is becoming an important part of an irrigation system. In some areas reuse of runoff from irrigation is mandatory. Even in areas without such laws, the farmer may risk legal action if he allows excessive runoff. Reuse systems are more commonly installed for economic reasons. Many times runoff water can be applied to the field at a lower cost than pumped or diverted water; moreover, water application efficiency may be improved if the system is properly designed. In other cases, the reuse of surface runoff from irrigation may be essential to prolong the life of the groundwater supply.

Davis (5) described a reuse system as an integral part of an irrigation system which is designed to achieve an economic balance between water, labor, capital, power, and land resources. He stated that if the cost or availability of labor and capital are greater than the cost of water, a farmer may be forced to sacrifice water as a substitute for labor and equipment. Reuse of irrigation water may be more economical than the use of additional labor or equipment to increase the efficiency of the system.

Whether a reuse system is installed for legal, economic, or conservation reasons, there exists a need for better design procedures.

A major problem in the design of reuse systems is the inability to estimate the amount and time distribution of runoff from furrow irrigation. There is little runoff data on which to base the design of reuse systems. To obtain the most economical reuse system, it is necessary to determine the optimum relationship of storage size and pumping rate based on the expected runoff and existing conditions for a given irrigation system.

Objectives

1. To determine the amount and time distribution of surface runoff from several furrow irrigated fields.
2. To determine the optimum relationship of storage size and pumping system capacity based on objective number 1.
3. To design systems to recirculate runoff water to the upper end of the field from which it occurs.
4. To determine the economic feasibility of recirculating the runoff water.

Limitations of the Study

The study was limited to furrow irrigation using gated pipe distribution systems with pumped wells as the water source. Data were collected from six irrigated fields with crops of corn or milo. Row lengths of 1/4 and 1/2 mile were studied. Each field was operated by a different farm manager.

CHAPTER II

REVIEW OF LITERATURE

In a response to the increasing need for irrigation reuse systems, several researchers have studied the use and design of such systems. This chapter contains information concerning the types of reuse systems being used, system functional analysis and design considerations, and a summary of the reported amounts of runoff from furrow irrigation.

Types of Reuse Systems

Bondurant (2) classified reuse systems according to the method of handling runoff water as follows: if the water is returned to a field lying at a higher elevation, it is usually referred to as a return-flow system; if the water is applied to a lower lying field, this is termed sequence use. The author also classified systems according to storage capacity. Systems which store collected runoff water are referred to as reservoir systems. Systems which immediately return the runoff water require little storage and are termed cycling-sump systems.

Irrigation farmers in the High Plains area of Texas are successfully using modified playa lakes to store and reuse runoff water, which is commonly called tailwater. Many advantages for reuse of runoff water from playa lakes and tailwater pits are given (4).

Bondurant and Willardson (3) conducted a study of recirculating systems in Southern Idaho and concluded that many of the systems could have benefited from better design. Reservoir systems were the most common type found in the survey.

Erie (6) suggests pumpback systems to minimize the extra effort and expense that is usually required to obtain high field application efficiencies and even distribution of water.

Davis (5) states that the size of the sump depends on the value of the land upon which the sump is constructed and on the desired control of water at the point at which the tailwater is returned. He suggests that if the irrigation distribution system utilizes a regulating reservoir or a concrete pipe line, a small sump with a rapidly cycling pump is satisfactory; but, if a head ditch and siphons or a head ditch and small overflow structures are used, a large sump should be recommended to insure a steady flow rate when the tailwater system is in operation.

Functional Analysis and Design Considerations

Larson and Allred (10) developed an expression for a pump drainage system relating inflow, pump capacity, sump volume and cycle time. This expression is:

$$SC = \frac{60\ I}{P}(P - I) \qquad (2\text{-}1)$$

where I and P are inflow and pumping rate, respectively, in gallons per minute, S is sump storage in gallons between start and stop levels, and C is the number of pumping cycles per hour.

When the inflow to a sump is zero or $I = P$, no cycling will occur and no storage is required. If equation (2-1) is differentiated with respect to I, it is found that maximum storage occurs when $I = 1/2\ P$. Substituting $I = 1/2\ P$ in equation (2-1) results in equation (2-2).

$$S = \frac{15\ P}{C} \tag{2-2}$$

By letting C equal the maximum allowable cycles per hour, the storage obtained from equation (2-2) will be a minimum.

Davis (5) applied these equations to the design of irrigation reuse systems. He described the use of various sizes of sumps and pumps for the reuse of irrigation runoff. Davis recommended the use of 15 cycles per hour in equation (2-2).

Larson and Manbeck (11) studied the effect of cycle length on pumping plant efficiency. They suggested a design cycle length of 4 to 8 minutes with a median value of 6 minutes or 10 cycles per hour recommended for typical farm use.

Davis (5) noted that the fluctuating and rather low flow from cycling systems may preclude its efficient use on some fields. For these fields, he recommended the use of a continuous pumping operation. The most flexible system would let the storage volume equal the total volume of runoff.

Bondurant (2) also did some work on the design or recirculating irrigation systems. According to his functional analysis, a reuse system should function in the following manner to accomplish its design purpose.

1. Runoff water should be applied to a different field or portion of the field than that on which runoff occurs. Recirculating runoff to the same irrigation set that is generating runoff results only in temporarily storing water on the field. This will not increase the infiltration rate, but will increase the rate of runoff and will probably increase erosion in the furrow.

2. When computed over the time interval required to irrigate the area contributing to the recirculating system, runoff water will have to be returned to the system at the same rate that it is accumulated if all runoff is to be reused. If temporary storage is provided, stored runoff will eventu-

ally have to be recirculated at a rate equal to storage accumulation to prevent loss by overflow.

3. Maximum improvement in total water use on the farm will result from using stored runoff water to achieve a reduced stream size for cutback irrigation; i.e., stored runoff water is pumped to increase the stream size during the advance period and pumping is stopped after the field has started to produce runoff. This reduces deep percolation and runoff so that a minimum amount of water must be recirculated. Runoff water collected from one irrigation set is returned to the head ditch and applied with the normal inflow on the next irrigation set.

Bondurant (2) developed some equations to express a volume balance for a reuse system where the runoff water is collected and used on the next set. His equations are as follows:

$$V_s = C_1 V_a + (n - 1)(C_2 - C_3) V_a \qquad (2\text{-}3)$$

where V_s is the volume of runoff water in storage after any irrigation set, V_a is the volume of water applied per set, n is the number of irrigation sets, C_1 is the ratio of amount of runoff to amount of applied water for the first irrigation set, C_2 is the same ratio for subsequent irrigation sets, and C_3 is the ratio of amount of water pumped from stored runoff to amount of applied water.

So that the rate at which water is pumped from storage can be determined, equation (2-3) may be restated in terms of flow rates:

$$V_s = C_1 q_o t_a + (n - 1)(C_2 - C_3)(q_o + q_p r_1) t_a \qquad (2\text{-}4)$$

where q_o is the rate at which water is diverted from external sources, q_p is the rate at which water is pumped from stored runoff, r_1 is the ratio of time stored runoff water is pumped to total time of application, and t_a is the total time of application.

The rate at which water is initially applied to the field for the second and succeeding sets is:

$$q_a = q_o + q_p = q_o (1 + C_4) \qquad (2\text{-}5)$$

where C_4 is the ratio q_p/q_o and is determined by field trial or analysis of existing irrigation practice.

The volume of water pumped from stored runoff is:

$$V_p = q_p \, r_1 \, t_a \qquad (2\text{-}6)$$

The volume of water applied during the first set will be less than that applied on succeeding sets as well as the area irrigated if the same furrow stream size is used. Therefore,

$$V_a = V_o \qquad \text{for } n = 1 \qquad (2\text{-}7)$$

$$V_a = V_o + V_p = q_o \, t_a + q_p \, r_1 \, t_a \qquad \text{for } n > 1 \qquad (2\text{-}8)$$

where V_o is the volume of water per set delivered from the primary source such as a well or canal.

C_3 may now be determined as:

$$C_3 = \frac{V_p}{V_s} \qquad (2\text{-}9)$$

Bondurant (2) stated that the data needed to design a reuse system for a given farm are the topographic features of the farm and an estimate of the amount of runoff water to be handled. He presented a graphical technique which required data on the intake rate and stream advance for the particular field.

Another technique for estimating the amount of runoff is presented by Willardson and Bishop (18). This method also requires data on the intake rate of the soil and the rate of advance of the furrow stream down the furrow, as well as the design depth of irrigation and the physical dimensions of the field. Their method predicts a minimum of about 20 percent runoff from fields with stream advance to total irrigation time ratios approximating 0.20, if nonreduced stream flows are used.

Amount of Runoff

A five year study of three large farm areas in the Rupert, Idaho region showed an average farm runoff of 18.5 percent of the total water delivered to the farm (17). Each of the three areas was newly developed when the study began. Portneuf silt loam soils in the area were deep, fertile and well-drained.

From surveys made in California, Davis (5) reported 10 to 20 percent runoff from farms averaging 160 acres in size.

Shockley (14) reported surface runoff losses of 35 percent from a field with 660 foot long rows and 12 hour sets when applying 5.67 inches of water.

Marsh (13) reported an average runoff of 31 percent of the water applied during 32 separate measurements between 1941 and 1953.

Bondurant (2) reported that a southern Idaho farm of 105 irrigatable acres produced an average runoff of 11.6 percent of the water applied.

Cost Analysis

Davis (5) estimates annual costs for tailwater systems surveyed in California would be nothing for a gravity drain, $1.50 per acre-foot pumped to a nearby outlet or field, and $3.00 per acre-foot for pumping back into the upper end of the same field.

Bondurant (2) presented a cost analysis for delivering runoff water to a lower ditch and to the farm delivery point for different pipe sizes and pumping rates. His study showed that the most economical total annual cost for pumping 1.0 cfs 1600 feet to the farm delivery point would be $400 using 8 inch pipe. This was based on steel pipe, an electrically driven centrifugal pump, and 6 percent interest over a 15 year expected equipment life. The corresponding total annual cost for an equal pumping rate through a 750 foot pipeline to a lower ditch was $225. Total annual costs were also given for several other pumping rates.

Halderman (8) presented an example cost analysis for an irrigation tailwater system. The system had a 450 gpm pump with a 5 horsepower electric motor, 2 acre-feet storage capacity, 2000 feet of 8-inch asbestos cement pipe, and included construction and installation cost. Total annual cost was $5 per acre-foot of water using a 20 year life and 7 percent interest.

CHAPTER III

MEASUREMENT EQUIPMENT

Selection of Equipment

The principal equipment needed for the study was instrumentation to measure the amount of runoff from the irrigated fields. For this purpose, instruments were needed that would measure the water over a wide range of flow rates with the best accuracy possible. A continuous record of the runoff was necessary with as little personal supervision as possible to facilitate the measurement of several fields at one time. A temporary measuring device which would adapt to field conditions and operate in all weather conditions was required.

Type H flumes instrumented with water level recorders were selected for the measurement devices. These systems were accurate over a wide range of flows, were easily installed, and offered the best service for the least amount of attention.

H Flume Construction

Three type H flumes were available which had been individually calibrated from previous work by Sweeten (16). These flumes had approximate head and discharge capacities of 0.75 feet and 450 gpm, respectively.

In addition, three 1.0 foot H flumes and their forebays were constructed and calibrated in the Agricultural Engineering Laboratory.

Each of these flumes had a capacity of approximately 900 gpm. These flumes had forebays 4 feet long with a cross section of 14 x 22.8 inches. The forebays were built of 24 gauge galvanized sheet metal nailed to a frame of 2 x 4 inch lumber with solder covering the nail holes. The H flumes were constructed according to the standard dimensions given in Agricultural Handbook No. 224 (1) using 20 gauge galvanized sheet metal. Figure 1 shows an H flume ready for assembly.

Calibration of H Flumes

The experimental procedures involved in obtaining head versus discharge data were essentially the same for each H flume.

The laboratory calibration was obtained by pumping water from a 1350 gallon capacity sump through a piping system into an entrance flume attached to the H flume which discharged back into the sump.

Two water supply pipeline systems were used. A 2 inch pipeline was used for discharges up to 100 gpm and a 6 inch pipeline was used for flows above 100 gpm.

The pumping system consisted of a 1/2 horsepower motor driven Bell and Cossett centrifugal pump connected to the 2 inch pipe and a 7-1/2 horsepower motor driven Berkeley centrifugal pump connected to the 6 inch line.

Small inflow into the system was measured with a 2 inch nutating disk water meter calibrated by weight and time measurements. Large inflow from 100 to 900 gpm was measured with a Sparling meter which was calibrated with a sharp edge orifice and U-tube manometer at the Outdoor Hydraulic Laboratory near Stillwater, Oklahoma. The Sparling meter was installed in the 6 inch pipeline.

The entrance flume had a cross section similar to that of the H flume and contained a baffle to dissipate excessive turbulence. The H flume also contained a fin type baffle, which was later used in the field measurements to straighten the flow lines.

The bottoms of the H flume and forebay were surveyed using a point gage and an engineer's level. Necessary vertical adjustments were made to insure that the H flume and forebay were positioned correctly.

For calibration of the H flumes, inflows of about 3 to 875 gpm were initiated in the test flume by regulation of the respective gate valves. Several water surface readings were taken at each increment of flow rate to insure stability. The discharge rates were calculated as the difference between the initial and final volumetric readings recorded in gallons through the water meter divided by the time increment in seconds between these readings as measured with a stop watch. Readings were taken over time periods of from 5 to 15 minutes. Figure 2 shows an H flume being calibrated.

Gage Zero

Measurements of head were taken with a point gage mounted on the stilling well of the H flume. A gage zero was established with an engineer's level using the following procedure.

1. A level reading A_1, as shown in Figure 3, was taken on the lip of the H flume.

2. The point gage was placed in its bracket at the stilling well of the H flume and a convenient foresight Z was established. At the same time, a corresponding vernier reading, A_2, was recorded.

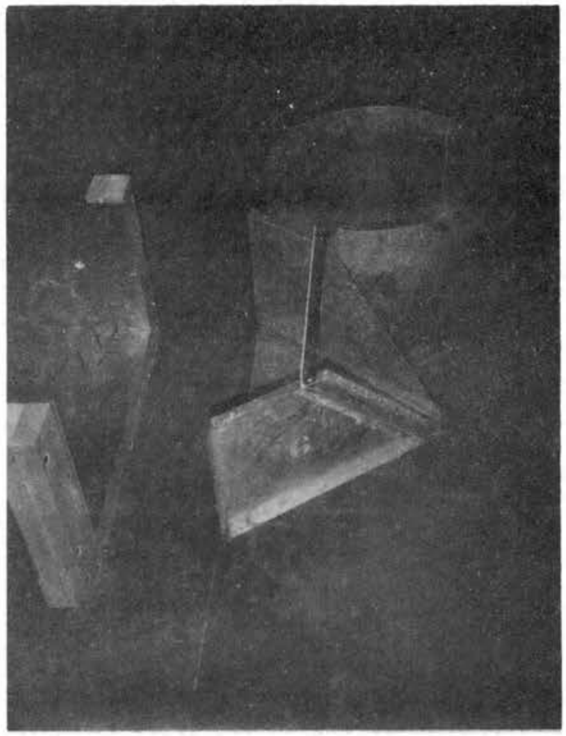

Figure 1. Construction of an
H Flume

Figure 2. Entrance Channel, Pumps, and Sump
Used for H Flume Calibration

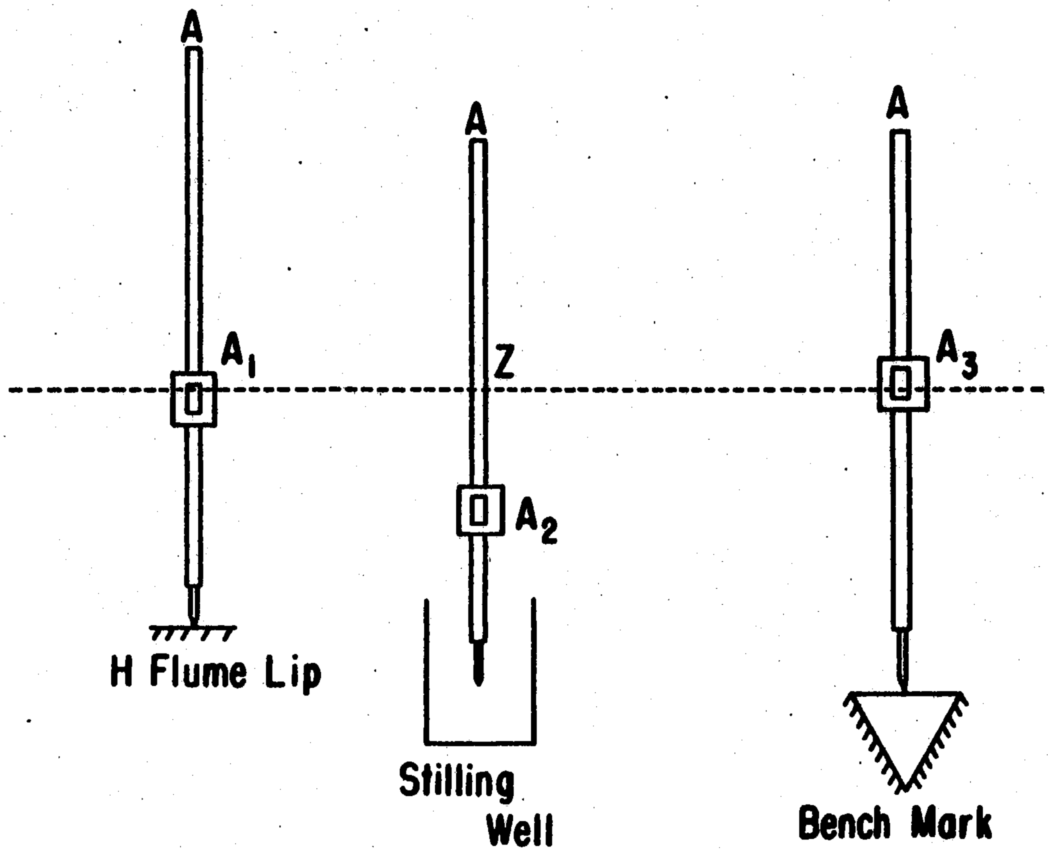

Figure 3. Gage Zero Determination

3. Gage zero was calculated as:

$$\text{Gage Zero} = A_2 - (Z - A_1).$$

4. The head at any given flow depth was then the vernier reading at which the point gage touched the water minus gage zero.

5. A level reading was taken on a nonyielding support known as the bench mark so that any change in lip elevation could be detected.

Results of H Flume Calibration

The 1.0 foot H flumes were calibrated for discharges ranging from about 3 to 900 gpm. The data were divided into groups according to the head which best fit a linear relationship of $\log_{10}Q$ versus $\log_{10}h$, where Q is the H flume discharge and h is the head. By linear regression using a Digital computer, three equations of the following type were obtained for each flume:

$$Q = a\,h^b \tag{3-1}$$

where a and b are the intercept and the slope of the curve, respectively. The coefficient, exponent, standard deviation, and correlation coefficient for each of these relationships are presented in Appendix A. The value of h at the intersection of adjacent equations was found by solving the equations simultaneously.

Water Level Recorders

Model A-35 Stevens water level recorders were used to obtain a continuous permanent record of outflow.

The recorders contained a strip chart driven by a mechanical clock.

The recorders had the capability to operate for over three months without changing the chart or rewinding the clock. A magnified gage scale of 2:1 was used allowing two inches of vertical pin movement on the chart to one inch of float travel. The recorder ink pin reversed directions at the edge of the 10 inch wide chart to accommodate the large range of depth needed. Three of the recorders had chart speeds of 9.6 inches per day so that each 0.1 inch division of chart paper represented a 15 minute time interval. The other three recorders had a chart speed exactly one-half as fast so that a corresponding division represented a 30 minute time interval. Both gear ratios operated satisfactorily, although the faster speed allowed a slight advantage in accuracy.

The recorders were housed in insulated boxes mounted on the H flumes to protect them from theft and the weather. The 20 x 28 x 13 inch boxes were constructed with 1/8 inch thick Aluminum sheet metal supported by small structural steel angles. A slot was cut in the wooden floor of each box to accommodate the float cable which extended from the float over the recorder pulley and to a counterweight. Sixteen inch diameter floats were used.

One inch thick styrofoam was used to insulate the top and sides of the boxes. Figure 4 shows one of the water level recorders mounted in the insulated box.

The Aluminum box containing the recorder was supported over the stilling well of the H flume with structural steel angles which were braced to prevent intolerable wind vibration.

Calibration of Inflow Meters

A 10 inch propeller type Sparling meter and a 12 inch Aluminum model of the same type were used for measurement of inflow. These meters were calibrated in the laboratory to insure that accurate measurements were made.

Much of the same equipment and procedures used to calibrate the H flumes were also used in the inflow meter calibration. Since the meters were calibrated for flows above 200 gpm, only the 6 inch centrifugal pump and pipeline were needed.

The meters were connected to the pipeline by means of a vinyl coated neoprene sleeve. This was necessary to dissipate the high velocity surging flow from the pipeline which would affect the flow measurement of a propeller type meter. A sleeve of at least 10 diameters in length was necessary to satisfy this requirement. The sleeve used was 10 feet long and had a 16 inch diameter which satisfied the requirement for both meters.

The meters discharged freely into the air at an inclined angle of about 45 degrees upward, as shown in Figure 5.

During calibration the meters were positioned at a slight adverse slope to insure that all air was out of the system. Increasing the slope above this level had no effect on the flow rate.

For calibration of the meters, inflows were incremented at approximately 50 gpm from 200 to 950 gpm. The Sparling flow meter in the 6 inch pipeline, which had been previously calibrated as described earlier, was used as a reference. With the aid of stop watches and the volume totalizer dials of each meter, discharge readings were taken simultaneously for the meters at each increment.

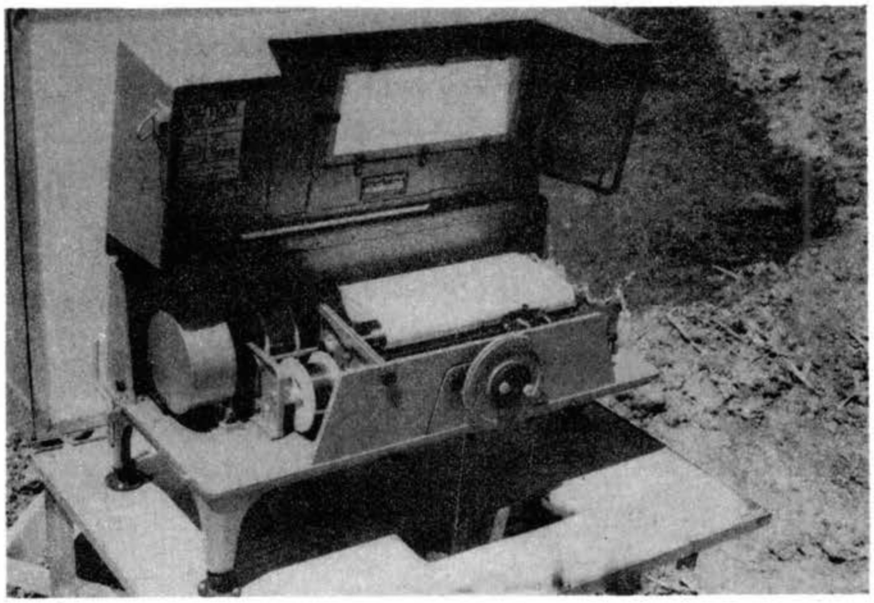

Figure 4. Stevens A-35 Recorder Showing Strip Chart and Mechanical Clock

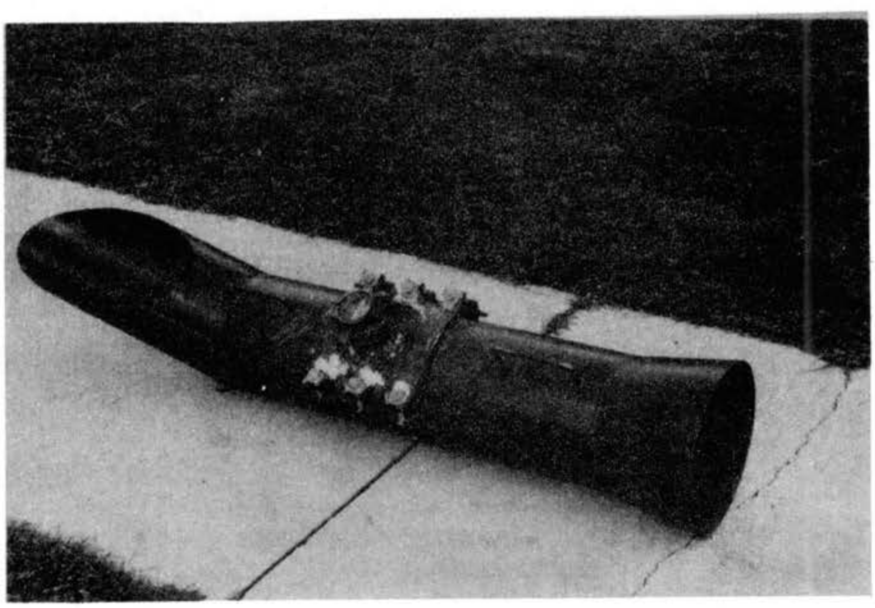

Figure 5. Sparling Propeller Type Flow Meter

The discharge readings obtained from the inflow meter were then plotted on arithmetic graph paper versus the corresponding corrected flow readings obtained from the lab meter in the 6 inch line. This data produced a straight line from which the actual inflow meter readings could be predicted at any flow rate in the range of the calibration.

CHAPTER IV

METHOD AND PROCEDURE

Selection of Location

Six fields in the Panhandle of Oklahoma were selected for study. The fields were near Guymon, in Texas County. This area was chosen because of the extensive use of furrow irrigation in the area. A large percentage of the land in the area is suitable for furrow irrigation with only moderate land leveling required. The soil type of all six farms was Richfield-Ulysses, a deep hardland soil (15).

Water in the area is supplied by the Ogallala formation which is a rich sandy deposit lying near the surface in most of the entire region. It is 200 to 300 feet thick (9).

The selection of the specific farms for study by a completely randomized method was precluded by several factors. The permission and cooperation of the individual farmer was essential, but generally not a problem. However, it was necessary for the runoff to drain to a common point on the farm so that it could all be measured, as shown in Figure 6. It was also desired to instrument farms with a constant water source. Table I summarizes some of the pertinent information about each farm.

TABLE I

DESCRIPTION OF STATIONS

Station No.	Farm Size (acres)	Row Spacing (inches)	Row Length (feet)	Average Slope (percent)	Well Yield (gpm)	Crop
1	80	56	1320	.337	930	Milo
2	150	56	2640	.33	960*	Milo
3	76	56	2640	.365	1075	Corn and Milo
4	130	60	2640	.14	1750**	Corn
5	152	56	2640	.33	1575	Corn
6	110	40	2640	.33	750	Corn and Milo

* An additional well was sometimes used which increased the flow to 1700 gpm.

** The flow was reduced to 700 gpm during part of the season.

Measurement of Inflow

Two propeller type Sparling meters were used for the measurement of the well yield. The meters operated equally well and the use of both was strictly for convenience and availability purposes. These meters were calibrated in the laboratory as previously described to insure accurate measurement.

The inflow for each farm was measured early in the irrigation season and again later in the season, in most cases, to check for any possible change in well yield. Three of the well power plants were governor controlled which helped to maintain a constant flow rate. The other three well power plants had tachometers which were used to keep the pump speed constant. A typical irrigation well is shown in Figure 7.

Installation of H Flumes

The exact location of the H flume on each farm was chosen such that a gentle slope in the drainage ditch would precede the H flume. This would eliminate the problem of excessive velocities associated with steep slopes which would alter the calibration of the H flume or carry large quantities of silt with the runoff water.

The support system used to hold each H flume solidly in position was four wooden posts buried approximately 2-1/2 feet in the ground. A frame, made of large steel angles bolted to the posts, supported the H flume which was held to the frame with lag bolts.

The flow from the runoff drainage ditch was funneled into the flume by using a piece of vinyl covered neoprene, as shown in Figure 8. This material was strong and completely rot proof.

A gage zero was determined for each H flume using the procedure

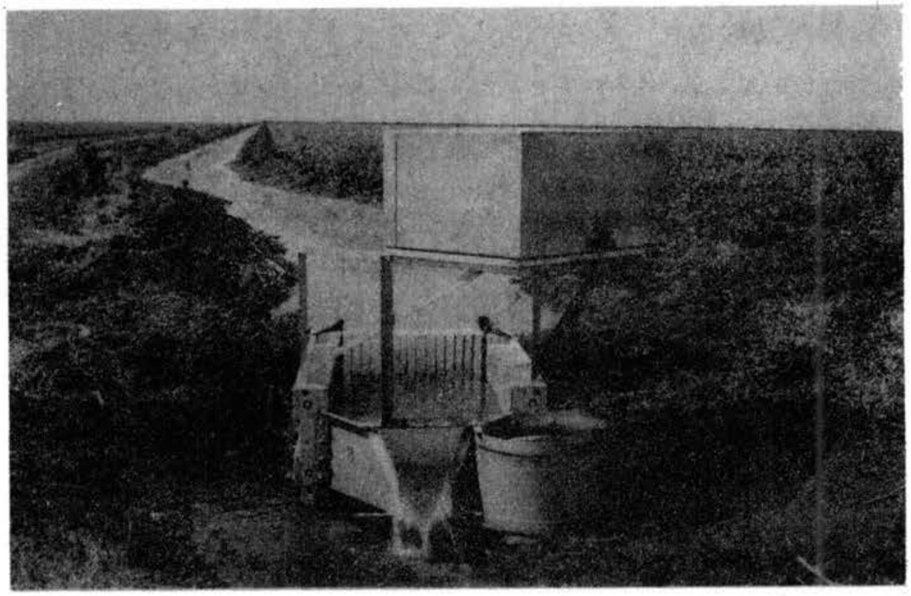

Figure 6. Measuring Runoff as It Leaves the Lower Corner of the Field

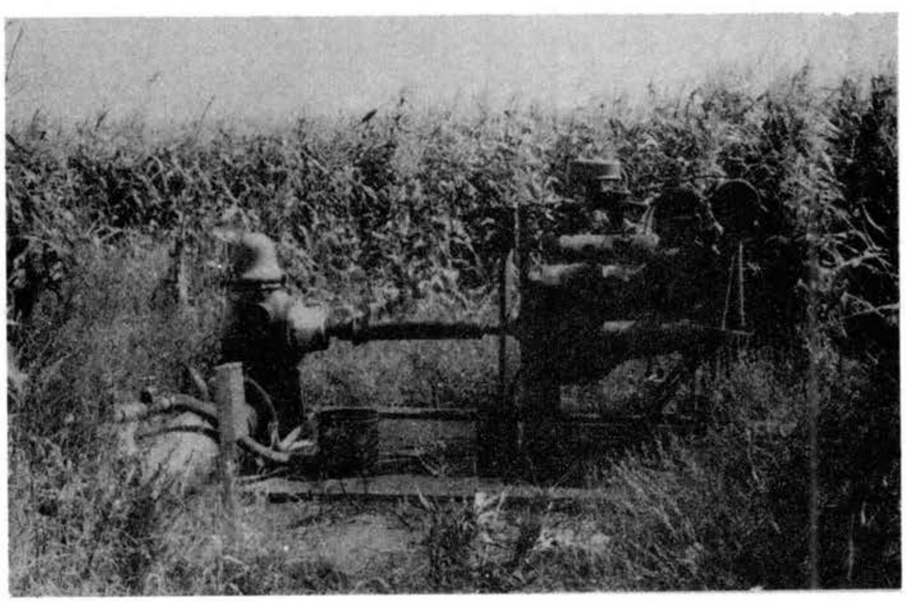

Figure 7. Typical Irrigation Well with Waterproof Record Box Near the Pump

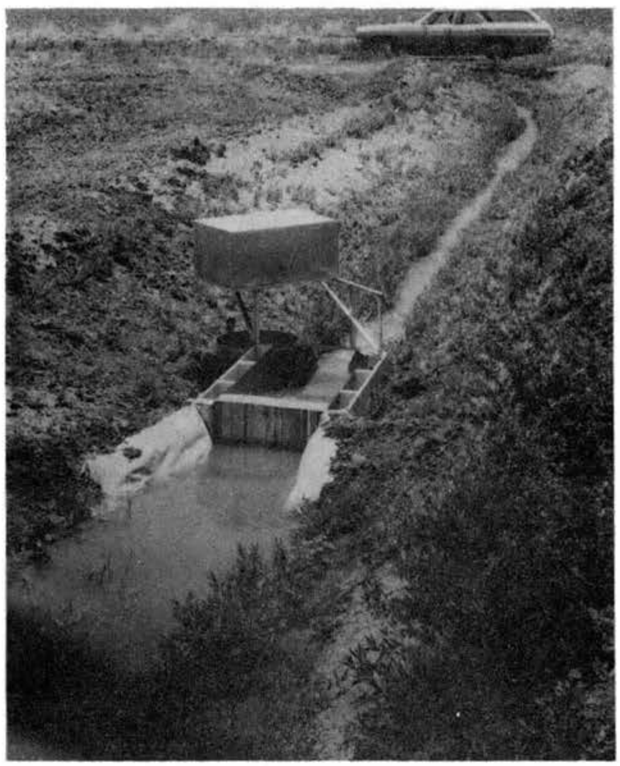

Figure 8. Vinyl Coated Neoprene Used to Funnel Water Through the H Flumes

previously described. Each water level recorder was adjusted so that the depth of flow indicated would agree with the actual head in the H flume as measured with the point gage. The point gage and water level recorder for a typical H flume installation are shown in Figure 9.

A nonyielding object was selected as a bench mark at each location so that any change in elevation of the H flume lip could be detected.

Data Collection

The surface runoff which occurred as a result of the furrow irrigation of six fields was measured. Although six fields could not be considered a complete sampling of all irrigation practice, it was hoped that a typical representation for farms in the study area could be obtained. Unlike a situation in which the investigator controls all experimental procedures, these data were obtained in an actual field situation where the farm manager for each location operated his respective irrigation system according to his normal practice. The data is thought to represent an unbiased sample of typical irrigation practice in the study area.

The runoff stations were usually checked daily to prevent any abnormalities in the data. Trash accumulation in the flumes was a problem at some stations. Trash guards were installed upstream from the H flumes to control this problem.

Since constant attendance at all of the stations was impossible, each farmer was asked to record pertinent information about his daily operation in a notebook provided for this purpose. A waterproof box containing a record book was conveniently located near the pumping plant at each station. The farmer was asked to record such things as

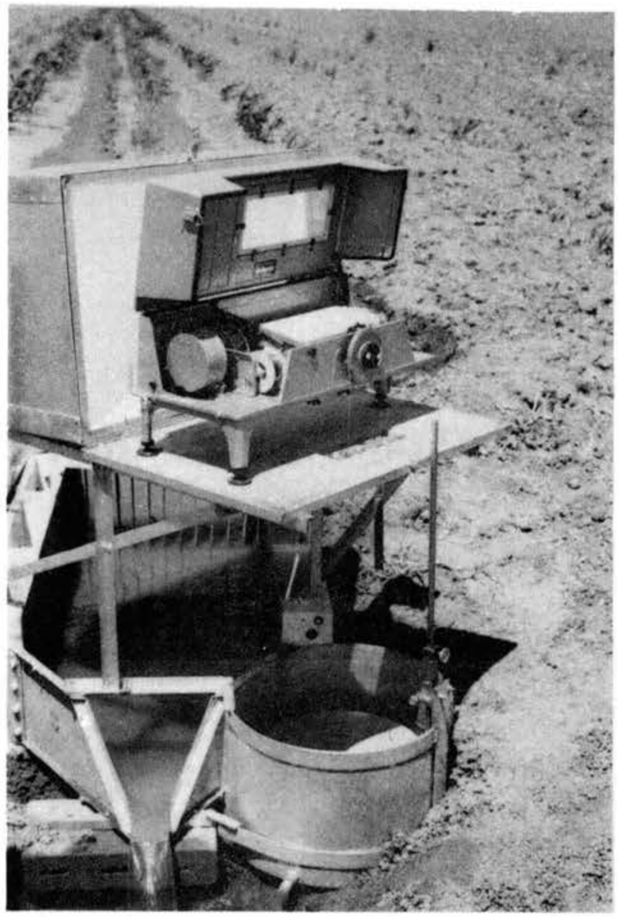

Figure 9. The Strip Chart Head Reading Was Periodically Verified with the Point Gage Head Reading

the date, time, and number of rows for each change of irrigation set. He was also asked to record rainfall and any time that the well was not pumping or pumping at a reduced rate.

CHAPTER V

PRESENTATION AND ANALYSIS OF DATA

The volume, average rate, and time distribution of surface runoff were of specific interest in the analysis of data. The volume of runoff is a function of several variables. The variables measured were the water application rate, row length, furrow spacing and field slope. The application time and number of rows were recorded for each irrigation set.

Other variables which may affect the volume of runoff from furrow irrigation are the variation in type of soil, the condition of the furrows throughout the season, the variation in moisture content before each irrigation, the water use rate of the crop, climatic factors, and the uniformity of flow to individual rows in an irrigation set. The measurement and exact relationship of all of these variables were not within the scope of the study.

The volume of water applied was figured as the product of the application rate in acre inches per hour and application time in hours. The area covered during this time interval, whether on an individual set basis or over several sets, was figured from the row length, row spacing, and number of rows covered. The depth of application, in inches, was then the acre inches applied divided by the acres covered.

Analysis Procedure

Measurement of the runoff from each field was accomplished with an H flume and water level recorder. The pin trace on the strip charts of the water level recorders formed a continuous record of time versus head in the H flume measuring devices. Head readings were converted to flow rates at 15 minute time intervals to obtain a hydrograph of time versus flow rate for each irrigation set. Calculations were made with a digital computer.

The head readings were recorded from the time runoff began until a time period equal to the total application time had been covered. This application time was generally from the time the irrigation well was started until pumping was stopped so that the total time of runoff was equal to the total pumping time. Data collected during periods of rainfall could not accurately be separated and were not used.

Although the head readings at each 15 minute time interval were taken continuously, the data were separated into individual sets and identified with the corresponding daily records describing each set. The application time, number of rows and calculations describing the individual set data are presented in Appendix B for each station except Number 3 for which no individual set data were available.

Some data could not be separated into individual sets. The separation problem usually occurred when a number of rows from one set had not watered completely through the field and the farm manager elected to water them for an additional set.

Quantity of Runoff

Table II presents the average depth of application, depth of runoff, and runoff percent for each station including both the data analized by individual sets and that not analyzed by individual sets. The runoff data from farms with two possible well flow combinations were analyzed separately.

The data presented in Table II for Stations 2, 3, 5, and 6 were plotted on log-log paper and an equation of the form

$$Y = a X^{b} \tag{5-1}$$

developed since they had nearly equal slopes and row lengths as previously shown in Table I. A least squares technique was used to fit Equation (5-2) where Y is the depth of runoff volume in inches and X is the average depth of application, also expressed in inches. The data points and Equation (5-2) are shown in Figure 10.

$$Y = 0.0044 X^{4.027} \tag{5-2}$$

Equation (5-2) can be used to predict the average depth of runoff as a function of the average application depth, within the range of the empirical data, for fields of similar characteristics.

The volume of runoff from the individual sets of each field varied considerably. A portion of this variation can be attributed to the set application time and another variable such as the size of set or average flow per row. Correlation coefficients for these relationships ranged from 0.724 to 0.095 with large standard deviations and percent deviations for the individual observations. The rest of the variation in runoff from the individual sets can be attributed to:

TABLE II

SUMMARY OF RUNOFF DATA

Station Number	Well Yield gpm	Amount of Data hours	Average Application Depth inches	Average Runoff Depth inches	Percent Runoff
1	930	277.75	5.236	1.085	20.72
2-A	1700	405.50	2.154	0.091	4.21
2-B	960	311.25	2.158	0.102	4.71
3	1075	443.00	3.105	0.431	13.90
4-A	1750	158.50	3.671	0.961	26.17
4-B	700	429.00	3.093	0.871	28.17
5	1575	474.50	3.276	0.507	15.47
6	725	587.00	3.508	0.686	19.56

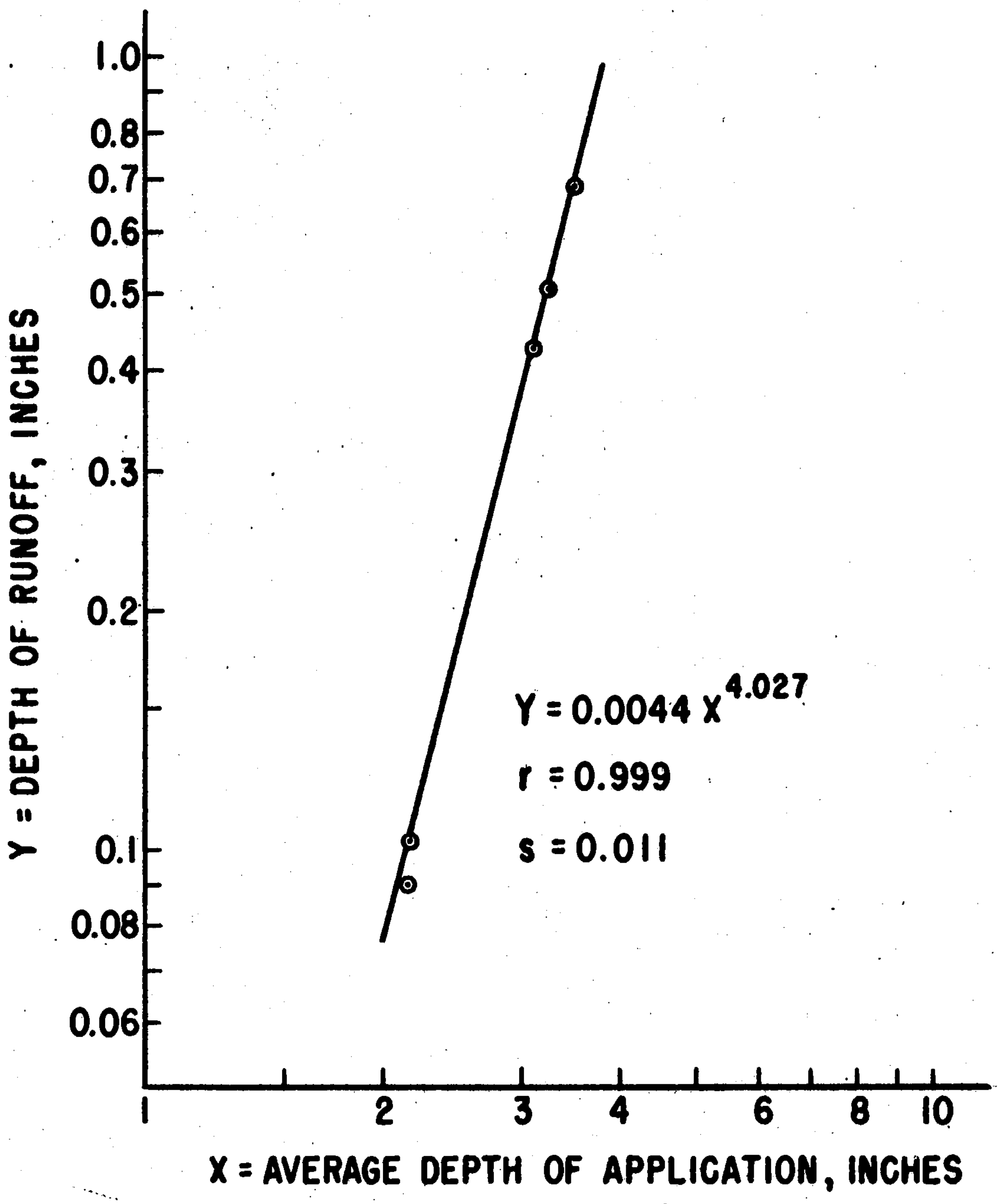

Figure 10. Relationship to Predict the Depth of Runoff for Fields with 1/2 Mile Row Lengths and Slopes Between 0.33 and 0.365 Percent

1. variations in slope within the field,
2. uneven flow to the furrows of the same set,
3. moisture content variations throughout the season,
4. climatic effects such as variation in temperature and evaporation, and
5. possible error in the size and length of the sets as recorded by the farmer and investigator.

A limited amount of data is available on the effects of the factors listed as numbers 2 and 4 above.

Variation in Individual Furrow Flow Rate

The furrow flows of several sets were measured with a submerged orifice plate. The value of the orifice coefficient was found in the laboratory.

Table III shows sixteen furrow flows measured for one set consisting of 75 rows. The flows measured are thought to be accurate since the average flow rate for the rows measured times the number of rows in the set was approximately equal to the total flow of the system. The calculated value was 1050 gpm as compared to 1075 gpm measured with the propeller flow meter.

The flow for some rows was nearly twice that of others. Measurement of individual flows for other farms yielded similar results. The variation in individual furrow flow seems to be a factor that would affect the quantity of runoff since the furrows with large flows would contribute more quickly to runoff.

TABLE III

INDIVIDUAL FURROW FLOW MEASUREMENTS

Furrow Location in Set		Flow Rate gpm
1		9.91
2		21.23
3		19.48
4		15.44
5		18.40
6		13.51
7		10.59
8		13.02
9		14.05
10		16.34
11		15.44
71		9.16
72		11.85
73		9.91
74		12.43
75		13.51
	Mean =	14.02
	Std. Dev. =	3.54

Variation in Day and Night Runoff

A series of consecutive sets at Station Number 2 with consistent application times and number of furrows per set were analyzed for a possible decrease in runoff due to the greater temperature and evaporation during the day sets. One other station used 12 hour sets but the application times and number of furrows per set were not constant and the sets could not be accurately compared. All other stations applied water for approximately 24 hours per set.

The volume of runoff was calculated as a percentage of the volume of water applied for the sets studied. Table IV contains the percentages of runoff obtained from 7 consecutive days of data. The set was changed at 7 A.M. and 7 P.M. resulting in a pair of observations for

TABLE IV

RUNOFF VARIATION IN 12 HOUR SETS

Day No.	Runoff Percent		
	Night	Day	Difference
1	4.04	0.51	3.53
2	3.04	0.49	2.55
3	7.82	6.62	1.20
4	9.38	5.02	4.36
5	7.62	5.01	2.61
6	5.05	3.51	1.54
7	4.51	1.43	3.08
Means	5.92	3.23	2.69

each day. These data were analyzed as a paired experiment with the hypothesis that there was no difference in runoff from day and night sets. The hypothesis was rejected and the difference in means was significant at the .01 level.

The average maximum and minimum daily temperature for each month during the irrigation season along with the total monthly evaporation are presented in Appendix C for two towns near the research location. Stations 1 and 2 were near Goodwell, Stations 3, 4, and 5 were near Hooker, and Station 6 was approximately equi-distant from each town.

Distribution of Runoff for Individual Sets

The volume of runoff for each set was calculated as a percentage of the volume of water applied for that set. ~~The runoff~~ percentages were plotted on log-probability paper and were found to approximate a straight line for each station. The probability was figured as

$$P = \frac{1}{t_p} \times 100 \qquad (5\text{-}3)$$

where

$$t_p = \frac{1 + N}{m}$$

and P is the probability of the runoff percent from a set being equaled or exceeded during any one set, t_p is the recurrence interval in number of sets, and m is the m^{th} largest runoff percent in the period of record, N sets (12).

Figure 11 shows the log-probability graph for Station 5. Using the graph, 90 percent of the sets from Station 5 would have less than 23.5 percent runoff. The runoff percent for any other desired

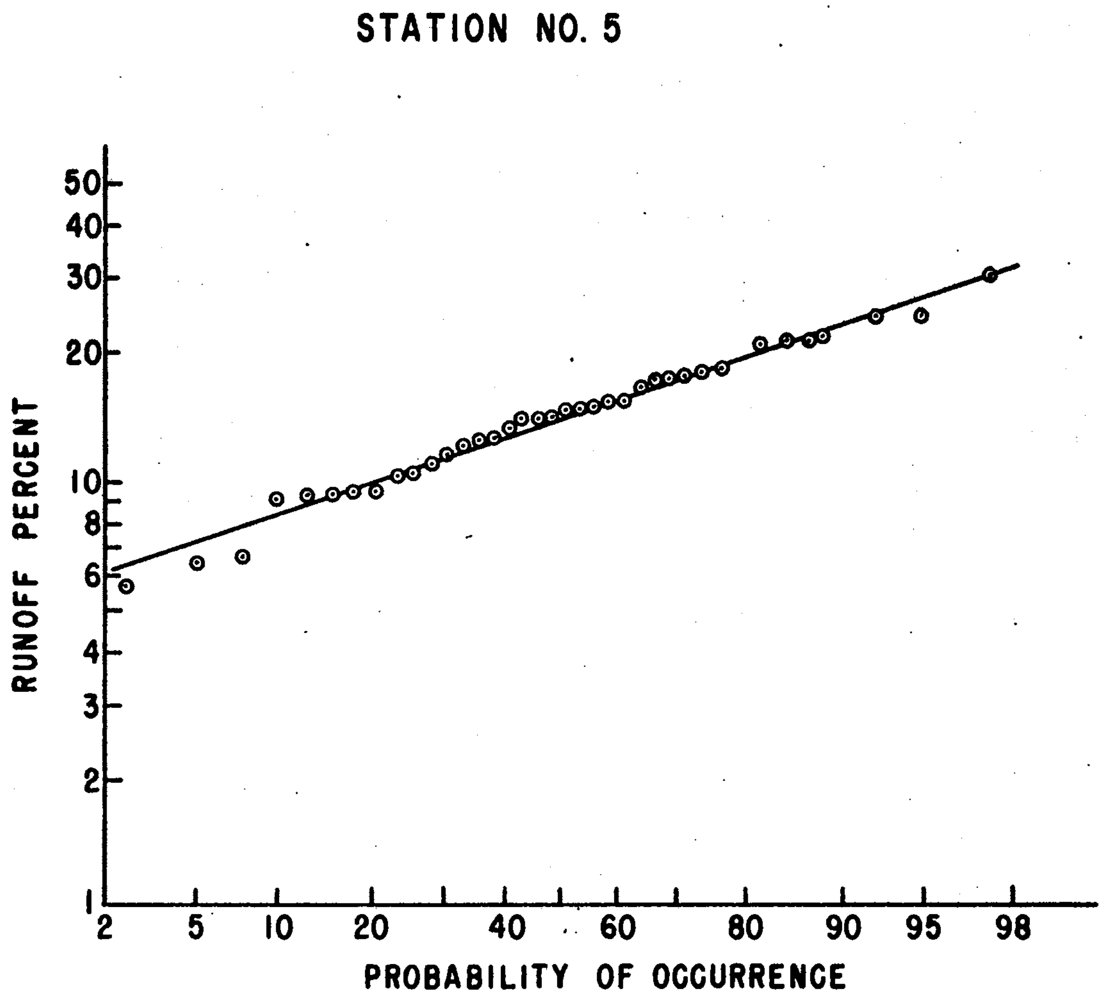

Figure 11. Example Log-Probability Relationship for Individual Set Runoff Percentages

probability can be obtained in a similar manner.

Similar log-probability graphs for the other stations are presented in Appendix D. Table V shows the runoff percent expected at common

TABLE V

LEVELS OF RUNOFF PERCENT AT COMMON PROBABILITIES

Station Number	Expected Runoff Percent		
	50%	75%	90%
1	20.5	22.8	25.0
2-A	4.3	6.7	10.0
2-B	4.1	6.0	8.5
4-A	23.0	32.0	43.0
4-B	23.0	31.0	40.0
5	14.0	18.2	23.5
6	18.0	23.0	29.0

probability levels for each station. Since variation in runoff for different sets does occur, these relationships are important in the design of reuse systems.

A reuse system designed to handle the water from the average set would lose a large percentage of the water to overflow. A system designed to handle 90 or 95 percent of the total runoff would be more acceptable to the farmer. If a reservoir large enough to store water from large sets were used, the water could be pumped later during smaller sets. By this method the pumping rate could be continuous and pump size minimized.

Time Distribution of Runoff

The rate of runoff which occurred with respect to time during an irrigation set was thought to be of importance in the design of recirculation systems.

Description

The hydrographs of time versus runoff rate for two typical irrigation sets are shown in Figure 12. These sets will be used to describe the time distribution of surface runoff.

The first set in the series began at 8 A.M. and was 24 hours long. No runoff occurred until the first stream had watered through the field at 7 P.M. The runoff rate increased as additional furrows watered through the field until all furrows were contributing to the runoff or the set was changed. The peak rate of runoff occurred about two hours after the set was moved. After the peak, the rate of runoff decreased rapidly until all the runoff water stored on the field was depleted. The area under the hydrograph curve would then represent the volume of runoff from that particular set. The hydrograph from the second irrigation set in the series illustrates that the time distribution for the different sets is very similar. The horizontal dotted lines in Figure 12 represent the average runoff rates if taken over the length of the respective sets.

To facilitate the study of the time distribution of surface runoff several characteristics of the runoff hydrograph are defined.

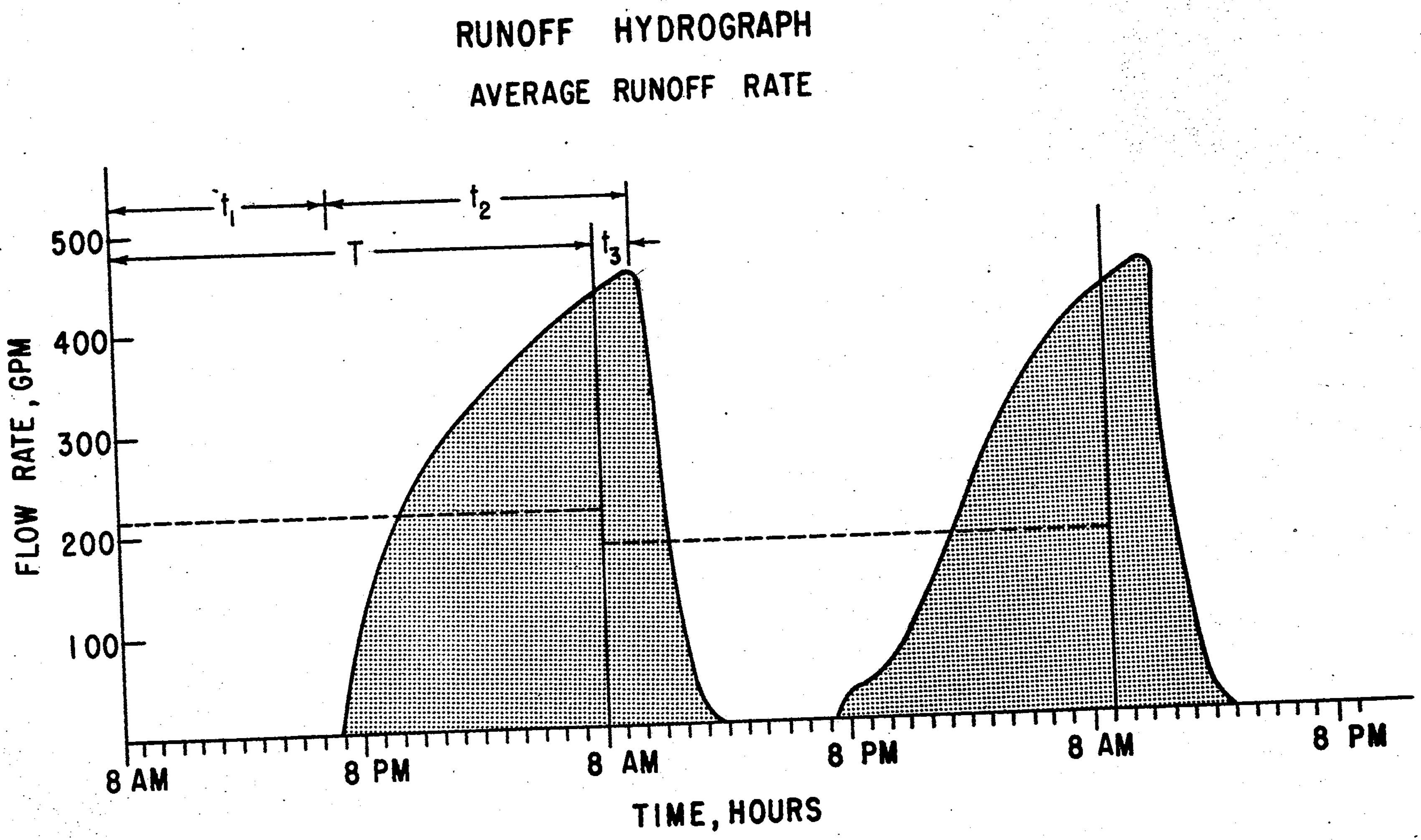

Figure 12. The Runoff Hydrographs from Two Typical Irrigation Sets Showing Characteristics of the Time Distribution

Symbol	Characteristic	Units
T	The total time water was applied to the set.	hours
t_1	The time interval from the beginning of set until the first runoff occurs.	hours
t_2	The time interval from the beginning of runoff until the peak occurs.	hours
t_3	The time interval from when irrigation is stopped or the set changed until the peak occurs.	hours
R_1	Ratio of the average runoff rate for the set to the maximum runoff rate of the set.	percent
R_2	Ratio of the volume of runoff which occurs before the set ends to the total volume of runoff for that set.	percent

The characteristics of the hydrographs for each station were found to be fairly uniform. Table VI presents the average value of each characteristic for the various stations.

The time distribution of the surface runoff is of interest when considering the possible methods of reusing the water. Two possible methods of returning the runoff water to supplement the main water source will be discussed:

1. Reuse by pumping in cycles, and
2. Reuse by continuous pumping.

Reuse by Pumping in Cycles

Since a large percentage of the runoff volume occurs over a time interval smaller than the application time, the reuse of the water by pumping in cycles has some merit. The runoff water could be used to accomplish a cut-back type irrigation (7) using the runoff water from the previous set to supplement the main water source. If the reuse

TABLE VI

AVERAGE CHARACTERISTICS OF TIME DISTRIBUTION FOR INDIVIDUAL SETS

Station Number	T hours	t_1 hours	t_2 hours	t_3 hours	R_1 percent	R_2 percent
1	22.71	7.29	15.96	0.71	43.92	84.63
2-A	11.75	8.77	4.79	1.96	34.78	35.01
2-B	12.00	10.48	3.75	2.26	25.23	23.67
4-A	17.61	4.86	12.81	0.33	58.13	80.92
4-B	24.00	9.75	15.00	1.32	46.45	70.98
5	12.46	5.36	8.41	1.36	51.74	64.20
6	23.21	11.68	13.38	1.82	31.30	63.37

pump were started at the same time as a new set and the water pumped at a rate such that the total volume of runoff from the previous set could be pumped in a portion of the application time, a cut-back flow would be developed when the reuse pump shut off.

Figure 13 illustrates such an example where a pumping rate of 430 gpm would be necessary to reuse the volume of runoff from the first set (shaded area) over a period of 12 hours (area in rectangle). The pump would be off 12 hours and then started again with the next set change.

The amount of storage required for this type of system would be the volume of runoff which occurred before the set was changed, since the runoff would be pumped as it occurred after the set change. The volume which occurred before the set change was found to be from 60 to 85 percent of the total in most cases as described earlier by R_2. Station 2 had an average value of R_2 lower than the other stations because of large t_1 values as compared to total set time, T.

A properly designed cut-back system would allow the irrigation of larger sets with higher application efficiencies as compared to irrigation without runoff water reuse.

A cycling system with a very small amount of storage, as presently used on some farms, would result in large overflows unless a pump large enough to handle the maximum flow were used. Since runoff would be repumped as it occurred, the additional water would be applied by increasing the furrow stream size after the furrows were wet. This method would accomplish the opposite results of a cut-back system and would decrease the application efficiency.

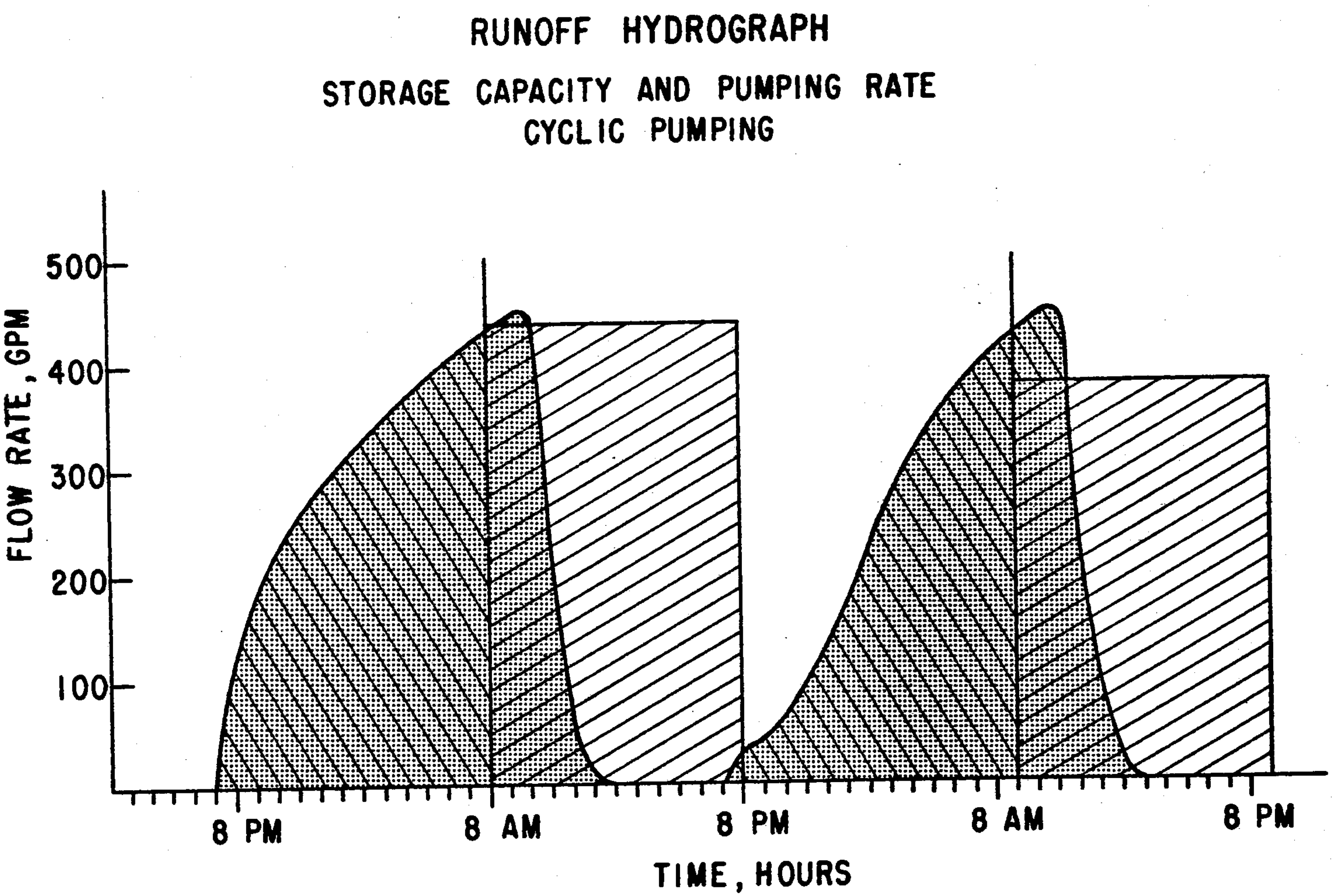

Figure 13. The Volume of Runoff Could Be Pumped in One-half the Set Time to Obtain a Cut-back Method of Irrigation

Reuse by Continuous Pumping

Another alternative for the design of a reuse system would be to pump the runoff water continuously and use this water to supplement the main water source. Figure 14 illustrates a continuous pumping arrangement. This would require a much smaller size pump than for the cutback cycling method. The quantity of storage would also be less since only the volume above the pumping rate line, shown in Figure 14, would be required in storage at any one time. This volume ranges from zero for the smallest sets to about 60 percent of the volume of the largest sets. The water in storage would then be pumped after the runoff rate dropped below the pumping rate.

Since the total amount of water applied would be greater with the addition of recirculated water, a larger set would need to be irrigated. If the percentage of runoff remained the same, the volume of runoff would increase so the reuse pump and storage pit would need to be designed to handle the additional amount.

Storage Routing of Runoff

A storage routing computer program was written to determine the effect of storage size and pumping rate on storage reservoir overflow and unused pump capacity. Unused pump capacity is defined as the volume of water that would have been pumped during the time interval if water were available.

Calculation of overflow or unused pump capacity was made at each 15 minute time interval and summed over the entire series of sets for each station. This was done at a constant pumping rate and storage size. Figure 15 illustrates how the program operated. If the runoff

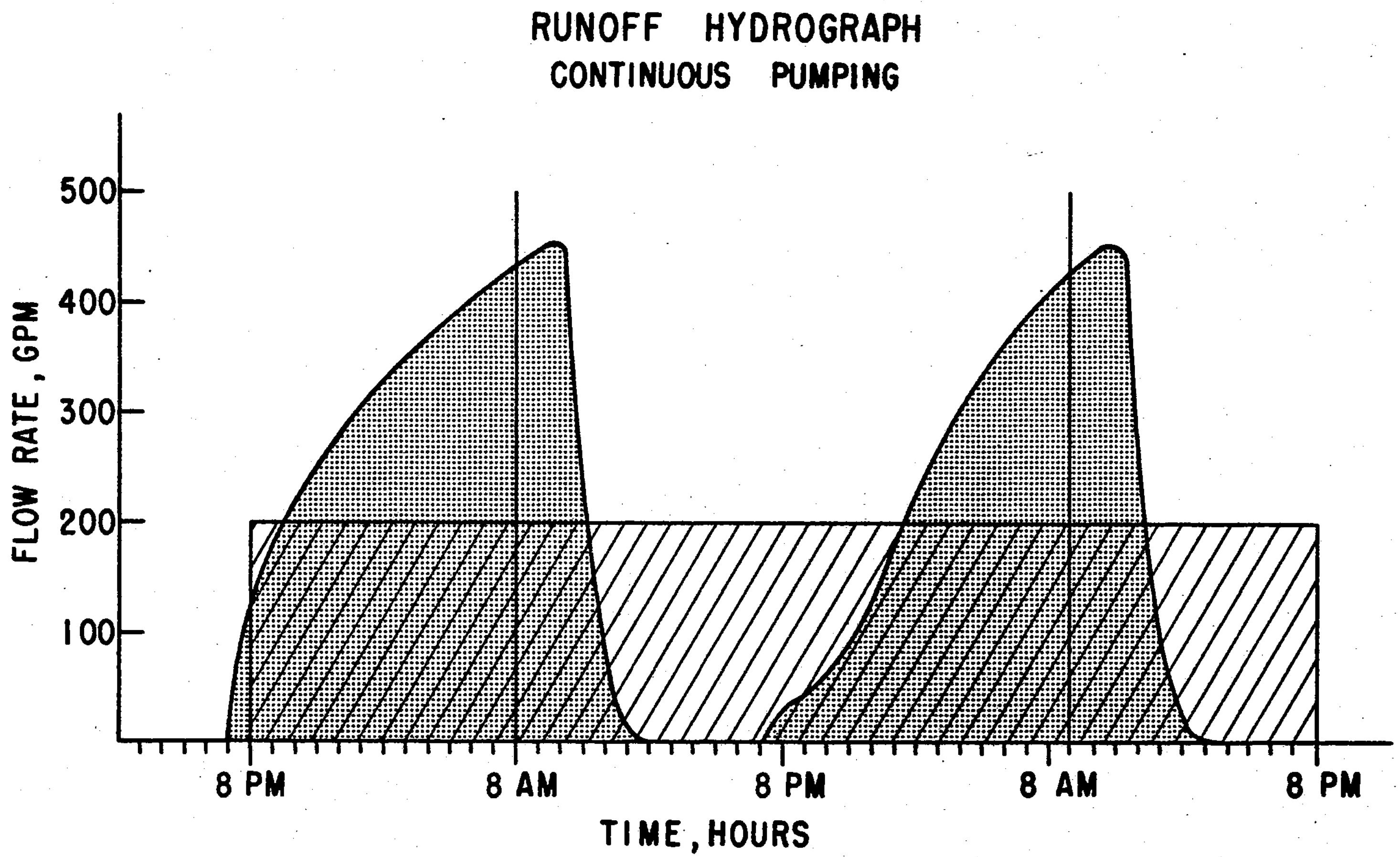

Figure 14. The Area Above the Constant Pumping Line Represents About 60 Percent of the Runoff Volume for Each Set

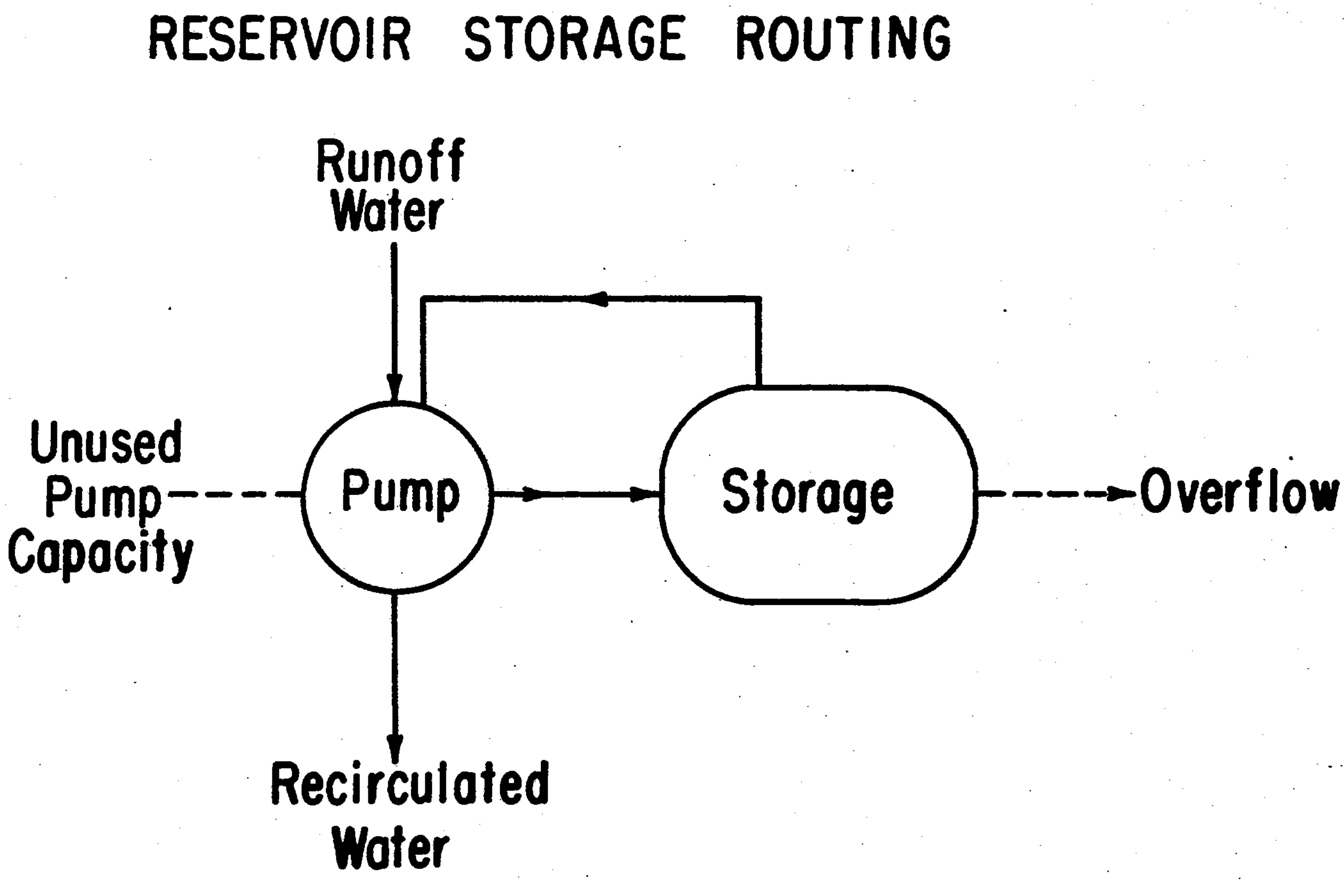

Figure 15. A Schematic Diagram of the Storage Routing Procedure

rate was in excess of the pumping rate for an extended period of time, the storage reservoir would overflow. And conversely, if the runoff rate was less than the pumping rate long enough to deplete the water in storage, water would be unavailable for pumping and unused pump capacity would result.

The effect of various pumping rates and storage capacities was checked for each station with the storage routing program. Figure 16 shows the volume of overflow that would be lost for three constant pumping rates and different storage sizes using the runoff data from one station. Overflow decreases with both an increase in storage capacity and pumping rate.

Using the same data, Figure 17 shows the volume of unused pump capacity for the same range of storage sizes and pumping rates. Notice that the unused pump capacity also decreases with increasing storage capacity; however, the larger pumping rates have larger unused pump capacities.

A comparison of Figures 16 and 17 would indicate that an increase in storage capacity would be more advantageous than an increase in pumping rate to minimize both overflow and unused pump capacity.

Similar storage routing graphs are presented in Appendix E for each station.

Comparison of Log-Probability Prediction with Storage Routing Results

The two main functional restraints on a reuse system are the amount of water allowable as overflow and unused pump capacity. If a system is designed with a continuously operated reuse pump, the unused pump capacity must be limited for the system to operate properly.

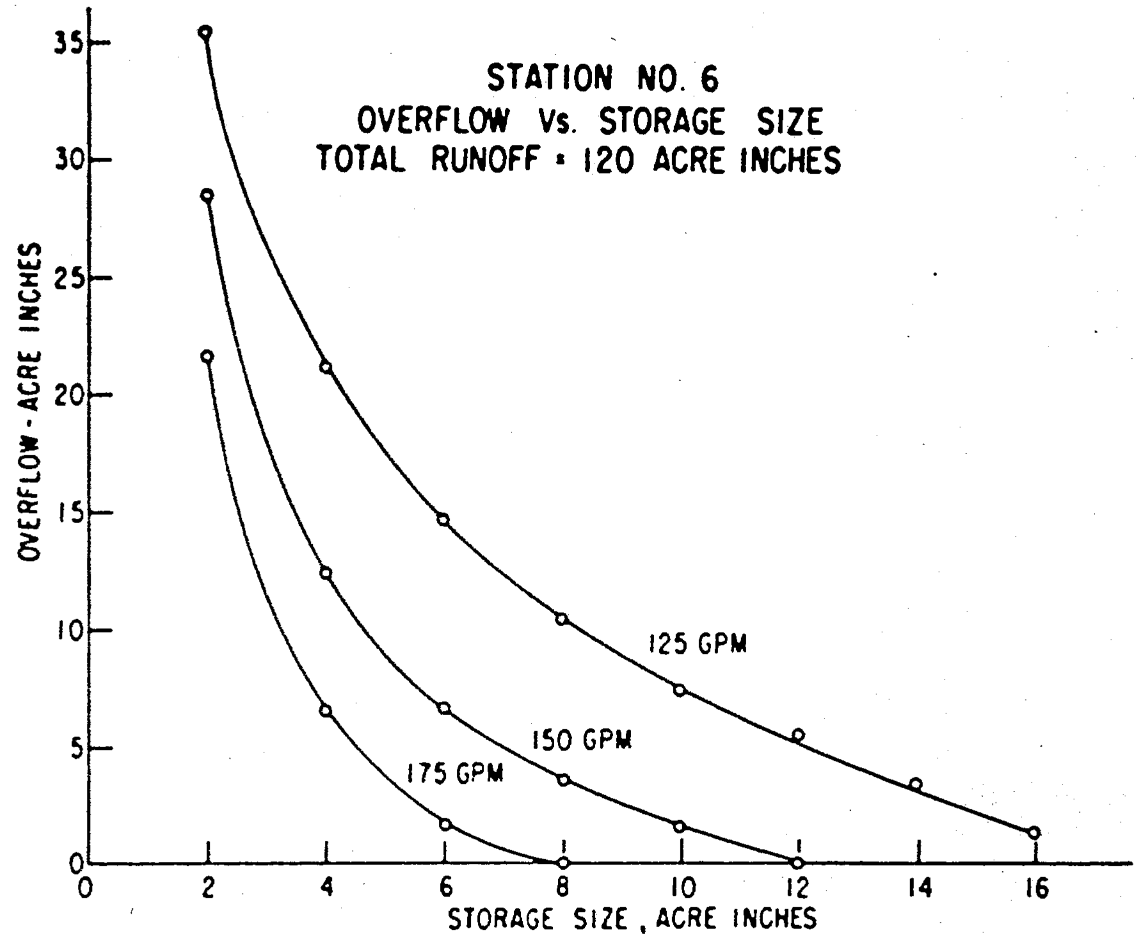

Figure 16. Example Curves Showing the Decrease in Overflow with an Increase in Storage Pit Size for Pumping Rates of 125, 150, and 175 gpm

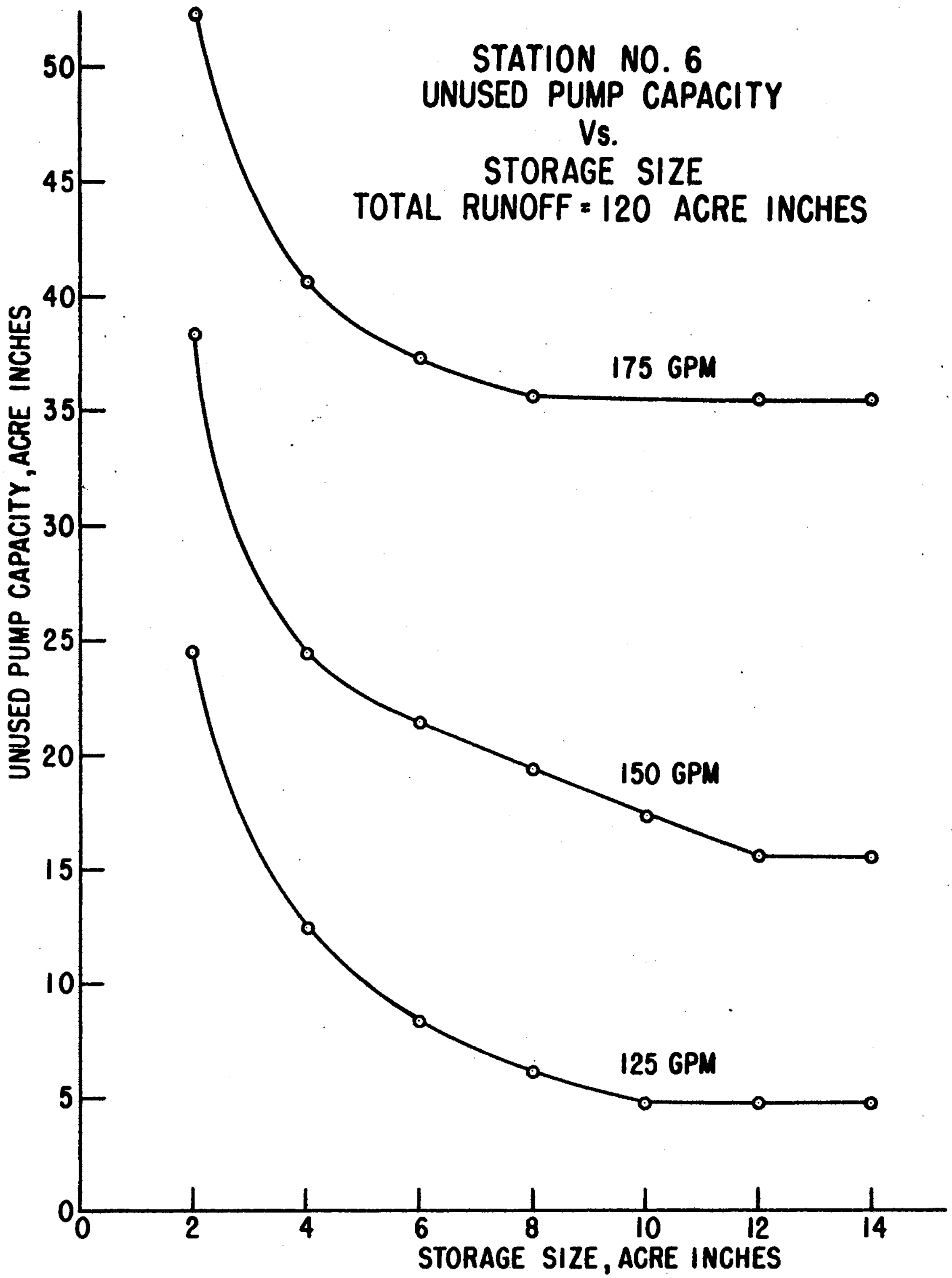

Figure 17. Example Curves Showing the Decrease in Unused Pump Capacity with Increasing Storage Pit Size for Pumping Rates of 125, 150, and 175 gpm

Since storage routing curves will not be available for most systems designed, the validity of using the more general time distribution and probability concepts was investigated.

Table VII presents the results of the validity check. The reuse system pumping rate was chosen to equal the average runoff rate for each station (Column 4). Storage size was chosen as follows: the volume of water applied per set was calculated by using the well flow and nominal set time for each station (Columns 2 and 3). The 90 percent column of Table V was used to determine the largest runoff percent expected during 90 percent of the sets. The 90 percent level times the volume applied is shown in Column 5. However, only 60 percent of the runoff volume is contained in storage at one time as discussed in the section on continuous pumping. This value was used to obtain the final storage size required, shown in Column 6.

Using the storage routing curves for each station, the amount of overflow and unused pump capacity can be obtained for the chosen pumping rate and storage size. Column 7 shows the percent of the total seasonal runoff lost as overflow. The percent of the time that the pump would be unused is shown in Column 8.

The design method is assumed valid since each of the systems resulted in less than 10 percent overflow; however, it may be profitable or necessary in some cases to reduce the overflow to zero by increasing the storage size or pumping rate.

Economic Analysis

Calculations were made to determine the cost of installing a reuse system at each of the stations studied. Table VIII shows the cost of

TABLE VII

OVERFLOW AND UNUSED PUMP CAPACITY LOSSES FOR SYSTEMS DESIGNED AT THE 90 PERCENT PROBABILITY LEVEL

1	2	3	4	5	6	7	8
Station Number	Nominal Set Length hours	Volume Applied Per Set ac. in.	Runoff Pumping Rate gpm	90% Level Runoff Volume ac. in.	Storage Size 0.6 x Col. 5 ac. in.	Seasonal Overflow Losses percent	Unused Pump Capacity percent
1	24	49.5	190	12.3	7.4	3.8	9.0
2-A	12	45.0	80	4.5	2.7	7.1	0.8
4-A	24	93.0	450	40.0	24.0	6.6	16.5
5	12	41.7	245	9.7	5.8	7.7	9.2
6	24	38.4	138	11.1	6.7	7.5	11.5

Note: Calculations were not made for Stations 2-B and 4-B since they were the same fields as 2-A and 4-A only with lower inflow rates.

TABLE VIII

COST ANALYSIS

Station Number	Total Annual Runoff ac.ft.	Pump Size gpm	Pipe Size inches	Storage Table VII Column 6 ac. in.	Annual Cost $/ac. ft.	Storage Size No Overflow ac. in.	Annual Cost $/ac. ft.
1	40	190	6	7.4	6.20	11.2	6.25
2-A	15	80	4	2.7	20.40	4.5	19.30
4-A	85	450	8	24.0	6.65	35.0	6.65
5	60	245	6	5.8	7.30	17.5	7.25
6	43	138	4	6.7	8.45	14.0	8.40

recirculated water on a cost per acre foot basis.

The systems were designed on the basis of the information presented in Table VII where from 3.8 to 7.7 percent of the annual runoff was lost to overflow. The annual cost per acre foot was calculated on the basis of the percentage of the water pumped.

Also presented in Table VIII is the cost per acre foot if the storage pit is designed for no overflow and all the annual runoff is reused.

Fixed costs were calculated using 1970 prices for installed low head plastic pipe and Gormun-Rupp self priming centrifugal pumps with Wisconsin internal combustion engines. Storage pit construction was based on $.20 per cubic yard and miscellaneous items were assumed to cost $100 for each installation. The pipeline was 1/2 mile long in each case except Station 1 where 1/4 mile rows were used.

The annual cost was figured using a capital recovery factor based on 7 percent interest and a 20 year equipment life. Fuel costs were based on 30 cents per 1000 cubic foot of natural gas, which was available in the study area. Repairs and upkeep were figured as $35 per year.

The systems designed would probably be profitable except for Number 2-A which had a low rate of runoff. The extra construction cost incurred with the larger storage pits was offset with the additional water saved. The value of the additional land used may need to be considered in some cases, but this would usually only involve around .2 acres.

The feasibility of installing a reuse system would depend on the individual situation. The potential yield production and the availability of additional water from the main water well needs to be considered. In areas where an eventual groundwater shortage is expected, the runoff water may need to be valued on future production potential.

The cost of reusing runoff can be reduced significantly if it can be used downstream rather than returning the water to the upper end of the field from which it occurs. If the water is returned to the same field, a larger set will be necessary if the same stream size per furrow is used.

CHAPTER VI

SUMMARY AND CONCLUSIONS

Summary

The surface runoff from six furrow irrigated fields in the Oklahoma Panhandle was measured. A relationship was developed to predict the average volume of runoff from fields with similar slopes and row lengths.

The variation in runoff from irrigation sets of the same field was studied. The runoff percentages for the individual irrigation sets were found to approximate a log-normal distribution. The log-probability relationships can be used to predict the largest runoff percentage expected for the desired confidence level.

The time distribution of the runoff was investigated. The rate of runoff increases gradually as furrows water through the field until the set is changed. The peak rate of runoff occurs between one and two hours after the set is changed and will be approximately twice the average runoff rate. Between 60 and 80 percent of the runoff has occurred by the time the set is changed. After the peak, the rate of runoff decreases rapidly.

A system may be designed with either a cycling or continuously operated pump using the information from the time distribution study and the log-probability relationships.

The cycling type reuse system can incorporate a cut-back type irrigation; however, this type of system will require a larger pump size and will have a higher annual cost.

Systems with continuously operated pumps were designed for several stations on the basis of the time distribution and log-probability results. Overflow and unused pump capacity were calculated with a reservoir storage routing program. Systems designed to pump the average runoff rate and to store a maximum of 60% of the water from the largest set expected at the 90 percent confidence level resulted in 3.8 to 7.7 percent overflow. Approximately double this design storage capacity was necessary to reduce the overflow to zero.

The annual cost per acre foot for installation and operation was calculated for the systems with two storage reservoir sizes. The systems designed to eliminate overflow had a higher total cost, but the cost per acre foot was equal or lower than the systems designed on the 90 percent confidence level since more water was pumped when overflow was eliminated.

Conclusions

The following conclusions are presented from the results of the study:

1. The average volume of runoff expected from furrow irrigated fields is mainly a function of the average volume of water applied per unit area.
2. The volume of water expected from an individual set is a function of several additional variables; however, the runoff percentages are approximately normally distributed, although each field may have a different mean and standard deviation.

3. Systems can be designed to reuse runoff water with little or no overflow and still be within functional and economic restraints.
4. Surface runoff water from furrow irrigation can provide an additional source of irrigation water at a cost competitive with other sources.
5. The reuse of runoff water will extend the life of groundwater supplies in areas where water is being removed by pumping at a higher rate than it is recharged. The potential crop production of the runoff water should then be considered.

Suggestions for Future Research

1. A study of the quality of irrigation runoff water is needed.
2. The installation and field evaluation of a reuse system would be desirable.
3. A study to determine the effects of row length, field slope, soil moisture conditions, crop water use rate, and infiltration rate of the soil would be desirable. An experiment completely controlled by the investigator would be recommended for a study of this nature.

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APPENDIX A

H FLUME CALIBRATION EQUATIONS OF THE FORM OF EQUATION (3-1) WITH THE DISCHARGE GIVEN IN GALLONS PER MINUTE

TABLE IX

H FLUME CALIBRATION EQUATIONS OF THE FORM OF EQUATION (3-1) WITH THE DISCHARGE GIVEN IN GALLONS PER MINUTE

Flume Identification Number	Intercept a	Slope b	Range of Applicability (ft.)	Standard Deviation	Correlation Coefficient
No. 4	383.03	1.798	0.060 - 0.138	1.091	0.999
	774.90	2.154	0.139 - 0.529	1.072	0.999
	937.98	2.454	0.530 - 0.980	0.568	0.999
No. 5	439.44	1.822	0.060 - 0.185	0.642	0.999
	784.25	2.165	0.186 - 0.576	1.088	0.999
	890.90	2.397	0.577 - 0.980	0.615	0.980
No. 6	451.53	1.847	0.060 - 0.178	1.044	0.999
	791.53	2.172	0.179 - 0.570	1.019	0.999
	904.02	2.410	0.571 - 0.980	0.441	0.999

APPENDIX B

INDIVIDUAL SET DATA AND CALCULATIONS

TABLE X

INDIVIDUAL SET DATA AND CALCULATIONS

STATION NO.	WELL YIELD, GPM	SET LENGTH, HOURS	SET AREA, ACRES	DEPTH APPLIED, INCHES	DEPTH RUNOFF, INCHES	PERCENT RUNOFF	VOLUME RUNOFF, AC IN	PEAK RATE, GPM	AVERAGE RATE, GPM
1	930	26.75	8.48	6.48	1.328	20.50	11.27	449.7	190.7
		23.75	9.19	5.31	0.978	18.41	8.99	446.5	171.2
		24.00	9.19	5.37	1.039	19.36	9.55	436.9	180.1
		23.25	9.33	5.12	0.901	17.60	8.41	436.9	163.7
		24.00	9.19	5.37	1.057	19.70	9.72	394.7	183.2
		12.75	9.19	2.85	0.156	5.48	1.44	205.4	51.0
		19.00	7.07	5.52	1.269	22.97	8.97	434.4	213.6
		23.00	7.92	5.97	1.144	19.16	9.06	412.1	178.2
		24.00	9.19	5.37	1.359	25.32	12.49	444.6	235.5
		24.00	9.19	5.37	1.340	24.97	12.32	440.7	232.2
		24.00	9.19	5.37	1.333	24.83	12.25	443.3	231.0
		24.00	9.05	5.45	1.190	21.83	10.77	440.1	203.0
6	725	11.00	7.47	2.36	0.379	16.09	2.84	407.9	116.7
		24.50	9.09	4.32	0.375	8.68	3.41	356.9	63.0
		23.50	9.09	4.14	1.032	24.92	9.38	494.9	180.7
		24.50	10.10	3.89	0.755	19.42	7.62	479.7	140.8
		24.00	10.10	3.81	0.276	7.24	2.78	224.9	52.5
		24.00	11.11	3.46	0.598	17.28	6.64	449.0	125.3
		24.00	10.91	3.52	0.830	23.54	9.05	528.0	170.7
		24.00	12.93	2.97	0.708	23.82	9.16	422.3	172.7
		23.00	9.70	3.80	1.197	31.51	11.61	449.6	228.4
		25.00	10.51	3.81	0.953	24.99	10.01	446.0	181.2
		24.00	10.51	3.66	0.561	15.33	5.89	397.2	111.1
		23.00	10.91	3.38	0.616	18.24	6.72	400.6	132.2
		24.00	10.91	3.52	0.444	12.61	4.85	339.8	91.4
		24.00	11.72	3.28	0.249	7.59	2.92	296.5	55.0
		24.00	10.51	3.66	0.681	18.59	7.15	476.6	134.8
		24.00	10.10	3.81	0.890	23.37	8.99	528.0	169.5
		24.00	10.10	3.81	1.056	27.73	10.66	538.0	201.0

TABLE X (CONTINUED)

STATION NO.	WELL YIELD, GPM	SET LENGTH, HOURS	SET AREA, ACRES	DEPTH APPLIED, INCHES	DEPTH RUNOFF, INCHES	PERCENT RUNOFF	VOLUME RUNOFF, AC IN	PEAK RATE, GPM	AVERAGE RATE, GPM
2	1700	11.00	21.21	1.95	0.103	5.31	2.19	312.7	90.3
		12.00	24.04	1.88	0.146	7.79	3.51	403.7	132.5
		12.00	21.78	2.07	0.076	3.67	1.65	133.6	62.3
		12.00	25.45	1.77	0.118	6.64	2.99	339.6	112.9
		12.00	21.21	2.13	0.026	1.21	0.54	106.4	20.5
		12.00	25.45	1.77	0.169	9.55	4.30	348.5	162.3
		11.00	29.70	1.39	0.042	3.00	1.24	161.0	50.9
		12.00	28.28	1.59	0.133	8.32	3.75	323.8	141.4
		11.00	24.04	1.72	0.054	3.12	1.29	153.9	53.0
		12.00	24.04	1.88	0.094	5.02	2.26	228.8	85.4
		12.00	21.21	2.13	0.073	3.42	1.54	180.9	58.1
		12.00	24.04	1.88	0.060	3.22	1.45	178.0	54.7
2	960	12.00	11.31	2.25	0.091	4.04	1.03	143.2	38.7
		12.00	9.90	2.57	0.013	0.51	0.13	39.3	4.9
		12.00	11.31	2.25	0.060	2.66	0.68	135.7	25.5
		12.00	9.90	2.57	0.013	0.50	0.13	23.8	4.8
		12.00	11.31	2.25	0.176	7.82	1.99	290.7	75.1
		12.00	9.90	2.57	0.124	4.83	1.23	216.6	46.4
		12.00	11.31	2.25	0.220	9.77	2.49	278.8	93.8
		12.00	9.90	2.57	0.132	5.12	1.30	219.8	49.1
		12.00	11.31	2.25	0.168	7.46	1.90	203.8	71.6
		12.00	9.90	2.57	0.145	5.64	1.43	233.6	54.1
		12.00	11.31	2.25	0.128	5.68	1.45	227.8	54.6
		12.00	9.90	2.57	0.079	3.07	0.78	177.7	29.5
		12.00	11.31	2.25	0.099	4.42	1.12	190.7	42.4
		12.00	9.90	2.57	0.045	1.73	0.44	105.2	16.6
		18.00	11.31	3.38	0.137	4.05	1.55	146.1	38.9
		6.00	9.90	1.29	0.135	10.50	1.34	255.1	100.8
		12.00	11.31	2.25	0.190	8.43	2.15	334.7	80.9
		12.00	11.31	2.25	0.054	2.41	0.61	120.3	23.1
		12.00	9.90	2.57	0.121	4.69	1.19	177.7	45.1
		12.00	14.14	1.80	0.054	2.98	0.76	136.3	28.6
		11.00	12.73	1.83	0.108	5.87	1.37	206.9	56.3
		12.00	14.14	1.80	0.154	8.57	2.18	217.0	82.3
		10.00	12.73	1.67	0.047	2.80	0.59	85.2	26.9
		15.00	22.63	1.41	0.071	5.06	1.61	137.1	48.6
		12.00	14.14	1.80	0.067	3.73	0.95	140.7	35.8
		12.00	13.29	1.92	0.051	2.67	0.68	106.0	25.6

TABLE X (CONTINUED)

STATION NO.	WELL YIELD, GPM	SET LENGTH, HOURS	SET AREA, ACRES	DEPTH APPLIED, INCHES	DEPTH RUNOFF, INCHES	PERCENT RUNOFF	VOLUME RUNOFF, AC IN	PEAK RATE, GPM	AVERAGE RATE, GPM
5	1575	18.00	13.58	4.61	0.994	21.54	13.49	898.0	339.2
		13.00	14.14	3.20	0.328	10.26	4.64	528.3	161.6
		11.00	15.56	2.46	0.430	17.47	6.69	582.4	275.2
		12.00	13.29	3.14	0.382	12.16	5.08	542.9	191.5
		12.00	13.29	3.14	0.549	17.46	7.29	638.0	275.0
		10.50	13.29	2.75	0.156	5.69	2.08	234.0	89.6
		13.50	14.71	3.19	0.537	16.80	7.90	465.7	264.7
		11.00	13.29	2.88	0.267	9.26	3.54	307.5	145.8
		13.00	13.29	3.40	0.504	14.79	6.69	379.9	233.0
		13.00	13.29	3.40	0.479	14.06	6.36	384.3	221.5
		10.50	13.86	2.64	0.240	9.08	3.32	289.7	143.1
		11.50	15.84	2.53	0.161	6.38	2.56	198.0	100.5
		12.50	24.61	1.77	0.434	24.57	10.69	439.3	387.0
		12.50	12.73	3.42	0.376	11.01	4.79	592.8	173.4
		11.50	13.29	3.01	0.426	14.13	5.66	572.7	222.6
		21.00	13.58	5.38	1.047	19.44	14.21	701.3	306.2
		20.50	11.60	6.15	1.505	24.47	17.46	788.6	385.3
		12.50	14.42	3.02	0.535	17.73	7.71	716.9	279.2
		12.00	13.58	3.08	0.678	22.03	9.20	729.5	347.0
		12.00	12.73	3.28	0.595	18.13	7.57	677.5	285.5
		12.00	12.73	3.28	0.613	18.69	7.81	703.6	294.3
		12.00	12.73	3.28	0.439	13.37	5.59	470.0	210.6
		12.00	12.73	3.28	0.492	14.99	6.26	498.0	236.1
		10.50	12.73	2.87	0.268	9.33	3.41	390.3	147.0
		12.00	12.73	3.28	0.503	15.32	6.40	376.2	241.3
		11.00	12.73	3.01	0.199	6.61	2.53	308.5	104.1
		12.00	13.86	3.01	0.473	15.68	6.55	334.5	246.9
		12.50	17.25	2.52	0.374	14.85	6.46	267.1	233.8
		11.50	17.54	2.28	0.322	14.09	5.64	299.5	221.9
		12.00	18.38	2.27	0.214	9.41	3.93	206.5	148.1
		12.00	17.54	2.38	0.223	9.38	3.92	216.7	147.7
		9.50	18.38	1.80	0.228	12.68	4.19	453.0	199.6
		12.00	19.52	2.14	0.450	21.01	8.78	661.7	331.0
		12.00	19.80	2.11	0.246	11.66	4.87	434.6	183.7
		12.00	16.97	2.46	0.759	30.83	12.88	701.3	485.6
		12.80	16.69	2.67	0.336	12.59	5.61	433.4	198.3
		12.00	16.97	2.46	0.532	21.61	9.02	533.6	340.3
		10.00	20.08	1.73	0.179	10.33	3.60	223.1	162.7

TABLE X (CONTINUED)

STATION NO.	WELL YIELD, GPM	SET LENGTH, HOURS	SET AREA, ACRES	DEPTH APPLIED, INCHES	DEPTH RUNOFF, INCHES	PERCENT RUNOFF	VOLUME RUNOFF, AC IN	PEAK RATE, GPM	AVERAGE RATE, GPM
4	1750	16.00	21.52	2.88	0.370	12.86	7.96	599.6	225.0
		8.00	15.15	2.04	0.279	13.68	4.23	484.1	239.4
		13.00	15.15	3.32	0.620	18.70	9.40	532.8	327.2
		12.00	15.15	3.06	0.430	14.05	6.52	414.1	245.8
		11.00	15.15	2.81	0.972	34.61	14.72	624.9	605.6
		25.00	21.21	4.56	0.772	16.95	16.39	588.6	296.6
		24.00	21.21	4.38	1.295	29.60	27.47	1019.6	518.0
		24.00	21.21	4.38	1.517	34.68	32.19	1062.5	606.9
		25.50	21.21	4.65	1.957	42.10	41.52	1224.1	736.8
4	700	23.50	10.61	3.43	1.315	38.37	13.95	541.7	268.6
		24.50	10.61	3.57	1.003	28.06	10.63	463.6	196.4
		24.00	10.61	3.50	1.052	30.06	11.16	585.0	210.4
		24.00	11.82	3.14	0.589	18.76	6.97	325.8	131.3
		26.50	19.09	2.15	0.329	15.32	6.28	284.0	107.2
		21.50	10.91	3.05	0.437	14.34	4.77	172.3	100.4
		24.00	10.61	3.50	0.900	25.70	9.54	295.5	179.9

APPENDIX C

CLIMATIC DATA

TABLE XI

CLIMATIC DATA

Month	Avg. Max. Temp. (°F)	Avg. Min. Temp. (°F)	Total Rainfall (inches)	Total Evap. (inches)
		Goodwell, Oklahoma		
June	90.2	58.3	0.84	13.77
July	---*	---*	2.87	13.26
August	93.7	63.2	3.11	10.91
Sept.	86.3	53.6	.54	9.54
		Hooker, Oklahoma		
June	90.0	57.8	0.33	
July	95.4	64.5	1.95	
August	94.2	62.3	4.70	
Sept.	84.9	52.1	1.09	

* Data not available, average daily temperature was 79.6°F.

APPENDIX D

LOG-PROBABILITY RELATIONSHIPS

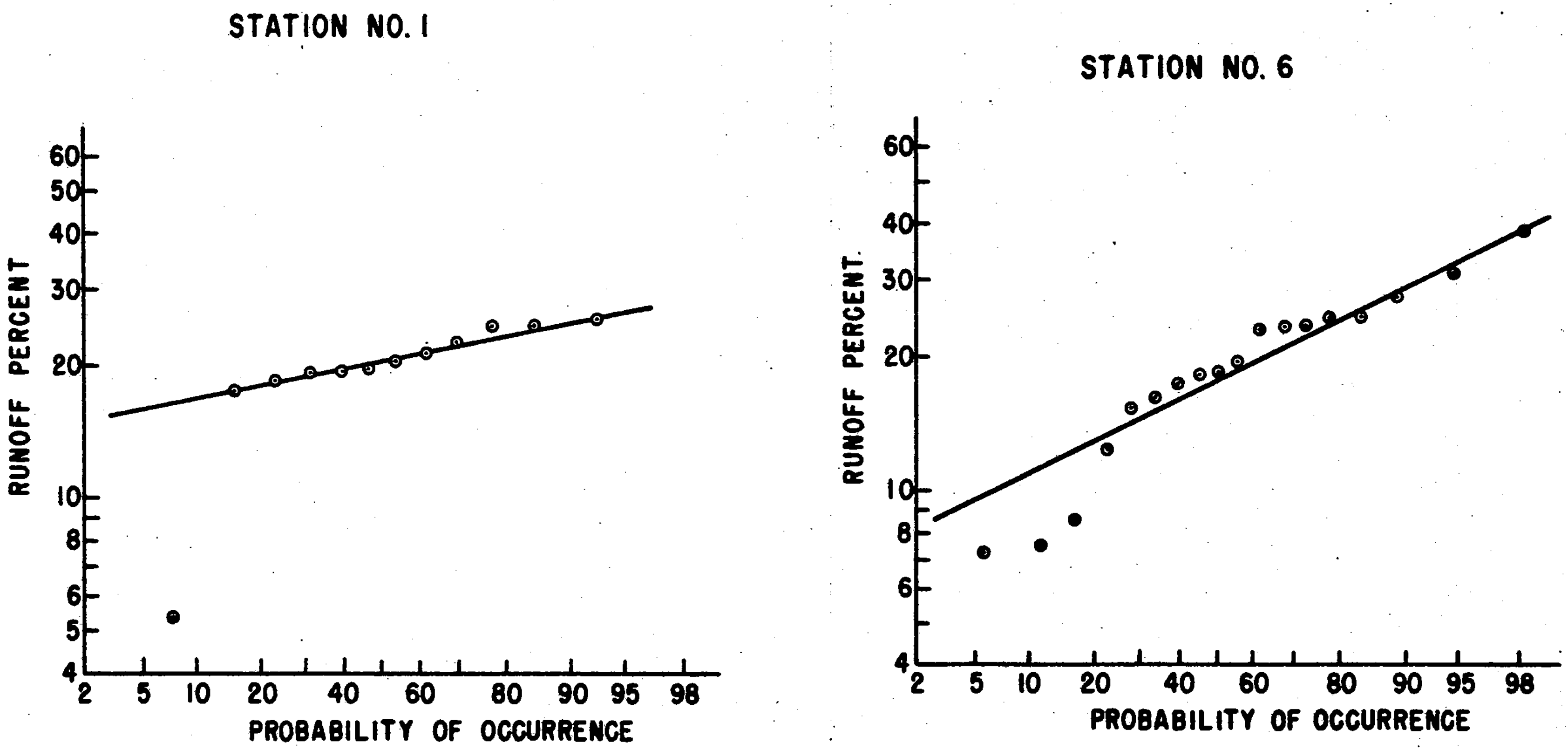


Figure 18. Log-Probability Relationships for Stations No. 1 and 6

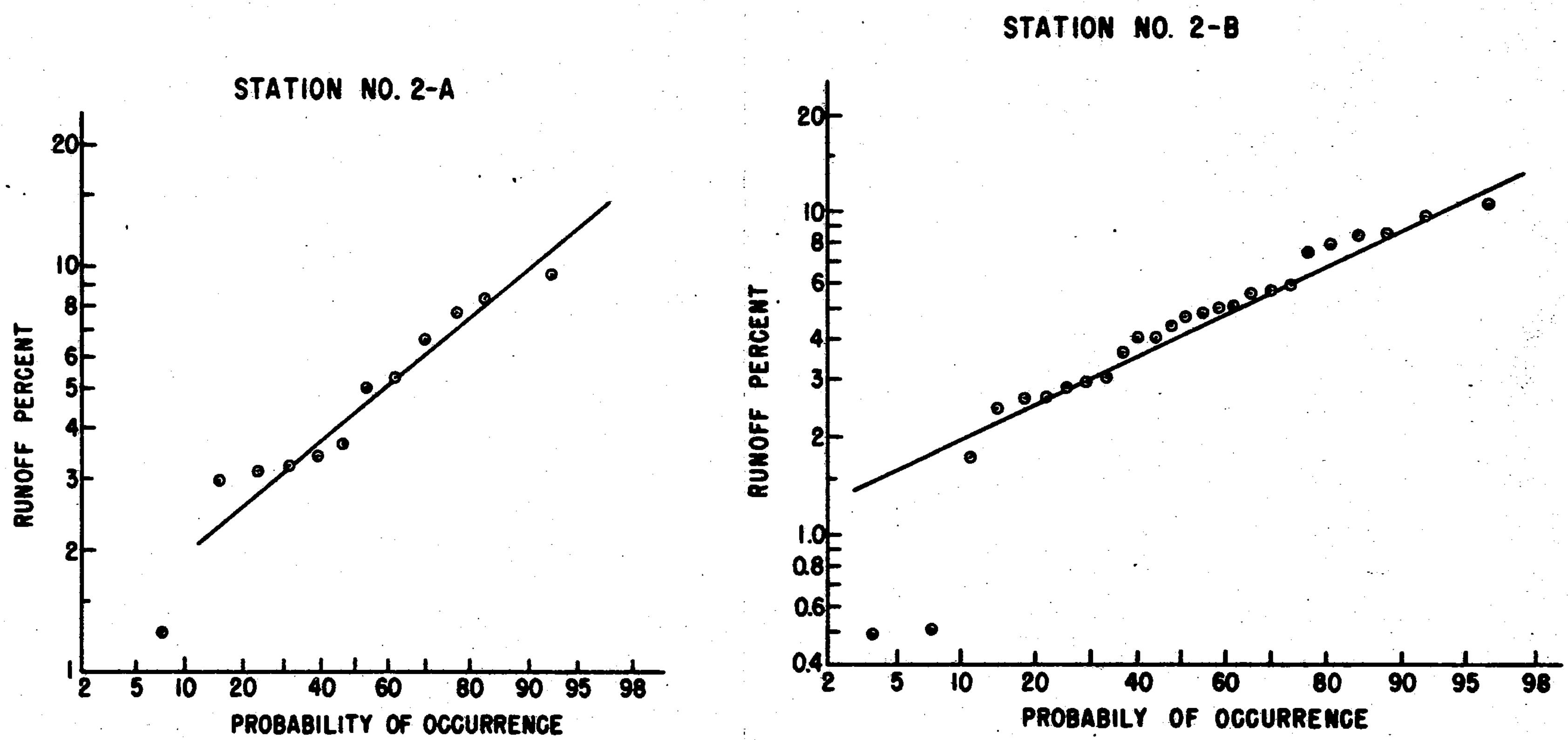


Figure 19. Log-Probability Relationships for Stations 2-A and 2-B

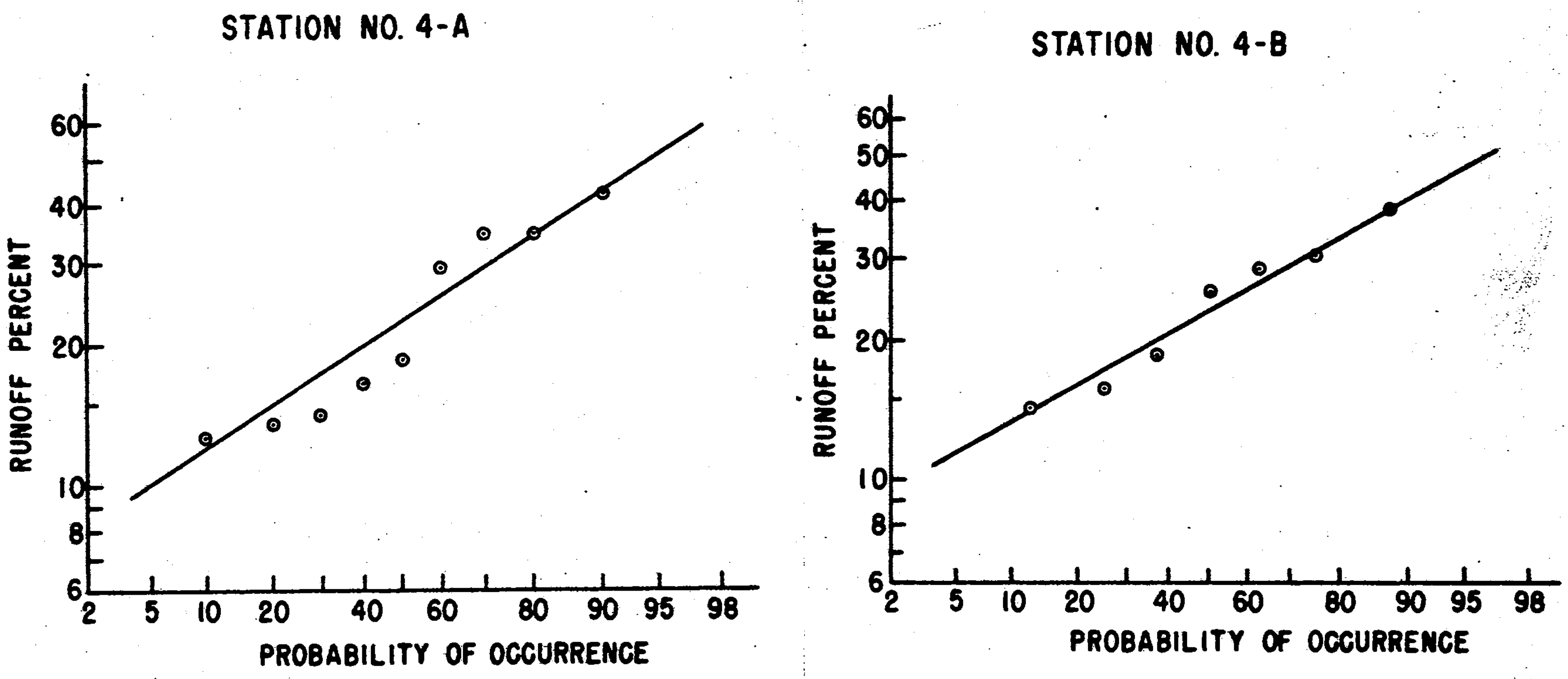


Figure 20. Log-Probability Relationships for Stations No. 4-A and 4-B

APPENDIX E

STORAGE ROUTING CURVES FOR OVERFLOW AND UNUSED PUMP CAPACITY

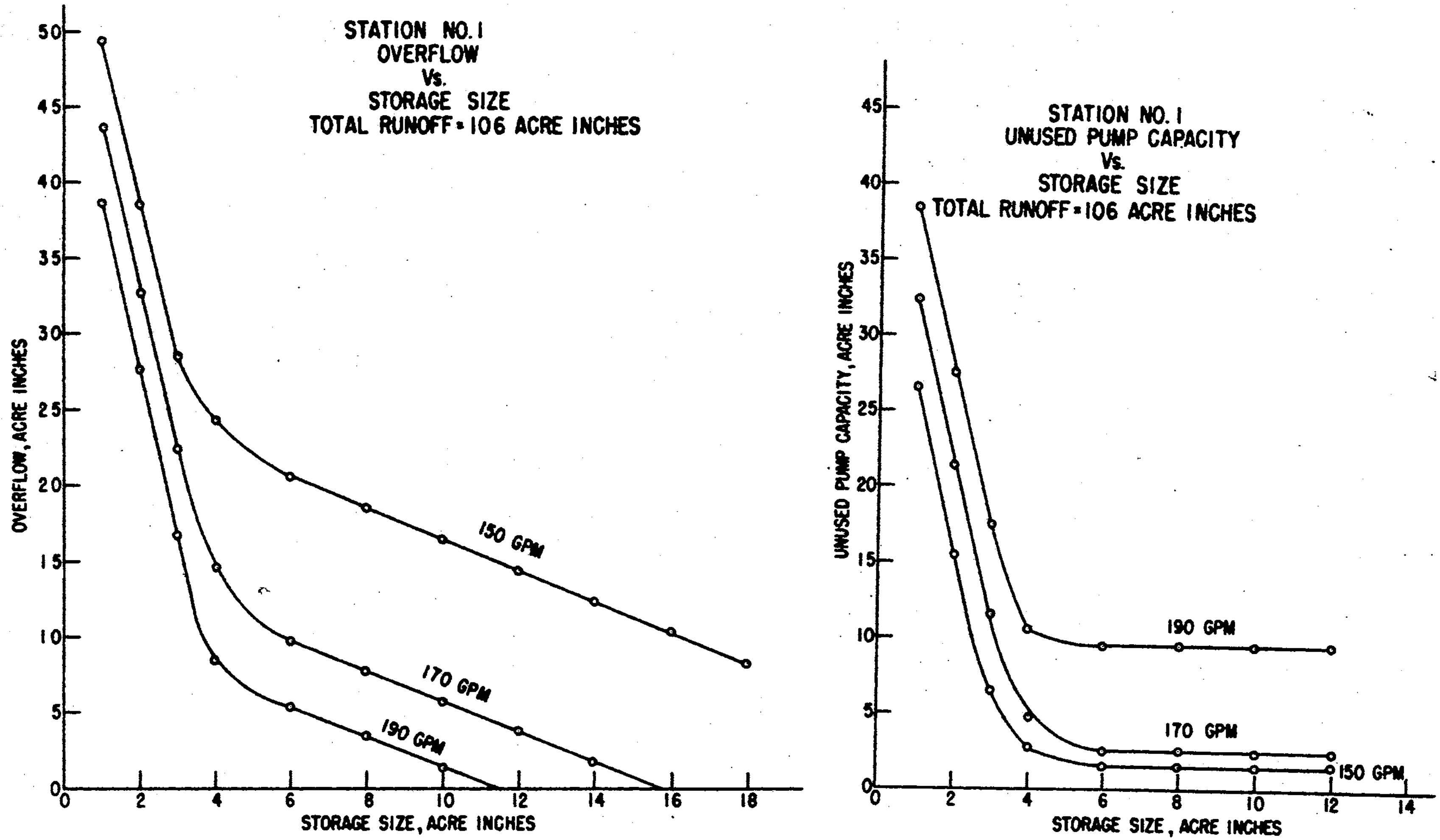
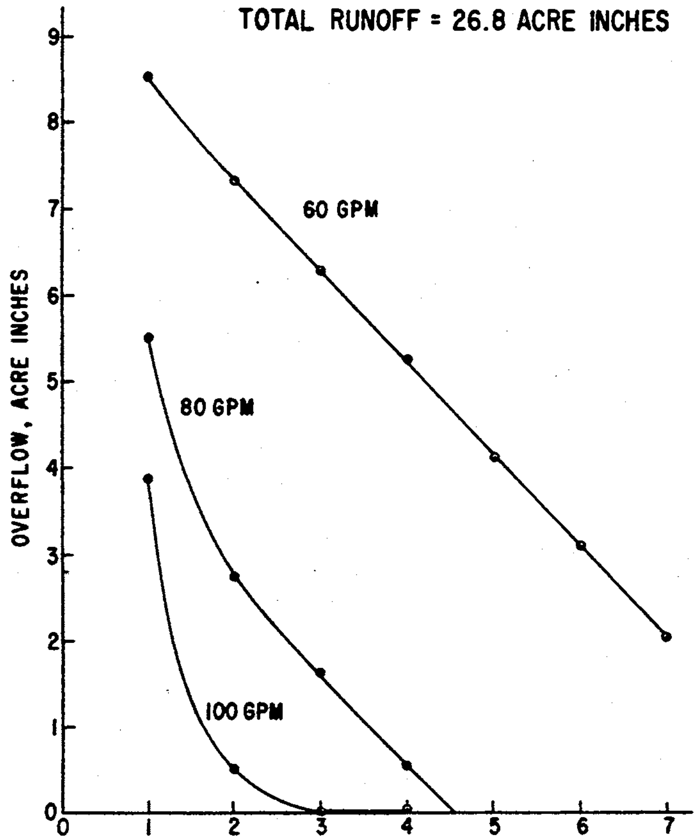


Figure 21. Overflow and Unused Pump Capacity for Station No. 1



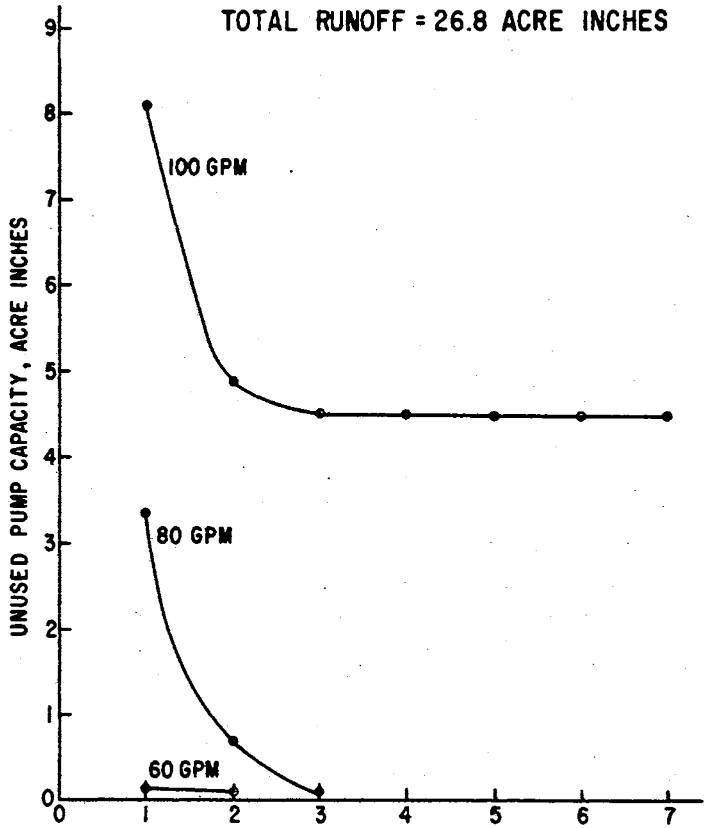


Figure 22. Overflow and Unused Pump Capacity for Station No. 2-A

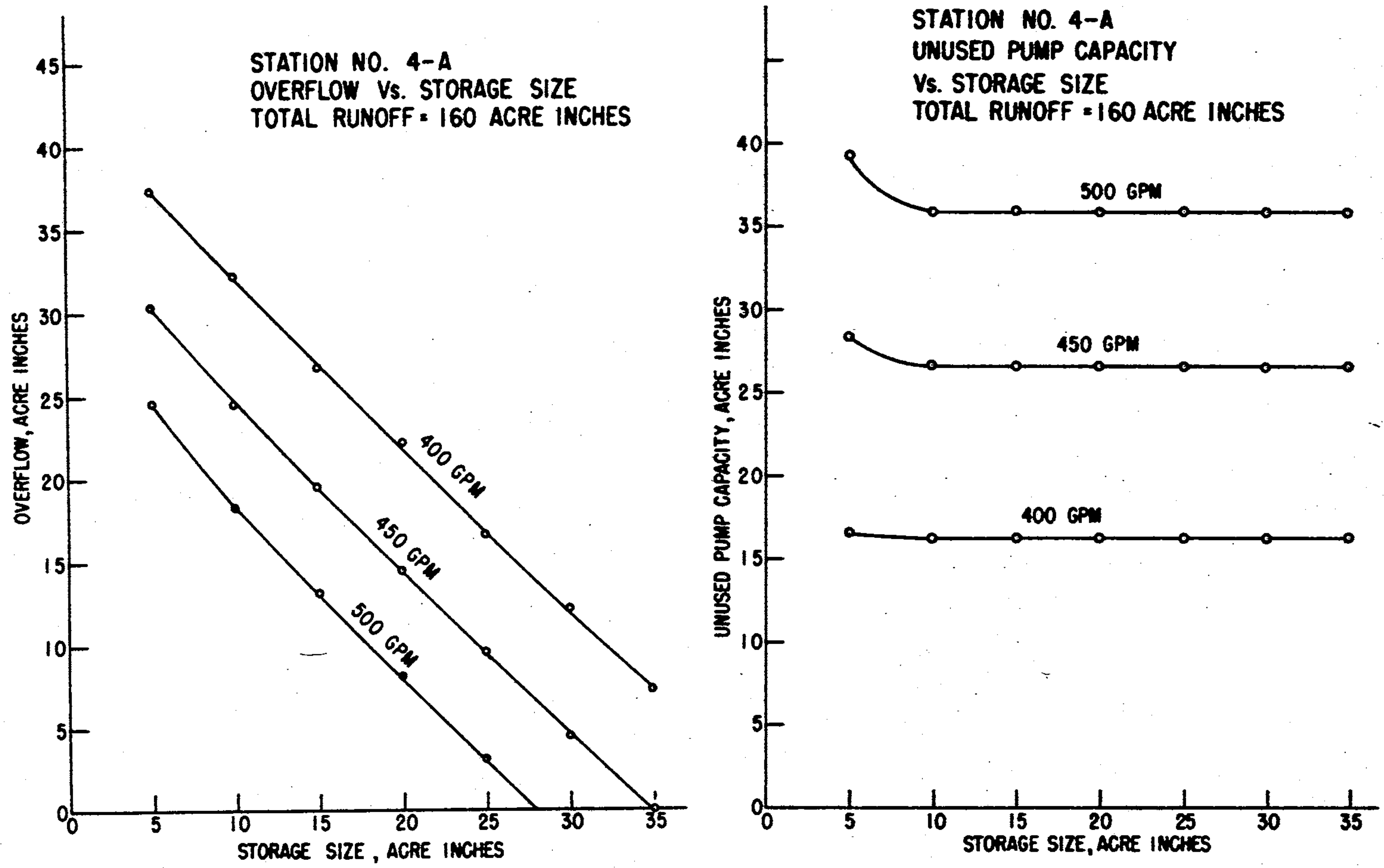


Figure 23. Overflow and Unused Pump Capacity for Station No. 4-A

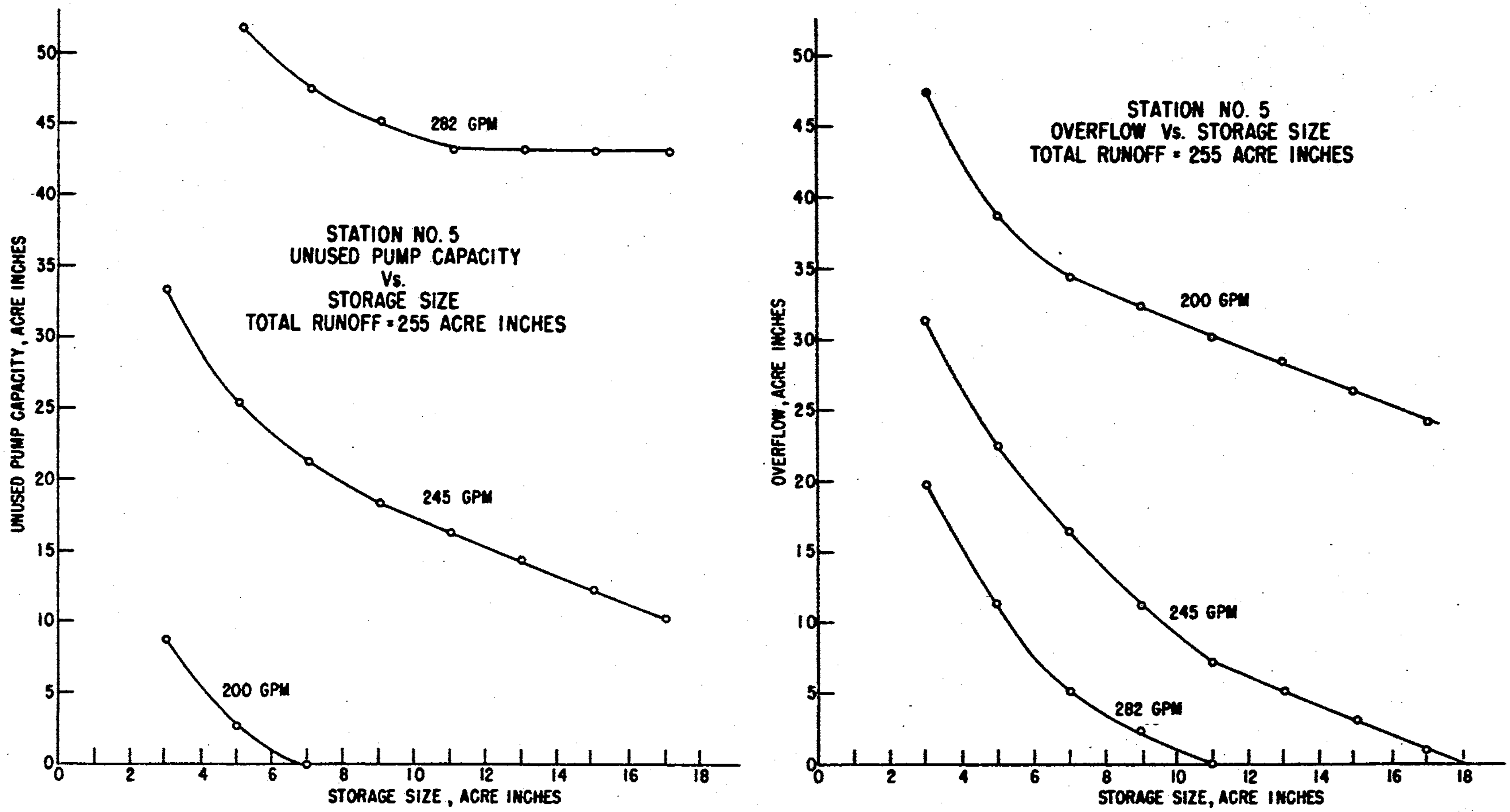


Figure 24. Overflow and Unused Pump Capacity for Station No. 5

VITA

David Lee Pope

Candidate for the Degree of

Master of Science

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