HYDROLOGIC CHARACTERISTICS

OF A

SEMIARID WATERSHED

by

David Arthur Woolhiser

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STATEMENT BY AUTHOR

SIGNED: David A. Woolhiser

APPROVAL BY THESIS DIRECTOR

This thesis has been approved on the date shown below:

Quentin M. Mees
Quentin M. Mees
Associate Professor of
Civil Engineering

Feb. 7, 1959
Date

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INTRODUCTION

1.1 General Discussion

Many western states have overdeveloped their ground-water supplies, and the water levels of many of the major ground-water basins are dropping at alarming rates. This rapid depletion of a very valuable natural resource can be stopped only by decreasing the pumping rate or by increasing the recharge rate.

Artificial recharge offers one possibility of increasing ground-water supplies in the semiarid areas of the West. Before recharge installations can be designed intelligently and economically, however, it is necessary to know the hydrologic characteristics of the area involved. At present, very little is known about the quantity, quality, and disposition of runoff in these semiarid regions.

Most of the hydrologic research in the Southwest has been conducted by the Soil Conservation Service and, more recently, by the Agricultural Research Service. The studies of the Soil Conservation Service were confined to plots or very small watersheds. The Agricultural Research Service is investigating the hydrology of an area of about sixty square miles near Tombstone, Arizona, but few published data are available. The U.S. Geological Survey has been measuring runoff from very large watersheds for many years, but has no information on the smaller areas that have been considered as possible sources of runoff for recharge purposes.

In the spring of 1955, the Agricultural Engineering Department of the University of Arizona initiated a project entitled "Hydrology and Water Utilization of Small, Semiarid Drainage Areas." The objectives of the project were:

1. To determine the amount of surface runoff available for diversion or storage.
2. To evaluate the hydrologic characteristics of small, semiarid watersheds.
3. To provide a basis for estimating the cost of recharging ground-water reservoirs with flood waters.

The Atterbury Reservoir Watershed was selected for this hydrologic study.

1.2 Scope of Thesis

The purpose of this thesis is to describe the instrumentation of the Atterbury Reservoir Watershed Project, and to present an analysis of the hydrologic data which have been collected since 1955.

Chapter 2

PHYSICAL DESCRIPTION OF THE ATTERBURY RESERVOIR WATERSHED

2.1 Location

The Atterbury Reservoir Watershed is located about ten miles east of the city of Tucson in southern Arizona (Figure 2.1). It is within the drainage area of the upper Santa Cruz River in the geologic area of Arizona known as the basin and range province. The topography of this area is typical of central and southern Arizona, and is characterized by mountains and valleys which have been formed by block faulting. Subsequent erosion of the mountains has partly filled the valleys, and this eroded material formed the valley slope on which the Atterbury Reservoir Watershed is located.

2.2 Climate

Climate exerts a controlling or limiting influence on the hydrology of a particular region. For this reason, it is necessary to understand the meteorological conditions which govern the supply of rainfall to a watershed before one can draw reliable conclusions about runoff rates and volumes.

The climate of Tucson is classified as semiarid.

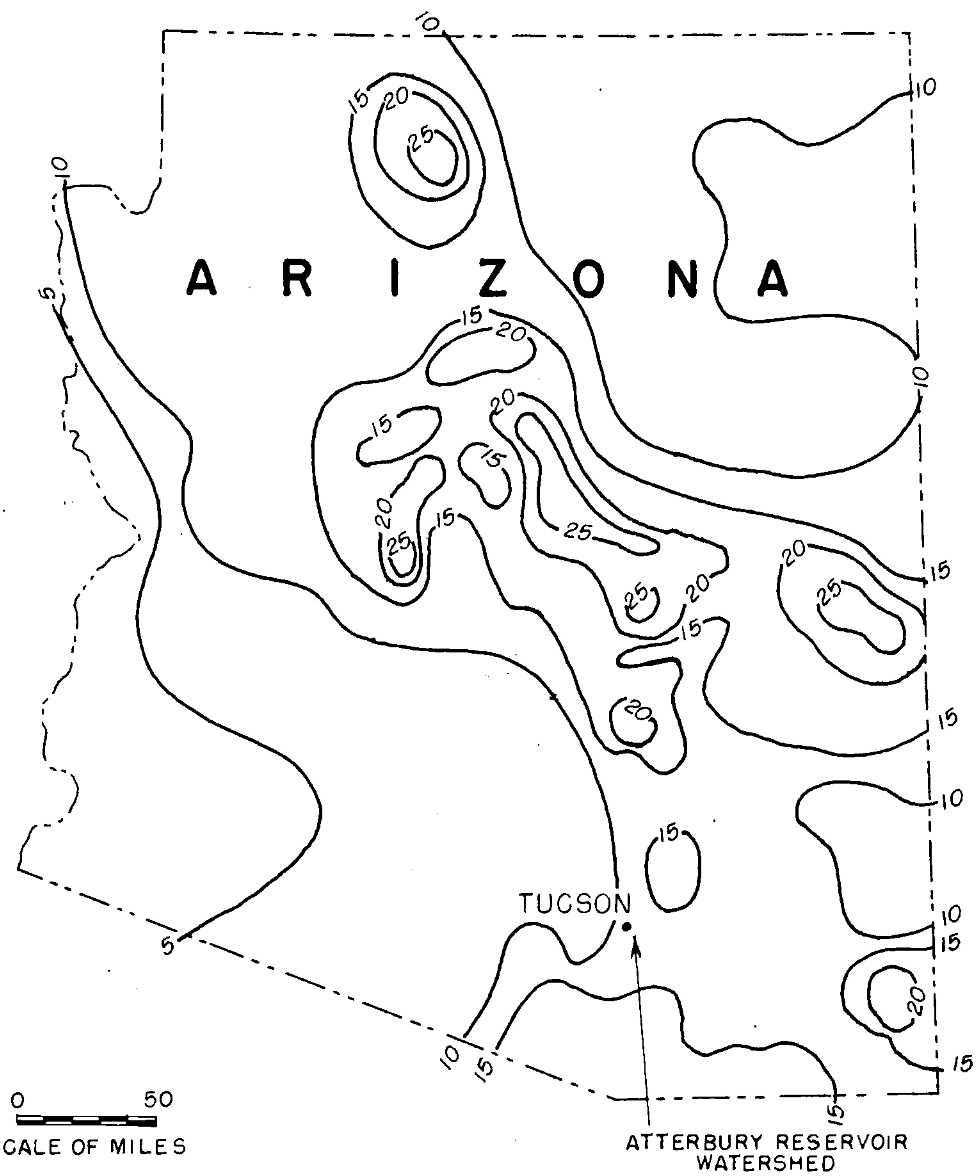

FIGURE 2.1

LOCATION SKETCH AND ISOHYETAL MAP OF ANNUAL PRECIPITATION FOR ARIZONA*

*H. V. SMITH, CLIMATE OF ARIZONA UNIVERSITY OF ARIZONA AGRICULTURAL EXPERIMENT STATION BULLETIN NO. 279 (TUCSON, ARIZONA, 1956).

The average mean temperature at the University of Arizona is 67.4° F., and the long term average annual rainfall at the University of Arizona is 10.83 inches. The index of wetness, which is the ratio of the rainfall in a particular year to the average annual rainfall, varies from 0.48 to 2.2. This extreme variability of the index of wetness is characteristic of semiarid and arid regions.

Arizona has two distinct rainy seasons. One season occurs during the winter months of December, January, and February; and the other during the summer months of July, August, and September.

The winter rains are the result of frontal activity or orographic uplift of Polar Pacific air masses.[1] These rains are normally of low intensity and may be of long duration. They are the most important source of runoff for the reservoirs in the mountain areas, and the resulting stream flow is the largest source of ground-water recharge.

The moisture source of the summer rains is air from the Gulf of Mexico which invades the southwestern United States in late June or early July. Updrafts caused by orographic lifting or thermal convection form thunderstorm cells. The resulting rain is typically of high intensity

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[1] J. H. Dorroh, Jr., Certain Hydrologic and Climatic Characteristics of the Southwest, The University of New Mexico Publications in Engineering No. 1 (Albuquerque: University of New Mexico Press, 1946), p. 2.

and short duration, and usually covers an area of less than twenty-five square miles. These storms cause very little runoff in the mountain areas, but are the cause of destructive "flash floods" on the valley slopes.

2.3 Topography and Drainage

The Atterbury Reservoir Watershed is about eighteen square miles in area. It is drained by two major ephemeral streams, Davis-Monthan Wash and Main Wash (Figure 2.2). Davis-Monthan Wash is an artificial interceptor channel which has been built to prevent flooding to the west of the watershed. Both Davis-Monthan Wash and Main Wash contribute runoff to the Atterbury Reservoir, which has a capacity of around four hundred acre feet. There is no evidence that the reservoir has spilled since it was built around 1920. There are three small reservoirs, or "tanks," located on the Main Wash. Tank No. 1 has a capacity of only 0.3 acre feet which is not significant; Tank No. 2 and Tank No. 3 have capacities of 29.8 and 7.0 acre feet respectively.

The topography of the area is illustrated by the topographic map and Figures 2.3, 2.4, and 2.5. Some of the topographic parameters other than area which are used in hydrologic analysis are : average landslope, drainage density, channel slope, and shape of basin.

The average landslope, S, may be computed from

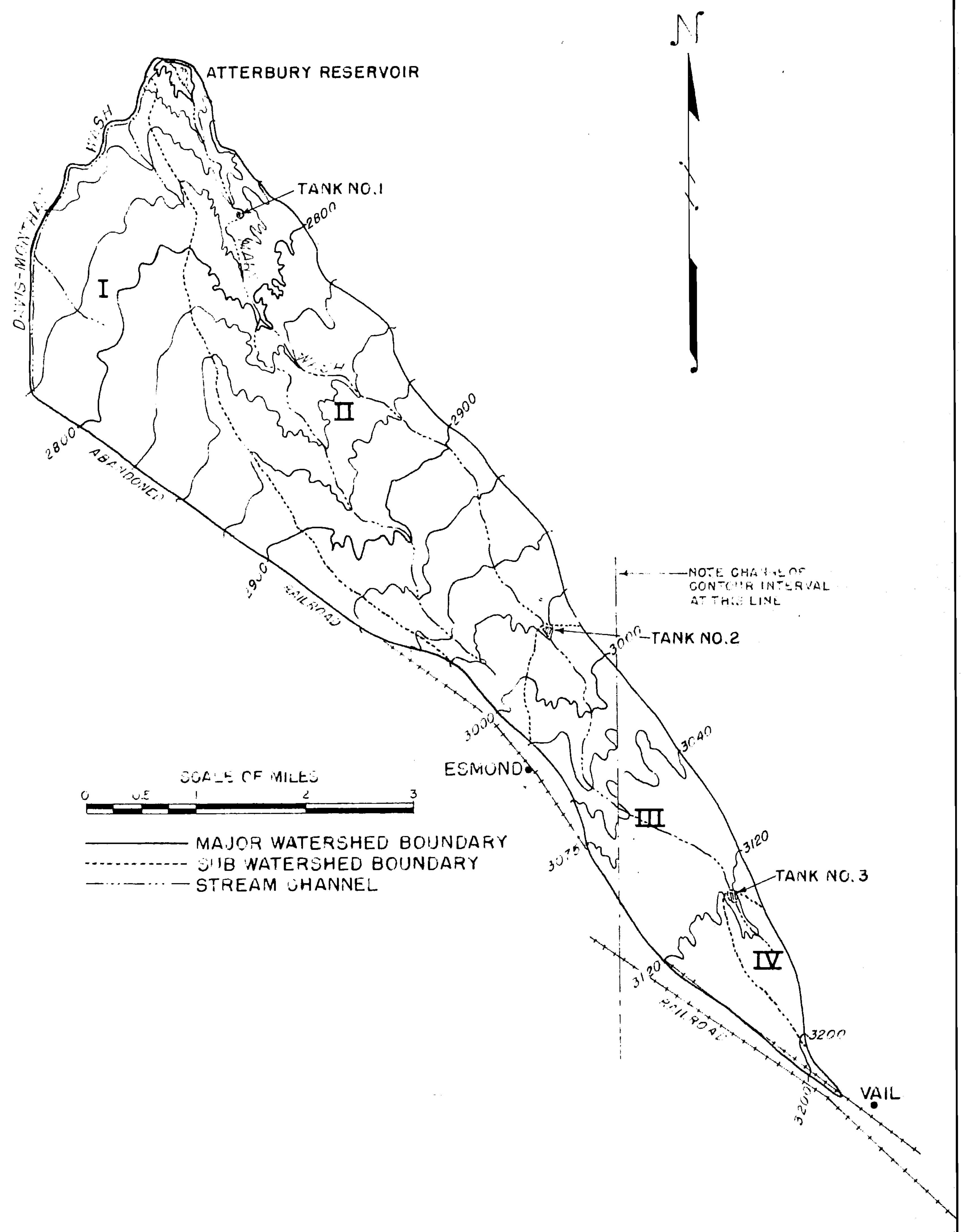

FIGURE 2.2

TOPOGRAPHIC MAP
OF
ATTERBURY RESERVOIR WATERSHED

FIGURE 2.3

LEVEL TOPOGRAPHY IN SUBWATERSHED I

FIGURE 2.4

ROUGH AND BROKEN LAND IN
LOWER SUBWATERSHED II

FIGURE 2.5

ROLLING TOPOGRAPHY TYPICAL OF
SUBWATERSHEDS III AND IV

measurements taken from a contour map by the relationship[2]

$$S = \frac{DL}{A} \tag{2.1}$$

where:

D = contour interval in feet

L = total length of contours in feet

A = Area of basin in square feet

The drainage density is expressed as the length of stream channel per unit of area. Let D_d represent the drainage density, L the total length of stream channel in the basin, and A the area; then

$$D_d = \frac{L}{A} \tag{2.2}$$

The drainage density is usually expressed in feet per acre. The channel length, L, may be measured from topographic maps or aerial photographs. Drainage density is a measure of the distance of overland flow, and exerts an important influence on the shape of a runoff hydrograph.

Channel slope also has an effect on the shape of the hydrograph, but the average slope of the basin has given better correlations for semiarid regions.[3]

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[2]C. O. Wisler and E. F. Brater, Hydrology (New York: John Wiley & Sons, Inc., 1949), P. 49.

[3]R. B. Hickock, R. V. Keppel and B. R. Rafferty, "Use of Watershed Characteristics in Hydrograph Synthesis for Small Arid Land Watersheds" (Paper presented at the summer meeting of the American Society of Agricultural Engineers in Santa Barbara, California, June 23, 1958).

Wisler and Brater[4] discuss two measures of basin shape which were first proposed by Gravelius. The "form factor" is defined as the ratio of the average width to the axial length of the basin. The "compactness coefficient" is the ratio of the perimeter of the watershed to the circumference of a circle whose area is equal to that of the drainage basin. Thus

$$K_c = \frac{P_1}{2\sqrt{\pi A}} = 0.28 \frac{P_1}{\sqrt{A}} \qquad (2.3)$$

where:

K_c = compactness coefficient

P_1 = perimeter of basin

A = area of basin

The preceding topographic parameters have been computed for the Atterbury Reservoir Watershed and its subwatersheds, and are shown in Table I.

2.4 Soils

The ability of a drainage basin to absorb the precipitation that falls on it, is of primary importance in determining the resulting stream-flow hydrograph. The absorption ability of a basin involves the process of infiltration. Infiltration is the process whereby water enters the surface stratum of the soil, while infiltration capacity

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[4] Wisler and Brater, op. cit., p. 45.

TABLE I

TOPOGRAPHIC PARAMETERS
ATTERBURY RESERVOIR WATERSHED

Parameter	Atterbury Reservoir Watershed	Subwatershed I	II	III	IV
Average Slope, S (%)	1.8	1.2	2.1	2.6	3.7
Drainage Density D_d* (ft/acre)	0.12	0.11	0.13	0.11	0.15
Channel Slope (%)*	0.84	0.62	0.86	0.81	1.3
Form Factor	0.14	0.15	0.10	0.16	0.19
Compactness Coeff. K_c	1.6	1.7	1.8	1.4	1.3
Area in Square Miles	18.0	5.3	12.7	4.3	0.5

*Channel length was measured from a U.S.G.S. topographic map with a scale of 1:62,500. Only channels which were shown as broken blue lines were measured.

is the maximum rate at which a soil is capable of absorbing water in this manner.

Infiltration capacity is determined by the size of interstices of the surface layer of soil. For this reason, the infiltration capacity of a basin has often been correlated with standard agricultural soil type classifications.

A soil survey of the Atterbury Reservoir Watershed was discontinued when it became apparent that the standard agricultural soil classifications did not adequately describe the infiltration characteristics of the soil. The investigator found that soils at different sites often would be included in the same soil series, even though it was obvious from observations of the surface condition that the infiltration capacities would not be the same. It is the investigator's opinion that where runoff-producing rainfall is of short duration, and where vegetation is so sparse that it has very little effect on the surface condition of the soil, the surface stratum of soil and the antecedent rainfall exert a controlling influence on infiltration capacity. It would seem that a soil classification procedure based on the surface condition of the soil would be helpful in evaluating the runoff characteristics of arid and semiarid lands.

During the reconnaisance for the soil survey, the soil series found on the basin were identified as Tubac, Mohave, Laveen, Pinal, and Tucson.[5] The textural classifications ranged from sandy loam to clay loam.

Beutner, Gaebe, and Horton[6] performed infiltration experiments on some of these soils and also found wide variation of infiltration capacities of soils within the same series at different sites. For comparative purposes, the infiltration capacity curves for two of the soils found on the Atterbury Reservoir Watershed are shown in Figure 2.6.

In general, the soils of Subwatershed Number I have the lowest infiltration capacity, probably in the range of Tucson Loam. The average infiltration capacity of the remainder of the watershed is probably near that of Laveen Loam.

2.5 Vegetation

The vegetation on most of the Atterbury Reservoir Watershed is very sparse and offers very little protection to the soil. Interception by the vegetative canopy is negligible; and the vegetation has little retarding effect on

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[5]F. O. Youngs, A. T. Sweet, A. T. Strahorn, T. W. Glassey, and E. N. Poulson, *Soil Survey of the Tucson Area, Arizona*, United States Department of Agriculture, Bureau of Chemistry and Soils, No. 19 (Washington: Government Printing Office, 1931).

[6]E. L. Beutner, R. R. Gaebe, and R. E. Horton, *Sprinkled Plat Runoff and Infiltration Experiments on Arizona Desert Soils*, Soil Conservation Service Technical Paper 38 (September, 1940).

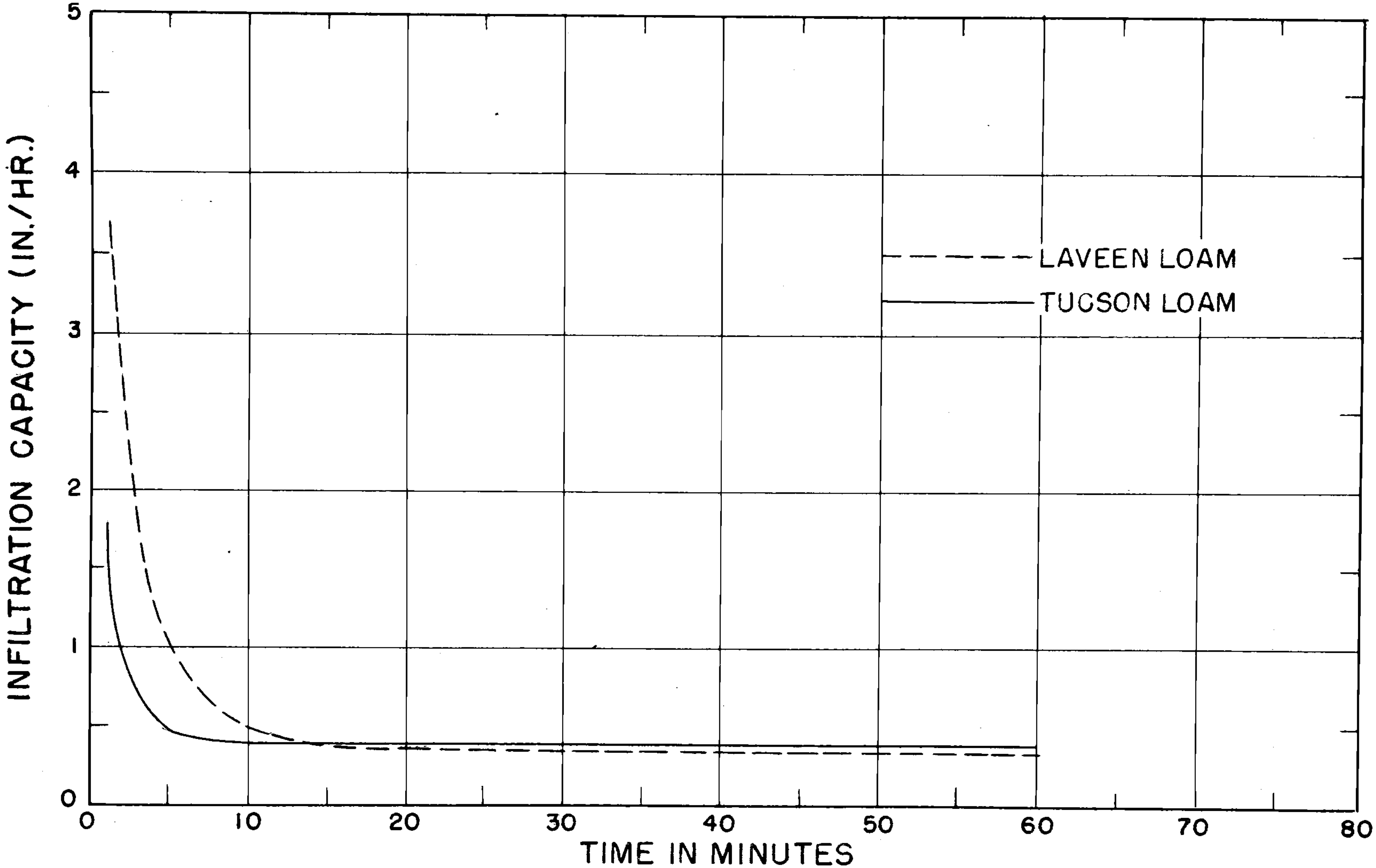

FIGURE 2.6

VARIATION OF INFILTRATION CAPACITY WITH TIME FOR REPRESENTATIVE SOIL TYPES *

* Yost and Gardner, Engineers. STORM DRAINAGE REPORT FOR CITY OF TUCSON, ARIZ. Book One (Mar. 1955) p.9

surface flow except in wide, grassy swales, or where vegetative trash forms debris dams.

Four major plant groupings were found within the drainage basin. These groups seemed to be associated directly with soil types and indirectly with topography. The groupings are:

1. Creosote bush (Covillea tridentata) dominant (Figure 2.7). Scattered mesquite (Prosopis velutina) and cacti are also found in this group. There is often very sparse grass cover. This group is associated with highly calcareous soils, and covers by far the largest area of Subwatersheds II, III, and IV.
2. Cacti dominant (Figure 2.8). This group consists mostly of prickley pear cactus (Opuntia sp.) and cholla cactus with a few mesquite and palo verde trees. There is some annual grass and alfilaria. The vegetation of Subwatershed I is composed almost entirely of this group.
3. Palo verde (Cercidium microphyllum and Parkinsonia torreyana) dominant (Figure 2.9). This group includes some creosote bush and cacti, mostly saguaro. There is very little annual grass. Group 3 vegetation is found in a strip

FIGURE 2.7

GROUP 1 VEGETATION--CREOSOTE BUSH DOMINANT

FIGURE 2.8

GROUP 2 VEGETATION--CACTI DOMINANT

FIGURE 2.9

GROUP 3 VEGETATION--PALO VERDE DOMINANT

FIGURE 2.10

UPSTREAM CHANNEL VEGETATION

FIGURE 2.11

DOWNSTREAM CHANNEL VEGETATION

northeast of Main Wash extending from east of Tank 2 to Atterbury Reservoir.

4. Channel vegetation (Figures 2.10 and 2.11). In the upper reaches of the watershed where channels are often wide and flat, grass is dominant. Mesquite is dominant in the lower reaches. This channel vegetation group is the most significant in hydrologic studies of semiarid regions.

Chapter 3

INSTRUMENTATION OF THE ATTERBURY RESERVOIR WATERSHED

3.1 Rain Gages

The location of standard and recording rain gages is shown in Figure 3.1. Twenty-seven standard and two recording gages, R-1 and R-23, were installed in July and August, 1955. The recording gage R-30 was installed in 1957.

The support for the standard rain gages consisted of half of a steel tee-post, to which the gage was clamped with two sheet metal straps (Figure 3.2). This support has proved to be satisfactory in this area. It was initially inexpensive, and its sturdy construction prevented cattle from upsetting the gages and also discouraged theft. Since the measuring tubes held only two inches of rain, the gage had to be dismantled when more than two inches of rain fell in a single storm. This was a time-consuming operation, and would make the support impractical where two-inch storms are a common occurrence. In this area the recurrence interval for a two-inch storm at a point is around six years, so it was necessary to dismantle an average of only four or five gages each year.

The recording gages were twelve-inch, singly reversing, weighing rain gages manufactured by the Instruments

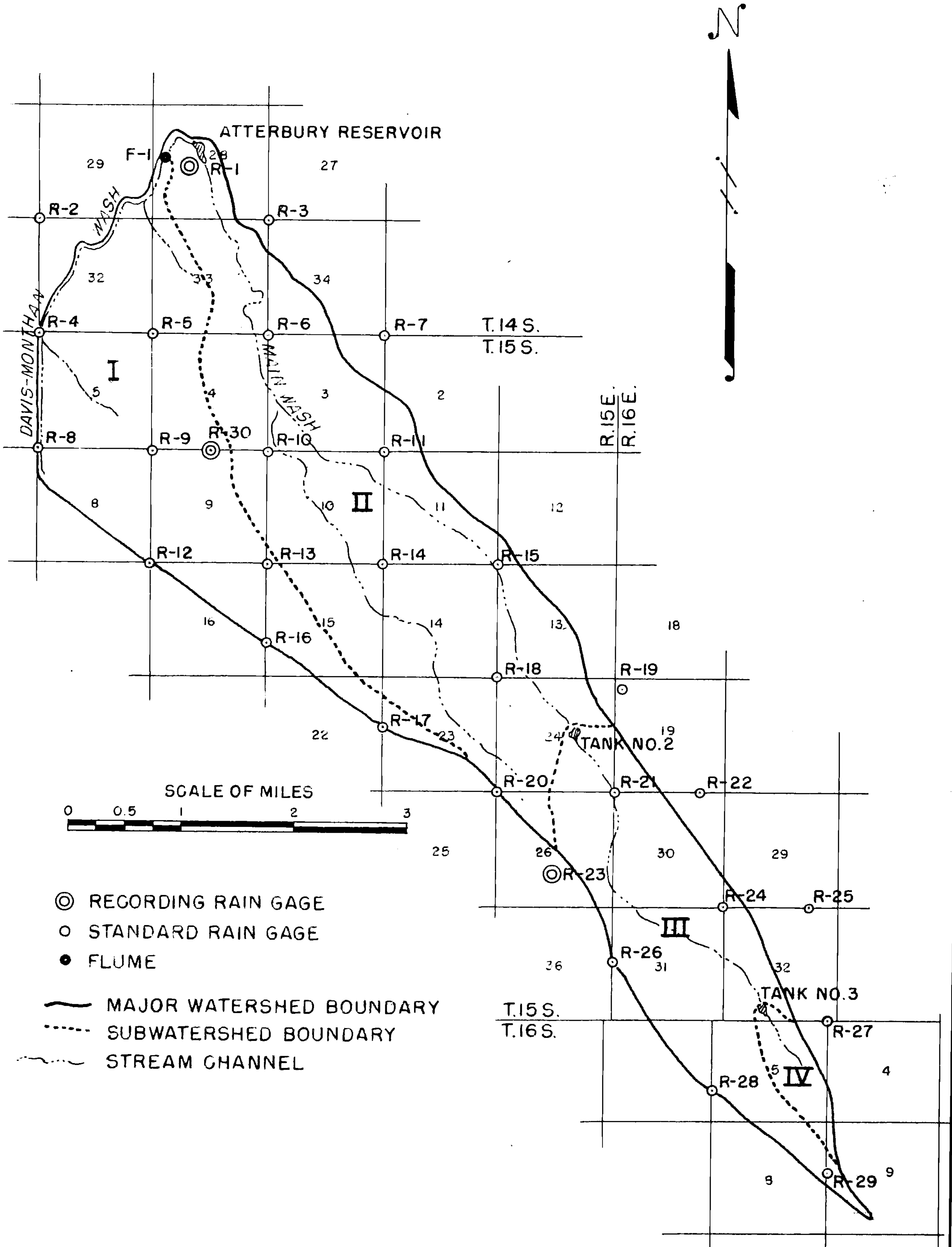

FIGURE 3.1

INSTRUMENTATION
OF
ATTERBURY RESERVOIR WATERSHED

FIGURE 3.2

STANDARD RAIN GAGE AND SUPPORT

Corporation of Baltimore, Maryland. The gages were equipped with eight-day clocks. During the summer period the gages were set for a 24-hour drum rotation, and in the winter period the gears were changed to provide 192-hour rotation. This arrangement made it possible to make accurate measurements of the intensity of thunderstorm rainfall when charts were changed after each storm, yet provided a legible record of winter rainfall when charts were changed weekly.

The rain gage network used on this watershed was probably of sufficient density to delineate the rainfall patterns produced by convectional thunderstorms. A sample network of gages on a one-fourth mile grid, however, would be helpful in determining the errors involved in using a one-mile grid.

3.2 Volumetric Measurement of Runoff

Runoff from the Atterbury Reservoir Watershed was measured volumetrically in three reservoirs. Accurate transit and stadia surveys of the reservoirs were made, and contour maps were prepared with a contour interval of one foot. Elevation-area and elevation-volume curves were prepared by measuring contour areas with a planimeter and computing the volume between adjacent contours by the average end area method. The curves showing area and volume of each reservoir as functions of elevation, are shown in Appendix A. Stilling wells were completed in the spring of 1956, and water level recorders were installed at the same time (Figure 3.3). The water level records for Atterbury Reservoir and Tank No. 2 are excellent. Records for Tank No. 3 should be classified as good, because of breaks in the record caused by silting of the inlet to the stilling well and recorder malfunction. The inlet was improved, and a new recorder was installed in July of 1958. Under ordinary conditions, the error in the volumetric measurement of runoff from the watershed is probably within five percent.

Inflow hydrographs were computed from the reservoir-stage records. The determination of peak discharge was subject to large errors because of the short duration of the peak.

It was found that the Atterbury Reservoir lost a significant amount of water by percolation during the rising stage. A correction curve (Figure 3.4) was prepared by extrapolation of the loss curve immediately after inflow had stopped. This curve was verified by measurement of the flow at Flume 1, when only Subwatershed Number I was contributing water to the reservoir.

3.3 Stream-Flow Measurement

Satisfactory stage-discharge relationships for natural channels in semiarid regions, cannot be obtained because of the "flashy" nature of the stream-flow and the absence of good natural controls. Since current meter calibration of

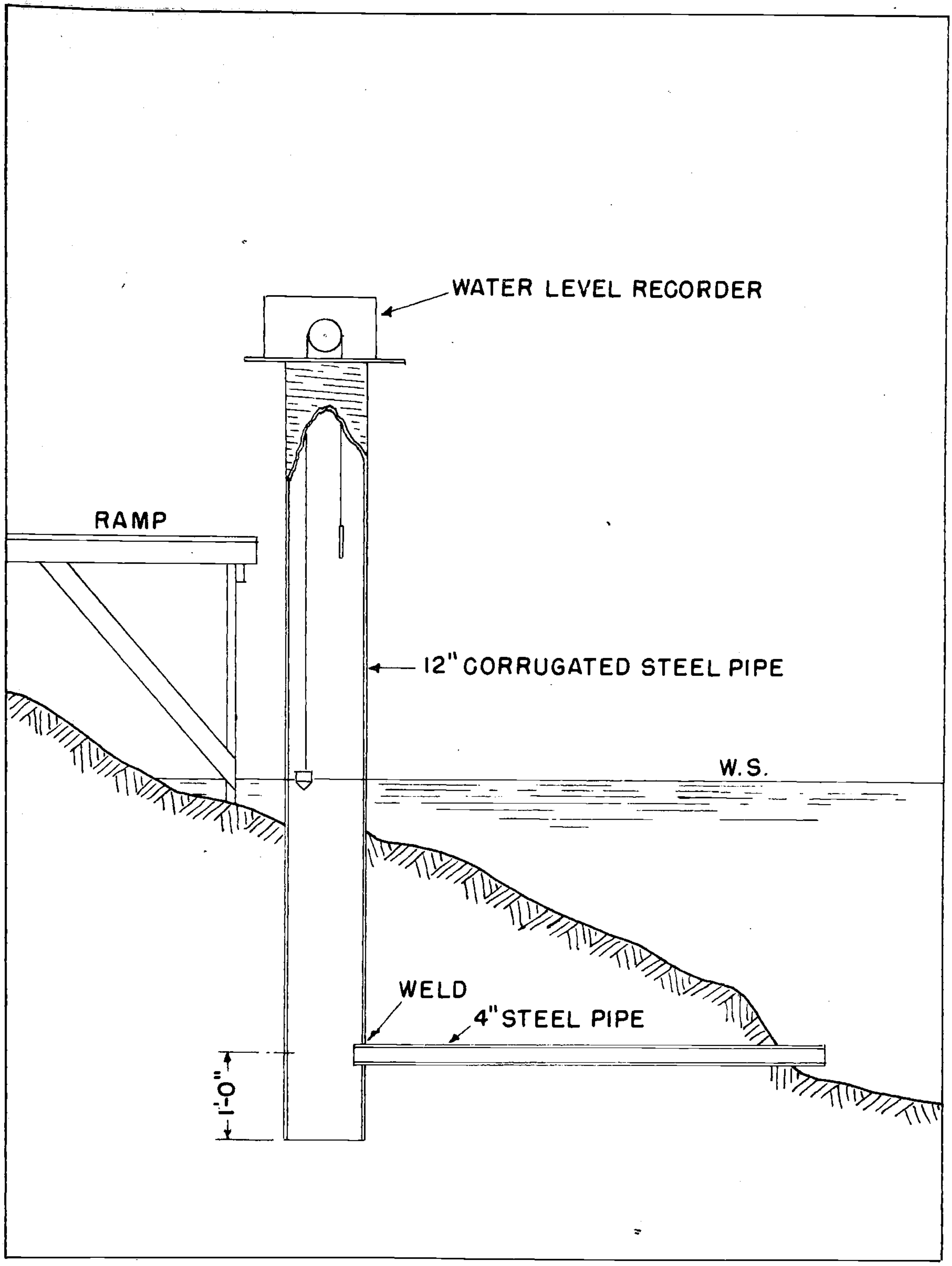

FIGURE 3.3

TYPICAL STILLING WELL INSTALLATION

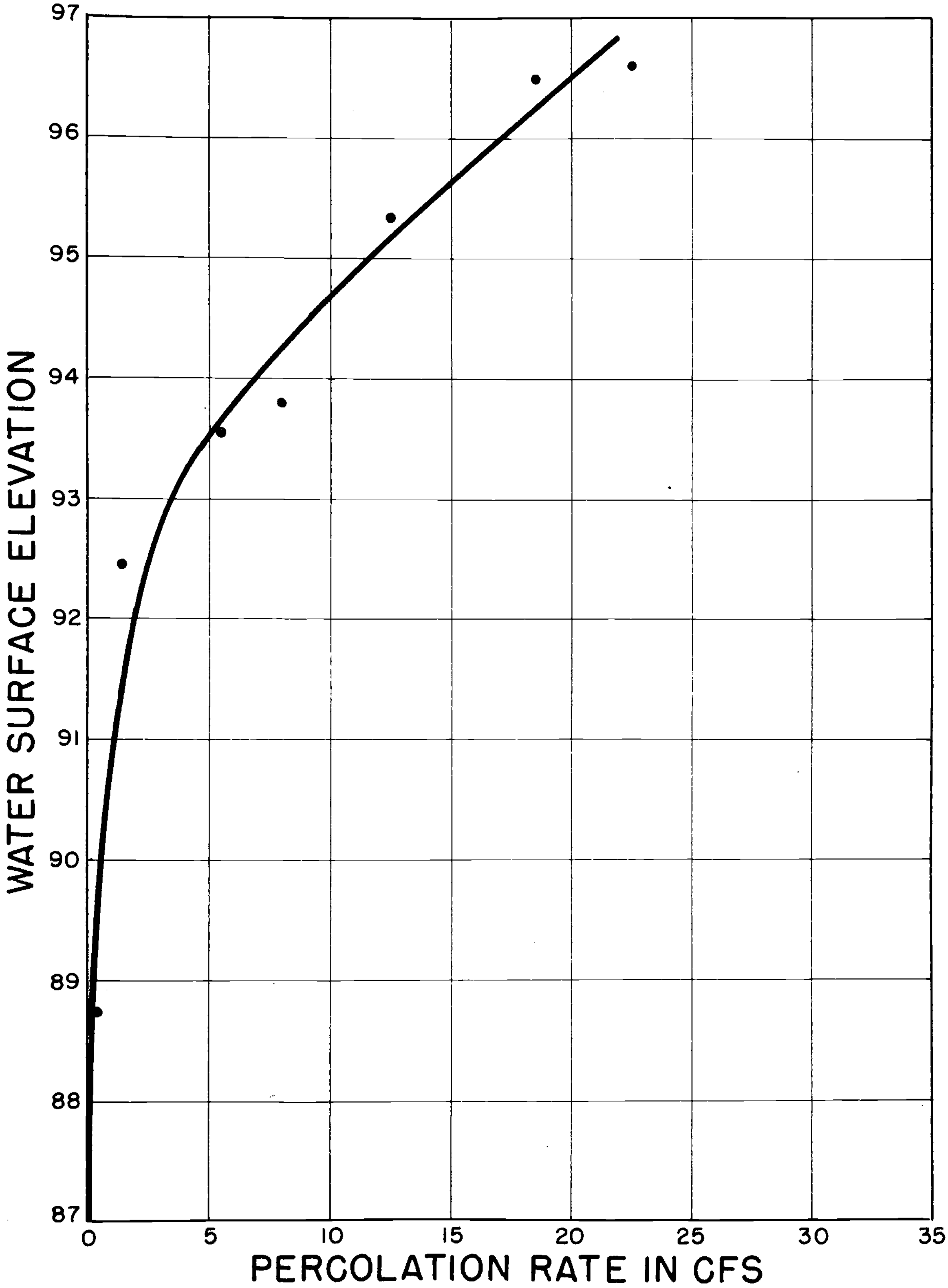

FIGURE 3.4–INFLOW CORRECTION CURVE FOR ATTERBURY RESERVOIR*

*RESERVOIR PREVIOUSLY UNWETTED.

an artificial control may require many years, it is desirable to build a structure which has a known relationship between gage height and discharge.

The ideal flow-measuring structure for semiarid regions would be: (1) accurate over wide ranges of discharge, (2) very similar to the natural geometry of the channel in which it is built, and (3) inexpensive. The first requirement is clearly apparent; accurate measurement at all stages of flow is necessary in experimental studies. The second requirement is dictated by considerations of sediment movement and energy dissipation. Any structure which contracts a channel or raises the channel grade, causes a reduction of velocities upstream and a subsequent deposition of sediment. If gage height is measured in these sections, the sediment interferes with the operation of the stilling well or destroys the stage-discharge relationship. A channel contraction also causes increased velocities downstream. This added kinetic energy must be dissipated in some manner, or serious erosion may result. The cost of the flow-measuring device is governed largely by the size of the channel and the degree to which the second requirement is met, since an energy-dissipating structure may be larger than the measuring section.

It is always desirable to use standard measuring devices for experimental work. It was found, however, that the accepted flow-measuring devices were not suitable for installation on the Atterbury Reservoir Watershed, because they did not meet one or more of the previously mentioned requirements. The Parshall Flume[7] was rejected because it is sensitive to the deposition of silt in the measuring section[8] and is not accurate over the required range of discharge. The San Dimas Flume[9] was rejected because its rectangular cross-section makes it insensitive to low flows and increases the cost of construction. Both broad-crested and sharp-crested weirs are subject to sediment deposition.

Since no suitable standard flow-measuring device was available, it was decided that a flume operating on the critical-depth principle would be the best solution. A trapezoidal, vee-bottom cross-section was used to increase sensitivity, and the measuring section was located in a region of critical or super-critical velocities to prevent silting.

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[7] R. L. Parshall, "The Improved Venturi Flume," Transactions of the American Society of Civil Engineers, Vol. 89 (1926).

[8] H. G. Wilm, John S. Cotton, and H. C. Storey, "Measurement of Debris-Laden Stream Flow with Critical-Depth Flumes," Transactions of the American Society of Civil Engineers, Vol. 103 (1938), p. 1237.

[9] Ibid.

3.4 The Critical-Depth Flume

The criterion for flow at critical depth in any channel is[10]

$$\frac{dH_c}{dD} = 0 \tag{3.1}$$

where

H_c = specific energy, which is the sum of the average depth, D, and the velocity head, $\frac{v^2}{2g}$

Now the cross-sectional area, a, is some function of the average depth

$$a = f(D)$$

and

$$Q = av = f(D)v$$

where

Q = volume rate of flow.

Substituting

$$H_c = D + \frac{v^2}{2g} = D + \frac{Q^2}{2ga^2}$$

$$= D + \frac{Q^2}{2g\,[f(D)]^2}$$

and differentiating both sides

$$dH_c = dD - \frac{Q^2}{g}\,\frac{f^1(D)}{[f(D)]^3} \tag{3.2}$$

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[10] H. W. King, Handbook of Hydraulics, revised by E. F. Brater (fourth edition; New York: McGraw-Hill, 1954), p. 8-7.

Now

$$f^1(D) = da \text{ and } da = TdD$$

Where T is the top width of the channel. Substituting these relationships into equation (3.2) we have

$$dH_c = dD - \frac{Q^2}{g}\frac{TdD}{a^3}$$

$$\frac{dH_c}{dD} = 1 - \frac{Q^2T}{ga^3} = 0$$

$$Q^2T = ga^3$$

or

$$\frac{Q^2}{g} = \frac{a^3}{T} \qquad (3.3)$$

Equation (3.3) states a necessary condition for open channel flow at critical depth.

The following is a derivation of a general equation for flow at critical depth in a channel with a vee-bottomed trapezoidal cross-section. See Figure 3.5 for an explanation of the notation. Because of the change in geometry of the cross-section, one equation will be developed for critical depth, $d_c \leqq d$, and another for $d_c \geqq d$. For the triangular section, $d_c \leqq d$ and

$$a = \frac{Td_c}{2}$$

$$\frac{Q^2}{g} = \frac{T^3d_c^3}{8T} = \frac{T^2d_c^3}{8}$$

where

$$T = 2d_cz_1$$

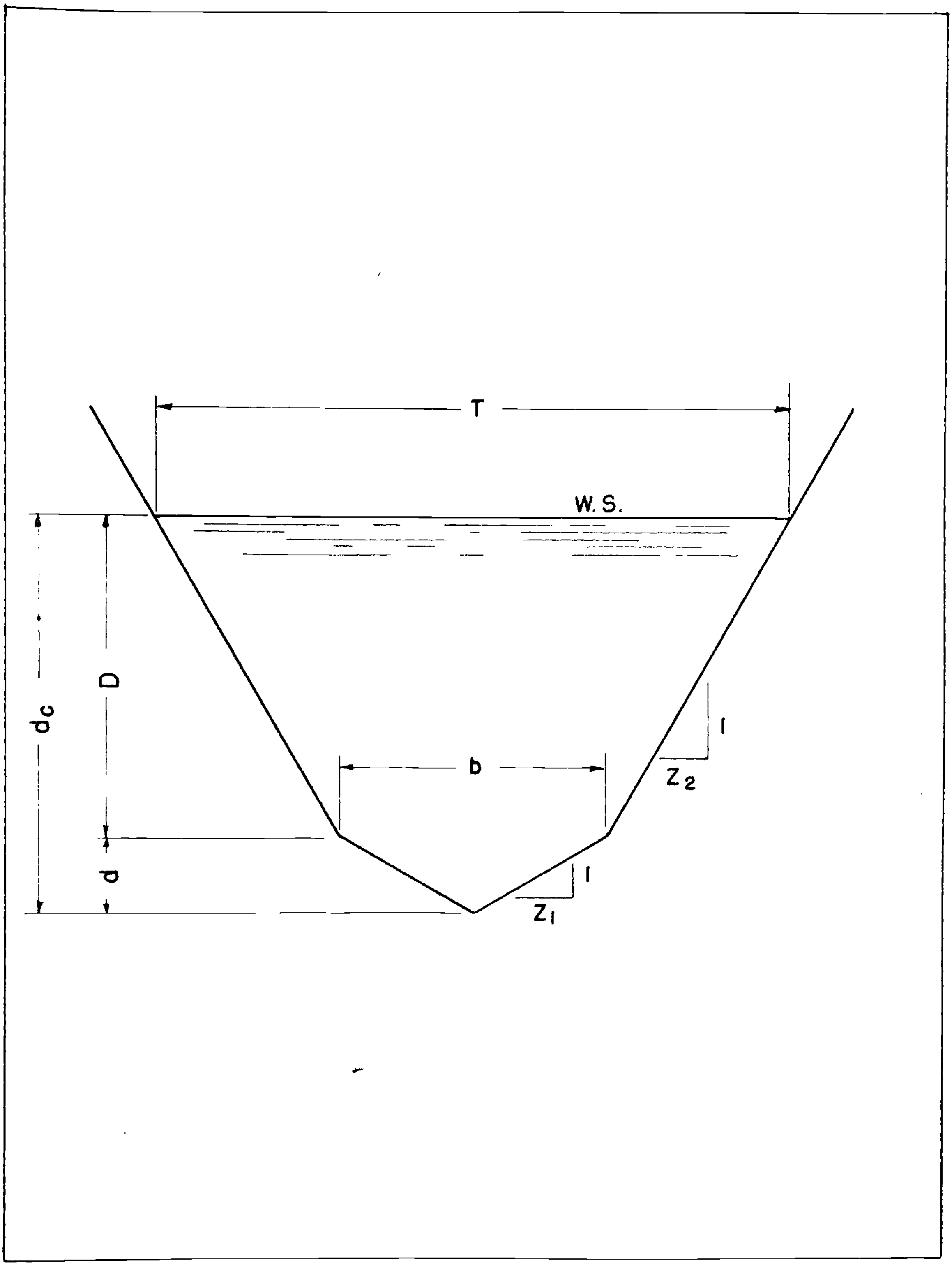

FIGURE 3.5

VEE- BOTTOMED, TRAPEZOIDAL OPEN CHANNEL

$$\frac{Q^2}{g} = \frac{d_c^5 z_1^2}{2}$$

$$Q = \sqrt{\frac{g}{2}}\, d_c^{5/2} z_1 = 4.01 z_1 d_c^{5/2} \qquad (3.4)$$

For $d_c \geq d$, then

$$a = \frac{bd}{2} + D\left(\frac{b + T}{2}\right)$$

$$d = \frac{b}{2z_1} \quad \text{and} \quad T = b + 2Dz_2$$

$$a = \frac{b^2}{4z_1} + D(b + Dz_2) = \frac{b^2}{4z_1} + Db + D^2 z_2$$

substituting in Equation (3.3)

$$\frac{Q^2}{g} = \frac{\left(\frac{b^2}{4z_1} + Db + D^2 z_2\right)^3}{b + 2Dz_2}$$

$$Q = \left[\frac{g\left(\frac{b^2}{4z_1} + Db + D^2 z_2\right)^3}{b + 2Dz_2}\right]^{1/2} \qquad (3.5)$$

where

$$D = d_c - d$$

Equation (3.5) is correct when stream lines are parallel.

When water is accelerated from tranquil flow to super-critical flow in any uniform section of channel, the exact location of critical depth changes with discharge. Experiments with sloping broad-crested weirs[11] have shown that with certain slopes the location of critical depth

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[11]J. G. Woodburn, "Tests of Broad-Crested Weirs," Transactions of the American Society of Civil Engineers, Vol. 96 (1932), pp. 387-416.

changes very slightly. Woodburn used a twelve-foot crest with a variable slope. He found that at a slope of 0.026 the location of critical depth moved about a foot when discharge varied from 2.12 to 8.86 cfs. Alex R. Webb, in the appendix to Mr. Woodburn's paper, described experiments on broad-crested weirs with a sloping upstream apron. He found that with an upstream apron slope of 0.0248 and a crest slope of 0.0392, the locus of critical depth was a vertical line over the apex.

A flume with a straight sloping section preceded by a converging entrance, is hydraulically similar to a sloping broad-crested weir with a sloping upstream apron. The function of both the apron and the converging section is to increase the velocity of the water before it enters the throat. It seems probable that a certain relationship between the geometry of the entrance and the throat of a flume would cause the locus of critical depth to occur in a vertical line.

3.5 Design of Flume Number 1

It was decided to measure the discharge of Davis-Monthan Wash, so that the hydrograph of Main Wash could be computed by subtracting the hydrograph of Davis-Monthan Wash from the Atterbury Reservoir inflow hydrograph. The Davis-Monthan Wash was in a process of degradation near Atterbury Reservoir as a result of the additional water diverted to it by the dike. For this reason, there were no uniform channel reaches. The channel reach chosen for Flume Number 1 is shown in Figure 3.6. The bottom of the wash at this location is composed of partially cemented clay and caliche and is very resistant to erosion.

The maximum discharge was estimated to be about 300 cfs. This figure was computed by the slope-area method, using high water marks to determine slope and depth. Because of the uncertainties inherent in this method, the flume was designed for a flow of 400 cfs.

The side slope, z_2, of $1\frac{1}{2}$:1 was selected because flatter slopes cause the formation of diagonal waves, and steeper slopes would require face forms for the placement of concrete. The bottom width, b, of 5 feet and the slope, z_1, of 5:1 were selected to conform to the original channel shape and to provide adequate capacity with a reasonable depth. The design of the inlet was arbitrary. The bottom slope of 3 percent was based on preliminary information received from the Agricultural Research Service, which was conducting model tests on a critical depth flume at the Outdoor Hydraulics Laboratory at Stillwater, Oklahoma. This slope insures that the flow will pass through critical depth in the throat.

A preliminary design of the flume was completed, and a sheet metal model was constructed to a scale of 1:12. The

FIGURE 3.6

SITE OF FLUME NUMBER 1

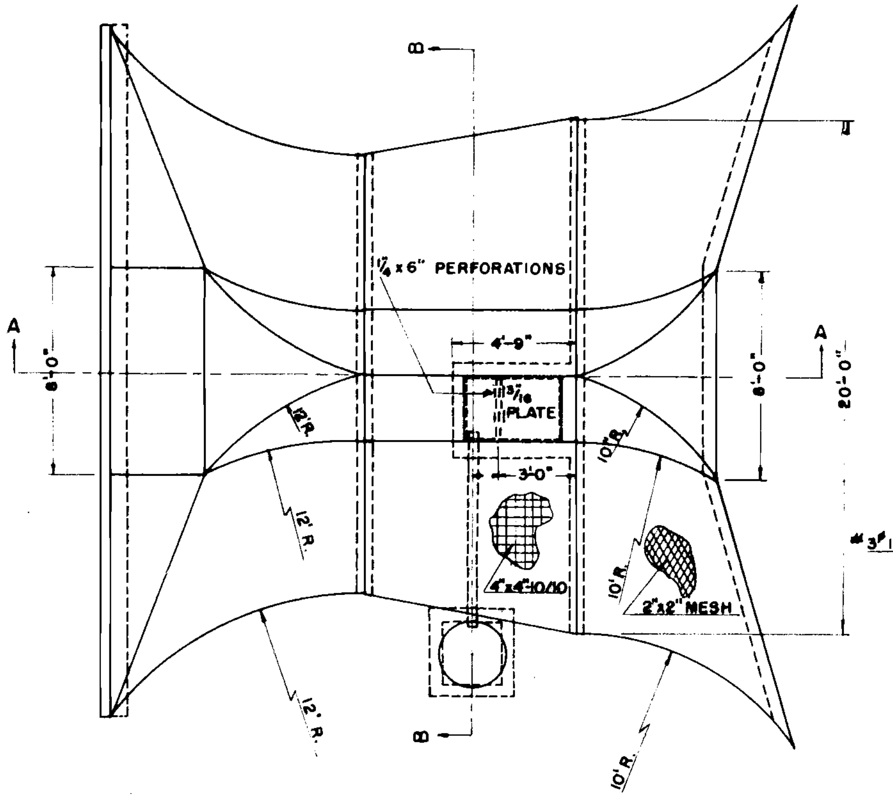
model was placed in an earth ditch and was tested with flows of up to 300 gpm. Although it was not possible to measure the flows accurately, flow profiles were obtained with piezometers and a point gage. It was observed that critical depth occurred in the upstream half of the throat. The downstream flow conditions were also observed. Minor modifications were made in the preliminary design, and the final hydraulic and structural design was completed (Figure 3.7). The inlet to the stilling well was designed so that the depth could be measured at any point from one foot to four feet below the entrance to the throat by using different cover plates.

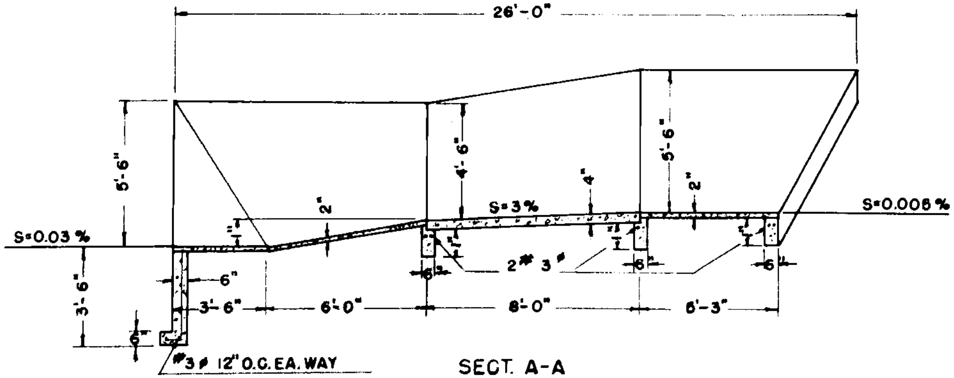
When the structure was completed, graduations were painted on the sides of the flume so that water surface profiles could be obtained visually or photographically. The inlet cover plate was not perforated until one flow had been observed. Using visual observations of the location of critical depth as a criterion, it was decided to measure the depth three feet downstream from the entrance to the throat.

3.6 Accuracy of Flume Number 1

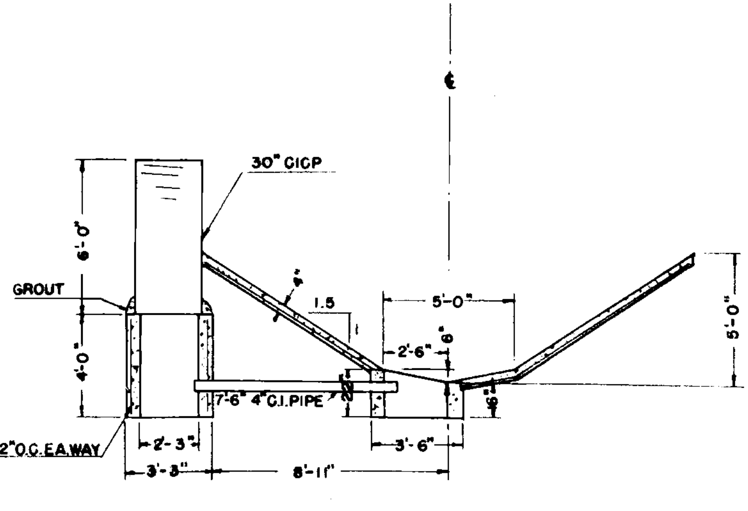
In designing Flume Number 1, the investigator realized that there was no criterion for assuring that critical depth would occur at, or very near, the selected measuring section for all rates of flow. It was known that, at least

FIG.—3.7





SECT. A-A



SECT. B-B

NOTE:

ALL REINFORCING SHALL HAVE MIN. CLEAR COVER OF 1" IN GROUND BEAMS AND 3/4 IN SLABS.

ENTRANCE AND EXIT SIDE SLOPES ARE 2: 1.

FIGURE 3.7
FLUME NUMBER 1.

CRITICAL DEPTH FLUME ON
ATTERBURY DRAINAGE AREA NEAR
TUCSON, ARIZONA
SCALE 1"=8'

AGRICULTURAL ENGINEERING DEPARTMENT
UNIVERSITY OF ARIZONA

9-23-58

D.A.W.

for some range of flow, the depth-discharge relationship would follow the theoretical form of equation (3.5).

Five measurements of discharge were made with a current meter to check the theoretical relationship of equation (3.5). The results are shown in Figure 3.8. The maximum error of these measurements is -3.20 cfs., the average error is +0.11 cfs., and the standard deviation of the errors is 1.78 cfs. Although these results appear to be promising, several more current meter measurements must be made to verify the theoretical rating.

Water surface profiles at several depths are shown in Figure 3.9. The photograph, Figure 3.10, shows flow conditions with a discharge of approximately 43 cfs. Critical depth theoretically occurs at the point of inflection in the water surface. In the photograph and in Figure 3.9, the point of inflection appears to be near the entrance to the throat, upstream from the measuring section. One explanation of this apparent contradiction of the current meter results, is that the water surface profiles and the photograph show the water surface at the wall of the flume. The transverse water surface is concave due to convergence, so that the "average" point of inflection occurs downstream from its apparent location. Another possibility is that the pressure at the measuring section is greater than hydrostatic because the streamlines are concave upward. If this

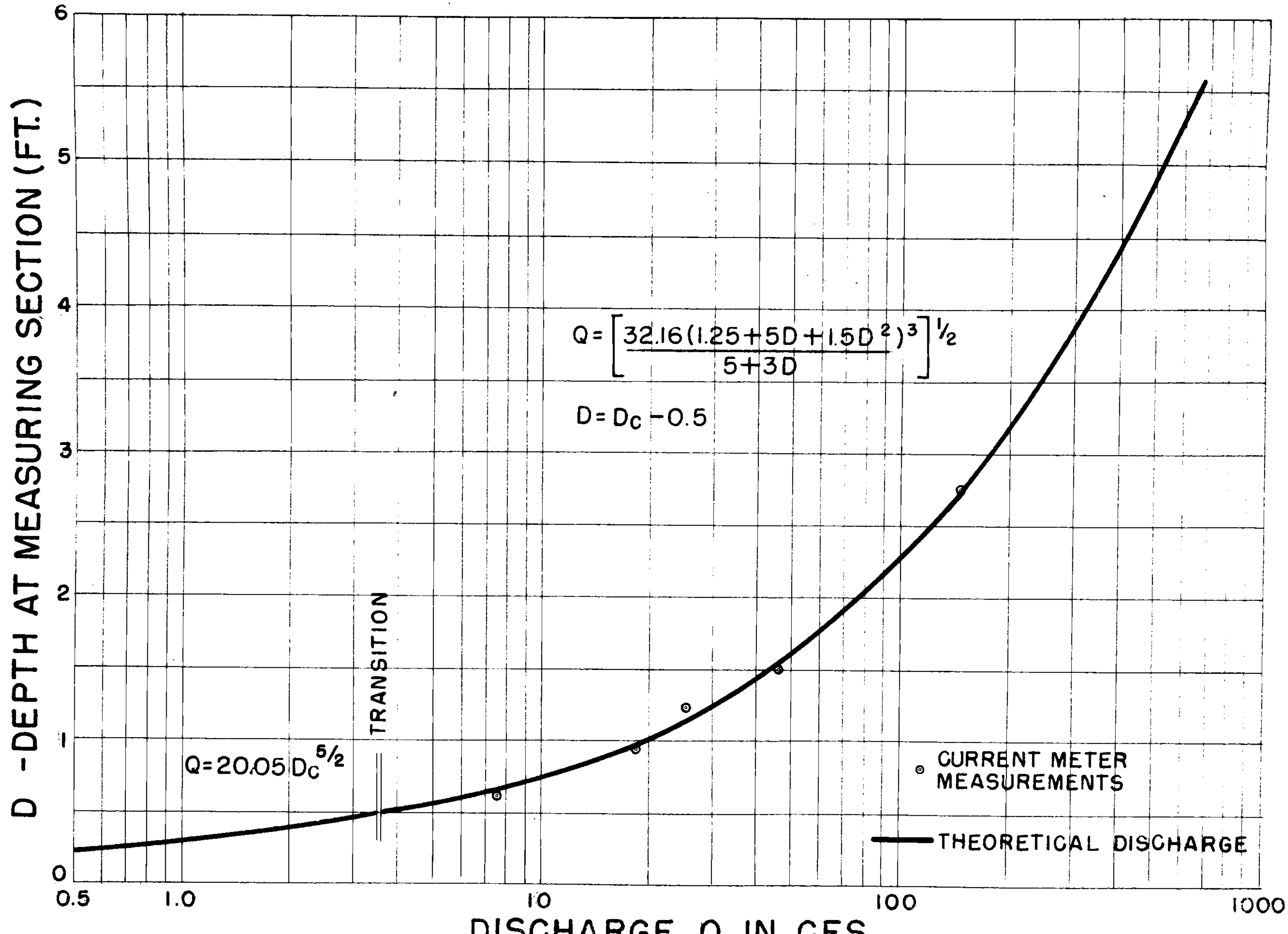


FIG.–3.8 CURRENT METER VERIFICATION OF THEORETICAL DISCHARGE CURVE FOR FLUME NO. I.

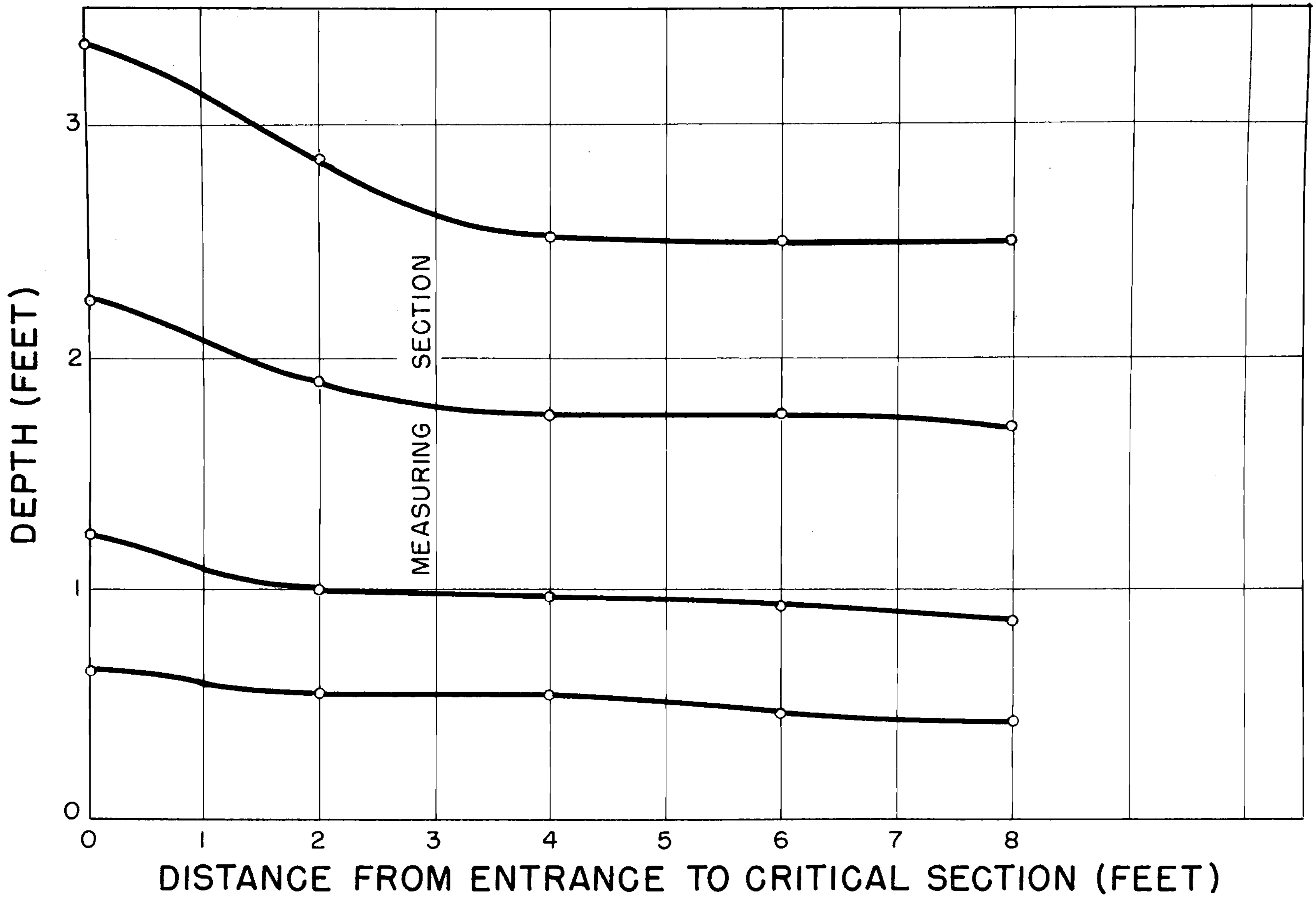


FIGURE 3.9—WATER SURFACE PROFILES—FLUME NUMBER I.

FIGURE 3.10

FLOW CONDITIONS--FLUME NUMBER 1

were true, the flow at the measuring section would be super critical; but the water surface in the stilling well would be higher than that in the measuring section, so that the flume would tend to be self-correcting.

The flume was found to be structurally sound, and downstream erosion has not become a problem. If the flume were constructed in less erosion-resistant material, it would be necessary to build a sill at the lower end to cause a hydraulic jump and lower downstream velocities. The stilling well inlet has remained clear during flows, and the sump has been cleaned out only once a year.

Chapter 4

PRECIPITATION

4.1 Annual Rainfall

The variability of average annual point rainfall in semiarid regions was mentioned in Section 2.2. Rainfall records from the Atterbury Reservoir Watershed illustrate this variability, and also indicate that a single point rainfall record is a poor measure of average annual areal rainfall.

Isohyetal maps of annual rainfall on the watershed for 1956 and 1957 have been prepared (Figures 4.1 and 4.2). The basis for the minimum and maximum isohyets for the year 1956 was especially interesting. In one instance, the total of 4.77 inches measured at Rain Gage 2 represents 0.39 inches less than the minimum annual rainfall reported by the University of Arizona station during the period 1891 to 1957. In the other case, 2.30 inches of the total of 9.57 inches measured at Rain Gage 19, fell during one storm on July 29. The latter record illustrates the significant effect of individual storms on the isohyetal configuration during dry years.

4.2 Monthly Rainfall

The monthly distribution of average rainfall on the

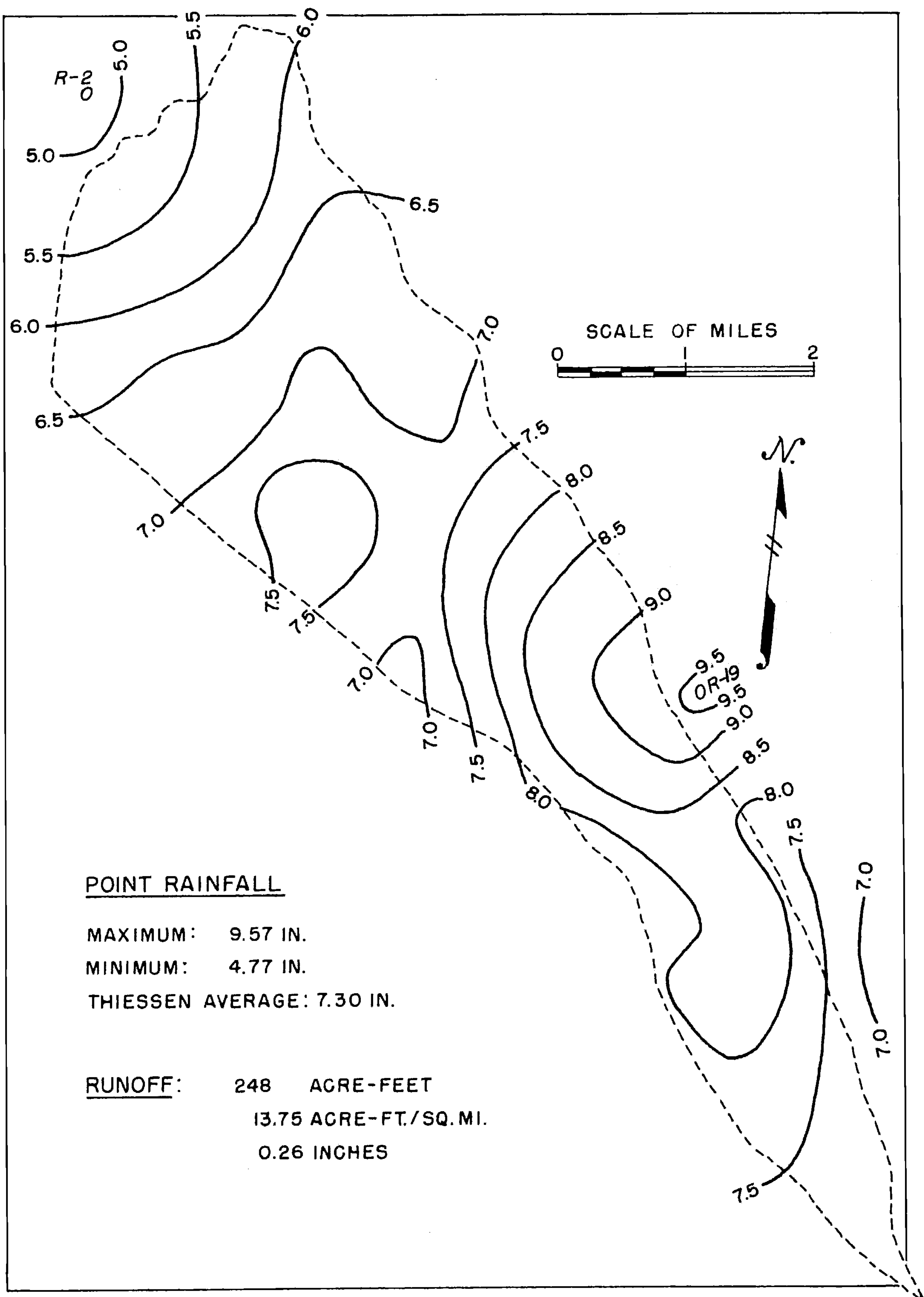


FIGURE 4.1-ISOHYETAL MAP OF TOTAL RAINFALL-1956
ATTERBURY RESERVOIR WATERSHED.

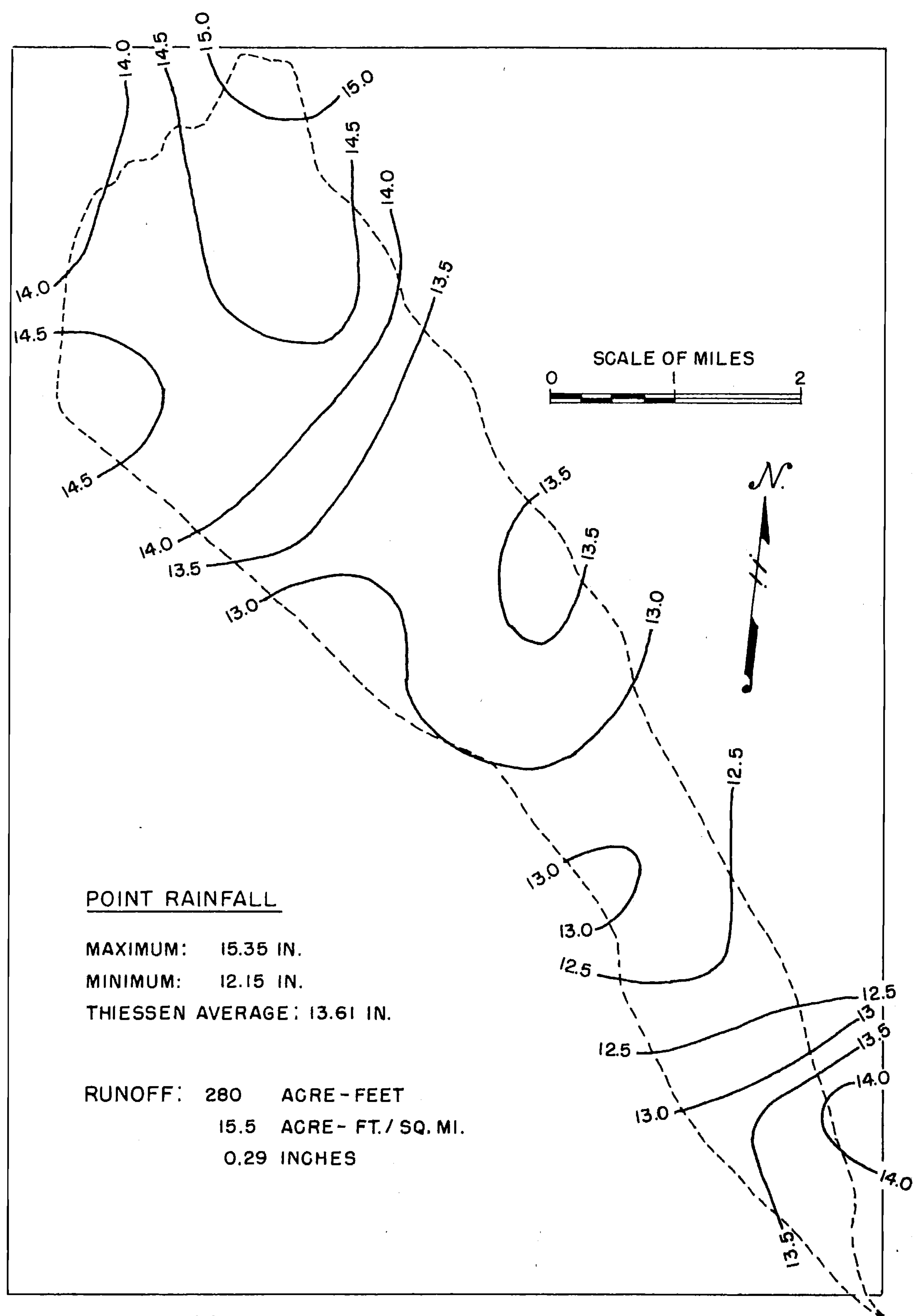


FIGURE 4.2- ISOHYETAL MAP OF TOTAL RAINFALL-1957
ATTERBURY RESERVOIR WATERSHED.

Atterbury Reservoir Watershed, and the departure of this average from the mean monthly rainfall at the University of Arizona for 1956 and 1957, is shown in Figures 4.3 and 4.4.

Monthly rainfall was below average for ten months in 1956 and for seven months in 1957. In both years rainfall was also below average for two months of the three-month period of summer rainfall.

4.3 Characteristics of Individual Storms

The areal depth patterns of precipitation from winter or frontal storms are irregular, and are not as predictable as the summer or thunderstorm patterns. The intensity of winter storms varies throughout the storm period, with occasional short periods of heavy rainfall. Although the winter storms may have long durations, they do not cause a significant amount of runoff from the valley slopes.

More important from the standpoint of water yield, are the summer storms which are the result of orographic and thermal circulation. This circulation causes the formation of thunderstorm cells, which are characterized by a strong updraft in the center of the cell and low velocity downdrafts around the periphera. The thunderstorm has been described as having three stages of development:

FIG. 4.3

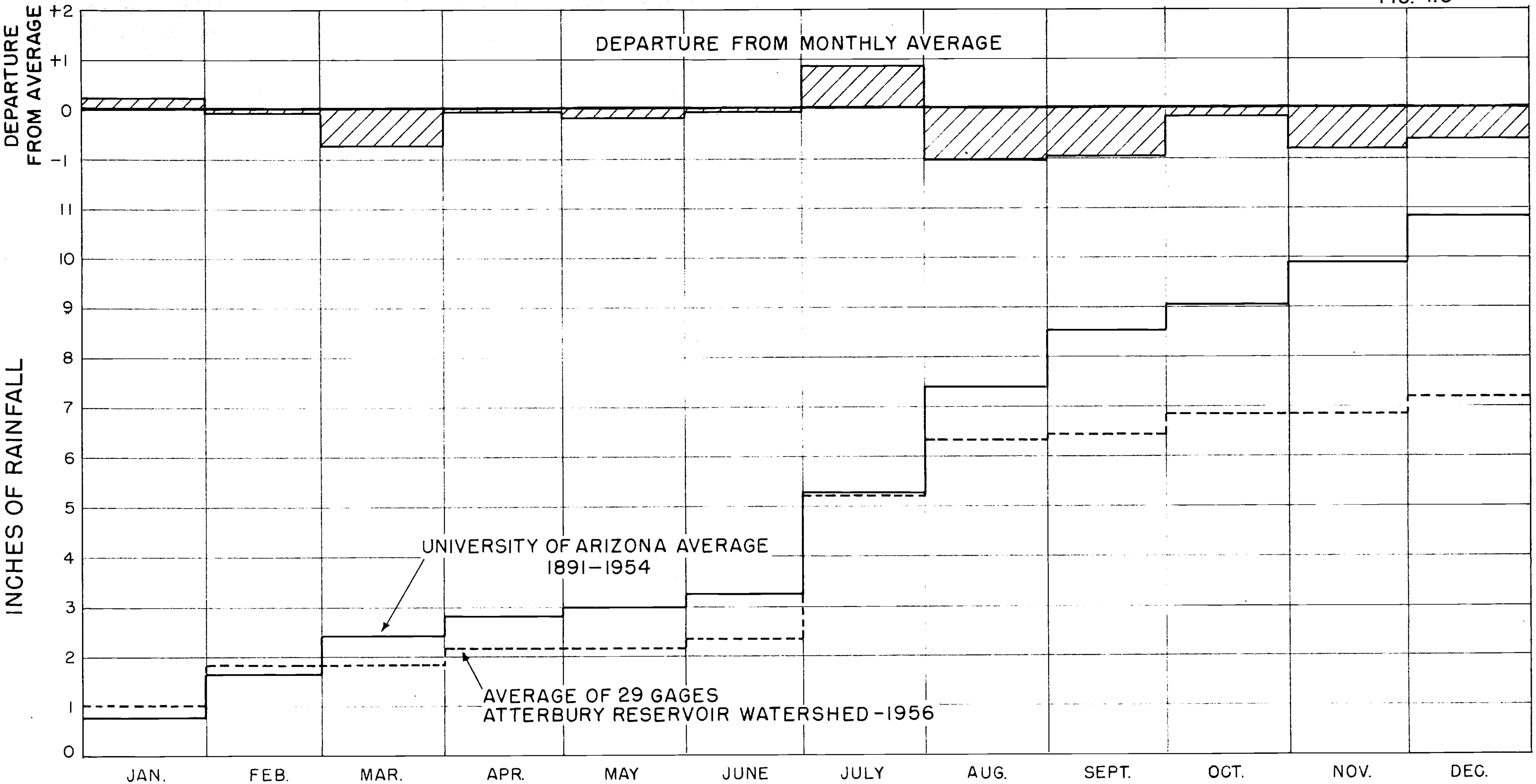


FIGURE 4.3- MONTHLY DISTRIBUTION OF AVERAGE RAINFALL-1956

FIG. 4.4

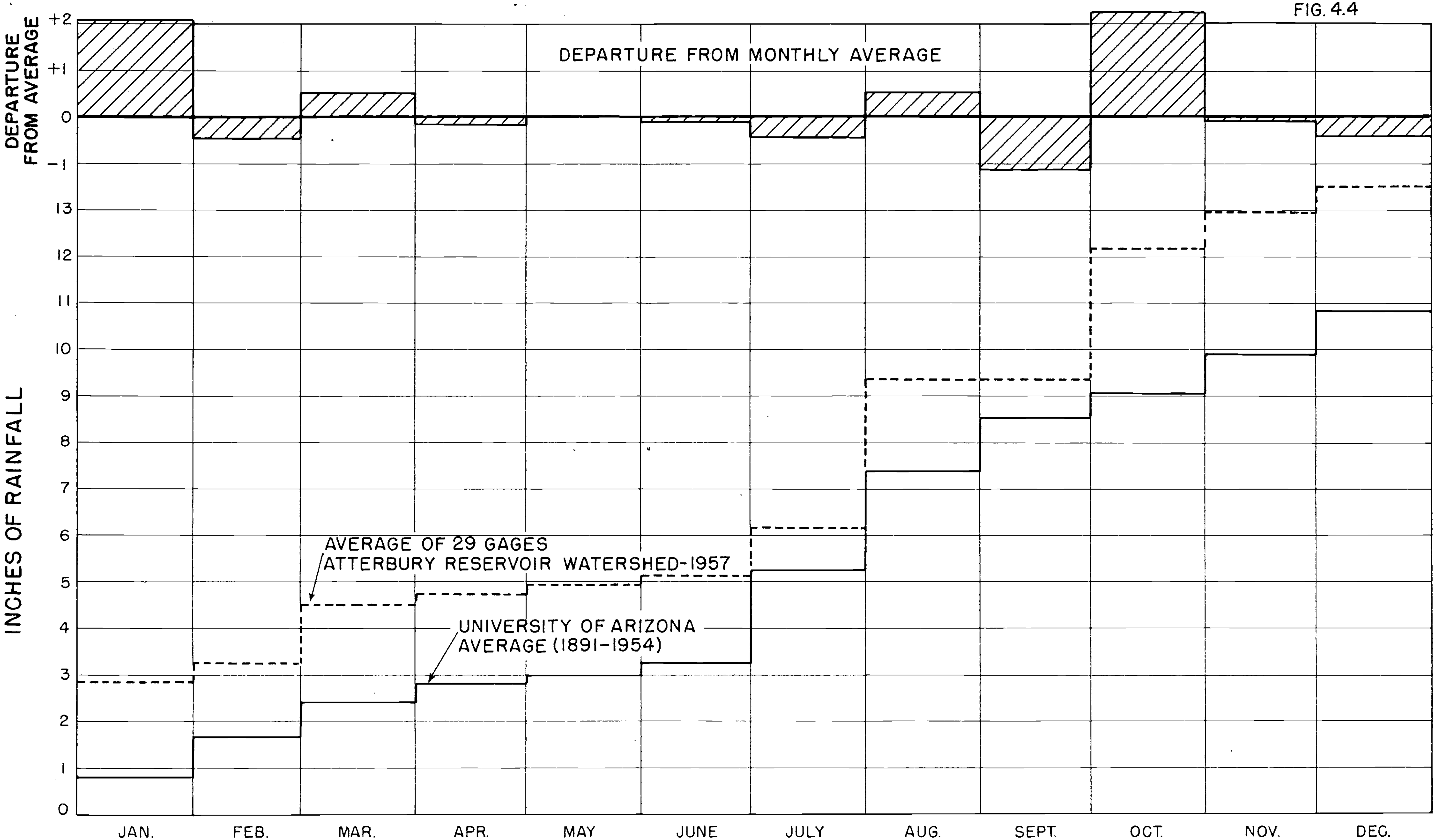


FIGURE 4.4- MONTHLY DISTRIBUTION OF AVERAGE RAINFALL-1957.

1. Cumulus stage--characterized by updrafts throughout the cell.
2. Mature stage--characterized by the presence of both updrafts and downdrafts at least in the lower half of the cell.
3. Dissipating stage--characterized by weak downdrafts prevailing throughout the cell.[12]

In its initial stage every thunderstorm is a cumulus cloud. It is not known what causes a particular cloud to develop into a storm, but the number that occur is related to the instability of the atmosphere.[13] Within each cumulus cloud is an updraft that extends throughout most of the cloud. As the cloud extends in height, free water is condensed, and is supported by the updraft until enough drops condense so that the updraft can no longer support them.

As the raindrops begin to fall, a downdraft develops in the center of the cell due to drag of the water particles. This downdraft adds to the intensity of the rain which varies from a low intensity at the beginning of precipitation, increasing rapidly to maximum rates in the mature stage of the storm, then dropping off slowly in the dissipating stage. An isohyetal map of thunderstorm rainfall shows a definite maximum, or storm center. The

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[12] The Thunderstorm, Report of the Thunderstorm Project, a joint project of four U.S. Government Agencies: Air Force, Navy, National Advisory Committee for Aeronautics, and Weather Bureau; United States Department of Commerce Weather Bureau (Washington: June, 1949), pp. 19-25.

[13] Ibid.

isohyets are elliptical with very steep rainfall gradients along the major axis. An isohyetal map of typical summer thunderstorm rainfall is shown in Appendix B.

4.4 Areal Rainfall Depth-Duration-Frequency Studies

A knowledge of detailed area-depth relations for small areas is often essential for engineering purposes. Few detailed data for small areas have been published. While the Illinois State Water Survey Division[14] has published data applicable to the Midwest, there have been no comparable studies for the Southwest. It has been suggested that an areal pattern for thunderstorm rainfall in the Southwest can be derived if the maximum rainfall at the storm center is known.[15]

An intensity-duration-frequency-area relationship has been developed from an analysis of the rainfall data from the Atterbury Reservoir Watershed. The analysis applies only to the valley slope areas near the watershed. No attempt has been made to generalize the study because the period of record was very short.

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[14]F. A. Huff and J. C. Neill, <u>Rainfall Relations on Small Areas in Illinois</u>, State of Illinois Department of Registration and Education, State Water Survey Division, Bulletin 44 (Urbana, Illinois: 1957).

[15]Luna B. Leopold, "Areal Extent of Intense Rainfalls, New Mexico and Arizona," <u>Transactions of the American Geophysical Union</u>, Part 2 (1942), pp. 558-563.

Five independent hypotheses concerning thunderstorm rainfall were made:

1. The occurrence of a storm center at any location is governed only by chance. The number of storm centers occurring on any area in a year may not be a constant, but there is an average number of occurrences and a certain frequency distribution about that average.
2. There is a continuous frequency distribution of storm center magnitude or depth.
3. The areal pattern of rainfall can be determined for any storm center magnitude.
4. There is a continuous statistical distribution of rainfall durations for any given storm center magnitude. For a constant magnitude, duration has a normal distribution about the mean.
5. Rainfall intensities for any storm duration may be adequately represented by distributing total rainfall with an average dimensionless mass rainfall curve.

4.5 Probability of Storm-Center Occurrence

Isohyetal maps were prepared for all runoff-producing storms. A storm center was considered to lie within the area if there was a definite rainfall maximum within the watershed boundary, or if the exterior angles between isohyetal lines and straight lines connecting the rain gages on the periphera of the network were equal to, or less than, ninety degrees. The area enclosed by straight lines connecting the rain gages on the periphera of the network was 18.46 square miles. Table II shows the monthly and annual distribution of storm center occurrences on the watershed for 1956, 1957, and 1958.

The probability of a storm center occurring on a square mile area in any year is then 6 ÷ 18.45 or 0.325. The recurrence interval, T, which is the reciprocal of probability, is then 3.08 years.

4.6 Frequency Distribution of Storm-Center Magnitudes

The magnitudes of the eighteen runoff-producing storm centers which occurred on the Atterbury Reservoir Watershed during the three years of record, were tabulated according to depth of rainfall. The probability of a given magnitude being equaled or exceeded was computed by the Kimball Method.[16]

$$P = \frac{m}{n + 1}$$

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[16] Bradford E. Kimball, "Assignment of Frequencies to a Completely Ordered Set of Sample Data," *Transactions of the American Geophysical Union*, 27 (Dec., 1956) p. 843.

TABLE II

MONTHLY AND YEARLY DISTRIBUTION
OF STORM CENTER OCCURRENCES

Month	Year 1956	1957	1958	Total	Average
January	0	0	0	0	0
February	0	0	2	2	0.67
March	0	0	0	0	0
April	0	0	0	0	0
May	0	0	0	0	0
June	0	0	1	1	0.33
July	1	2	2	5	1.67
August	1	4	1	6	2
September	0	0	0	0	0
October	0	4	0	4	1.33
November	0	0	0	0	0
December	0	0	0	0	0
Total	2	10	6	18	6

where:

m = order number

n = total number of items

A log-normal plot of the frequency distribution of storm magnitudes is shown in Figure 4.5. From this Figure it may be seen that the median value of storm center depth was 1.1 inches, and only ten percent of all storm centers had a magnitude equal to, or greater than, 2.1 inches. The data was also plotted on normal probability paper and Gumbel frequency paper, but the log-normal distribution gave the best fit.

4.7 Rainfall Patterns

Of the eighteen storms that occurred on the watershed, nine had three or more closed 0.10 inch isohyets and could be analyzed to determine a depth-area relationship. The area enclosed by each isohyet was measured with a planimeter. A regression line was computed for the relationship between the logarithms of the difference between the maximum depth at the storm center and isohyetal depth, and the area enclosed by the isohyet. The regression equation was

$$\log. Y = 1.57 \log. X + 1.08 \qquad (4.1)$$

where:

Y = isohyetal area in square miles

X = storm center depth minus the isohyetal depth in inches

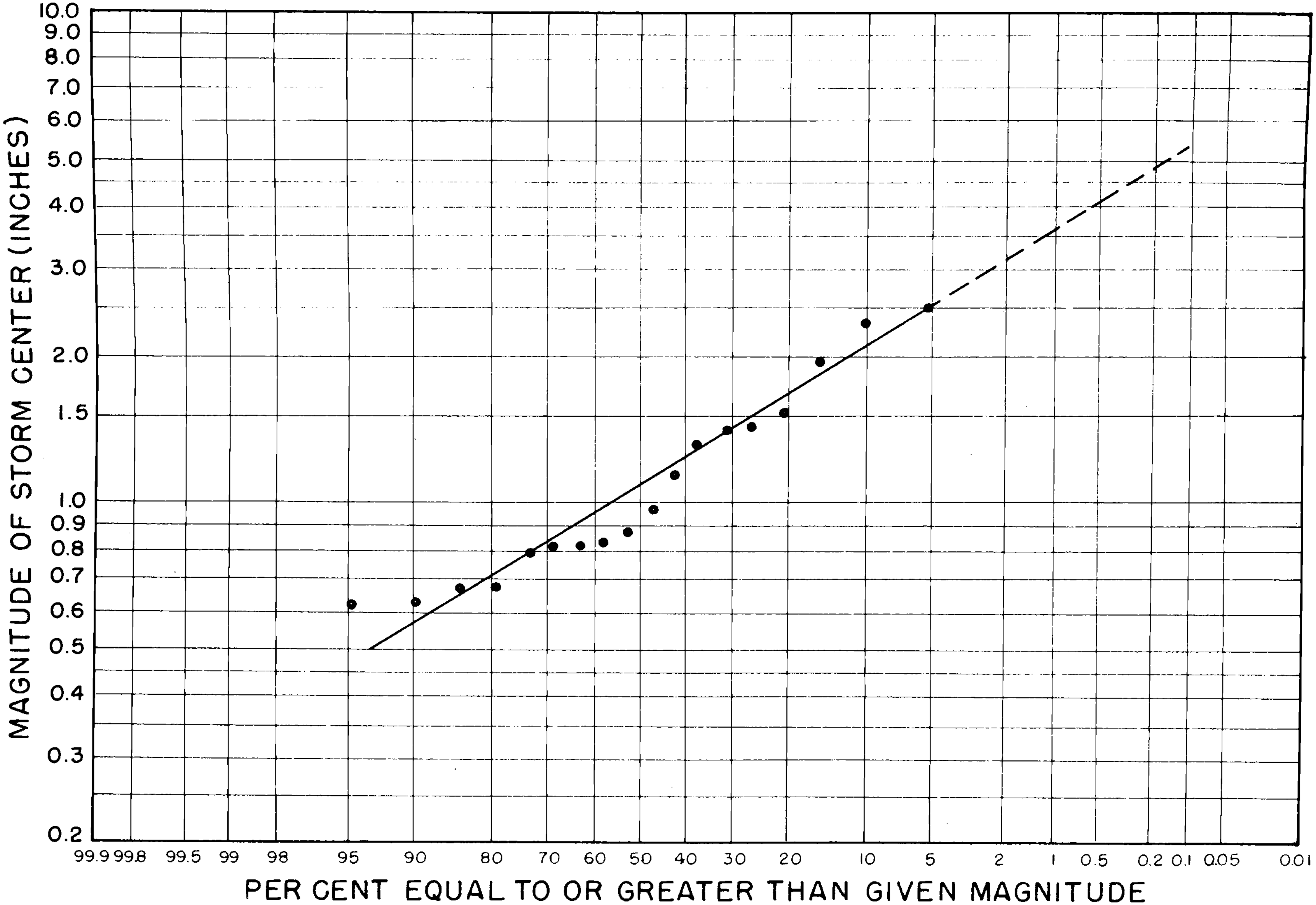

FIGURE 4.5-FREQUENCY OF MAGNITUDE OF STORM CENTERS

The correlation coefficient was 0.94, which by "Students" t test is significant at the 0.1 percent level.

The regression line is shown in Figure 4.6. Control curves are shown as dashed lines, a distance of one standard error of estimate from the curve. Approximately two-thirds of all points should fall between these control curves.

4.8 Rainfall Intensity and Duration

All of the runoff-producing storms which were recorded by an automatic gage were included in this analysis. Only one of the eighteen storms was centered over a recording rain gage. For the other storms, the recording-gage records were converted to storm center values by constructing dimensionless mass curves of rainfall at the recording gage, and assuming that the rainfall at the storm center had the same time distribution. (See Figures 4.7 and 4.8)

In attempting to correlate storm center magnitude with total duration, it was found that for most storms a long period of low intensity rainfall at the beginning or end of the storm accounted for much of the duration, but very little precipitation. The duration of ninety percent of the total storm depth was selected as the independent variable in a correlation analysis because it appeared to give the best fit. The duration of ninety percent of the

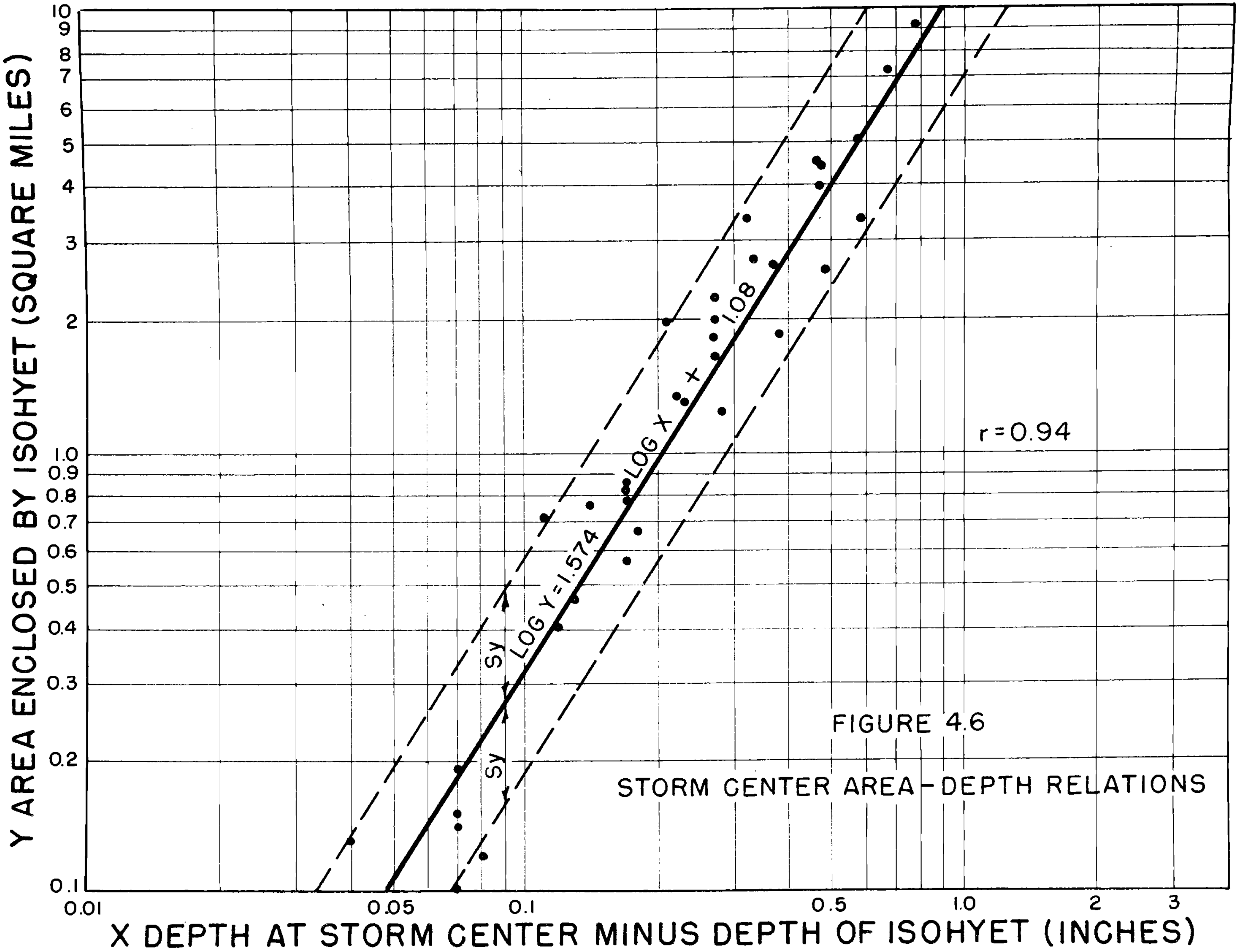

FIGURE 4.6

STORM CENTER AREA-DEPTH RELATIONS

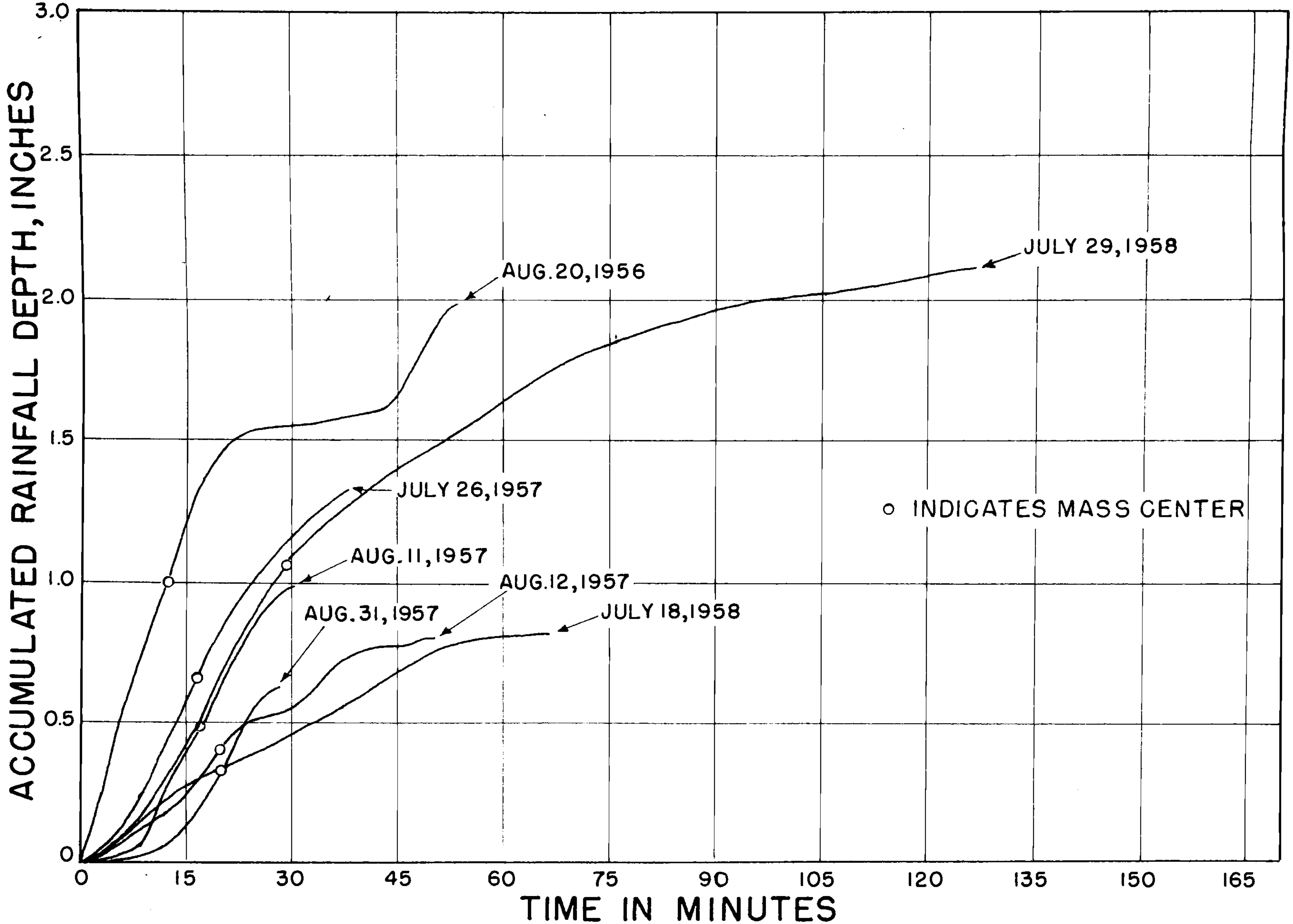

FIGURE 4.7-MASS CURVES OF STORM CENTER RAINFALL.

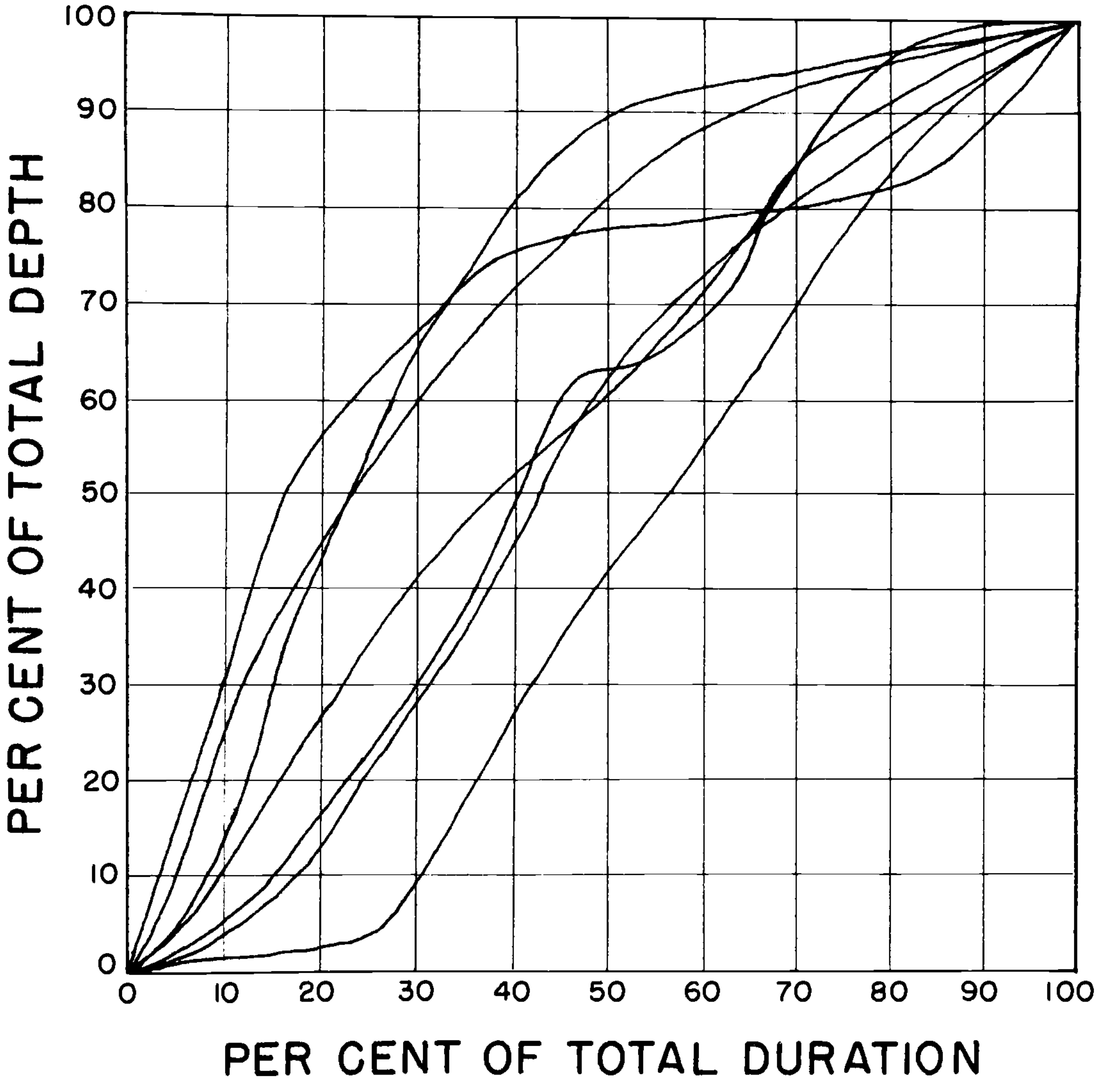

FIGURE 4.8-DIMENSIONLESS MASS RAINFALL CURVES.

depth was determined from the mass curves of storm center rainfall (Figure 4.7). The mass center was located, and the duration of forty-five percent of the total rainfall was measured from each side of the mass center. The ninety percent duration was then the sum of the forty-five percent durations.

The following straight line regression equation was computed by the method of least squares:

$$Y = 31.4X + 2.1 \tag{4.2}$$

where:

X = storm center magnitude in inches.

Y = duration of ninety percent of storm center magnitude in minutes.

The coefficient of correlation was 0.76, which according to "Students" t test was significant at the five percent level.

The regression equation is shown by the heavy, straight line in Figure 4.9. The assumption was made that the durations associated with any magnitude have a normal distribution about the mean duration. The standard error of estimate was 14.7 minutes. From this value and a table of areas under the normal curve,[17] control curves were constructed. These are shown as dashed lines in Figure 4.9.

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[17] M. J. Moroney, Facts From Figures (second edition, Baltimore: Penguin Books, Inc., 1953), p. 116.

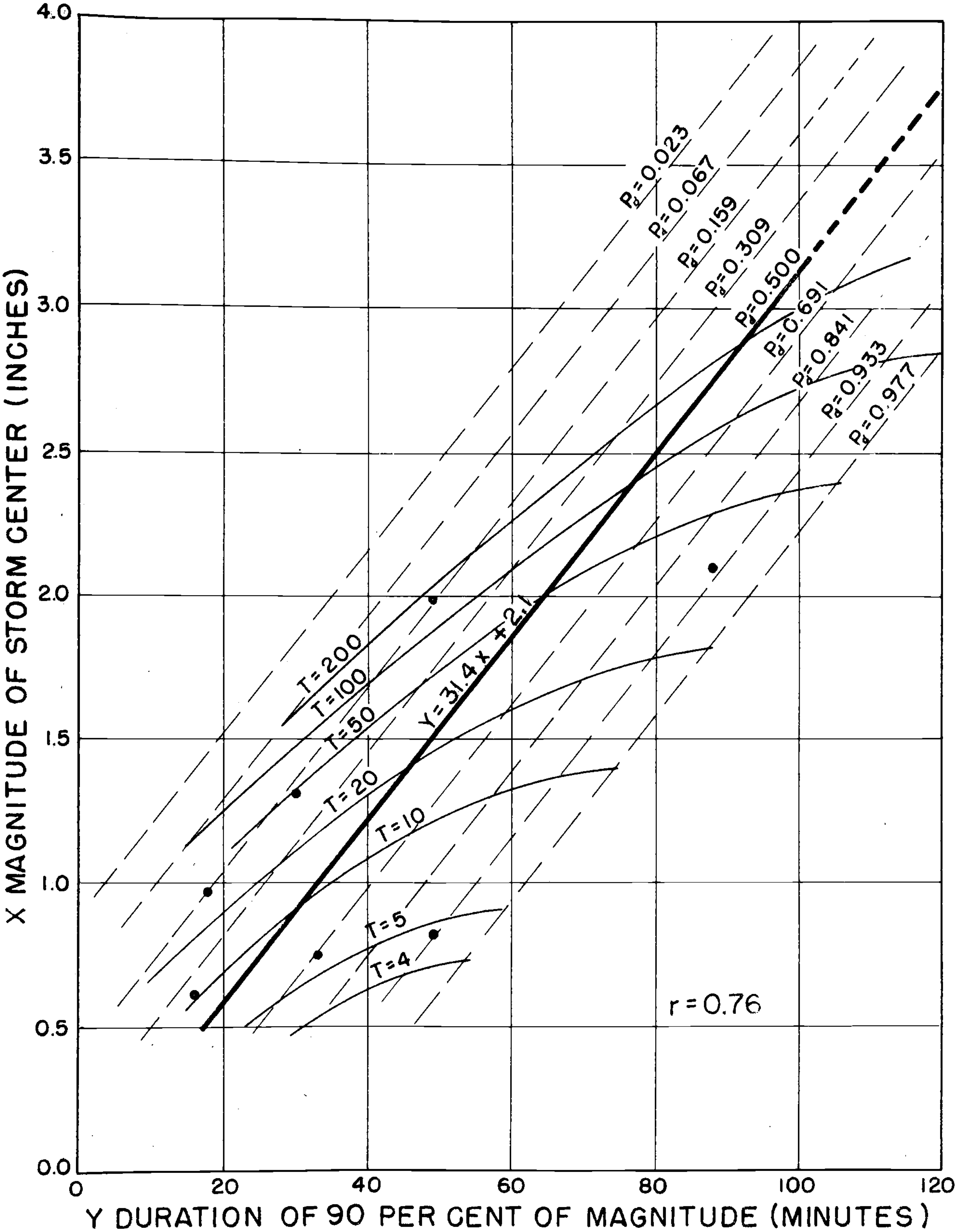

FIGURE 4.9 - STORM CENTER DEPTH-DURATION-FREQUENCY RELATIONS FOR AN AREA OF ONE SQUARE MILE.

The probability, P_d, associated with these lines is the probability of a duration being equal to, or less than, the given duration. Since the occurrence of a storm center on any area, the magnitude of the storm center, and the duration of the storm must be simultaneous events; the probability, P_{amd}, of a particular storm occurring on some area is the product of the probabilities that the storm will occur, that its magnitude will be larger than a given size, and that its duration will be smaller than a given time. Or

$$P_{amd} = P_a \, P_m \, P_d \tag{4.3}$$

where

P_{amd} = Probability of occurrence of a storm on area, a, of magnitude equal to, or greater than, m; and with duration equal to, or less than, d.

P_a = Probability of a storm center occurring on area, a, in one year.

P_m = Probability that the magnitude of a storm center is equal to, or greater than, m.

P_d = Probability that the duration of any storm is equal to, or less than, d.

The recurrence interval, T_{amd}, will then be:

$$T_{amd} = \frac{1}{P_{amd}} = \frac{1}{P_a \, P_m \, P_d} \tag{4.4}$$

Magnitude-duration curves for various recurrence intervals for an area of one square mile are shown in Figure 4.9. In order to compute short term intensities during the storm period, the storm center magnitude must be distributed according to the average dimensionless mass rainfall curve, Figure 4.10. This curve is an average of the mass curves for the seven storms studied.

4.9 Comparison of Areal Depth-Duration-Frequency Relations With Point Rainfall Records

The validity of the areal-depth frequency relation was tested by computing the probabilities that various point rainfall measurements would be equaled or exceeded, and comparing these probabilities with long term point gage records. For an M inch rain to be equaled at a particular rain gage, a storm with a center of M inches must be almost directly centered on the gage. If rainfall isohyets are assumed to be concentric circles, a storm of magnitude $M + d_1$ may occur anywhere within a circle of area a_1, and still cause rainfall of at least M inches at the gage; and a storm of magnitude $M + d_n$ may be anywhere within a circle of area a_n, and still cause rainfall of at least M inches at the gage. Each area has an associated probability of storm center occurrence, P_{a1}, $P_{a2} \ldots . P_{an}$

and

$$P_{an} = \frac{1}{T_1} a_n \tag{4.5}$$

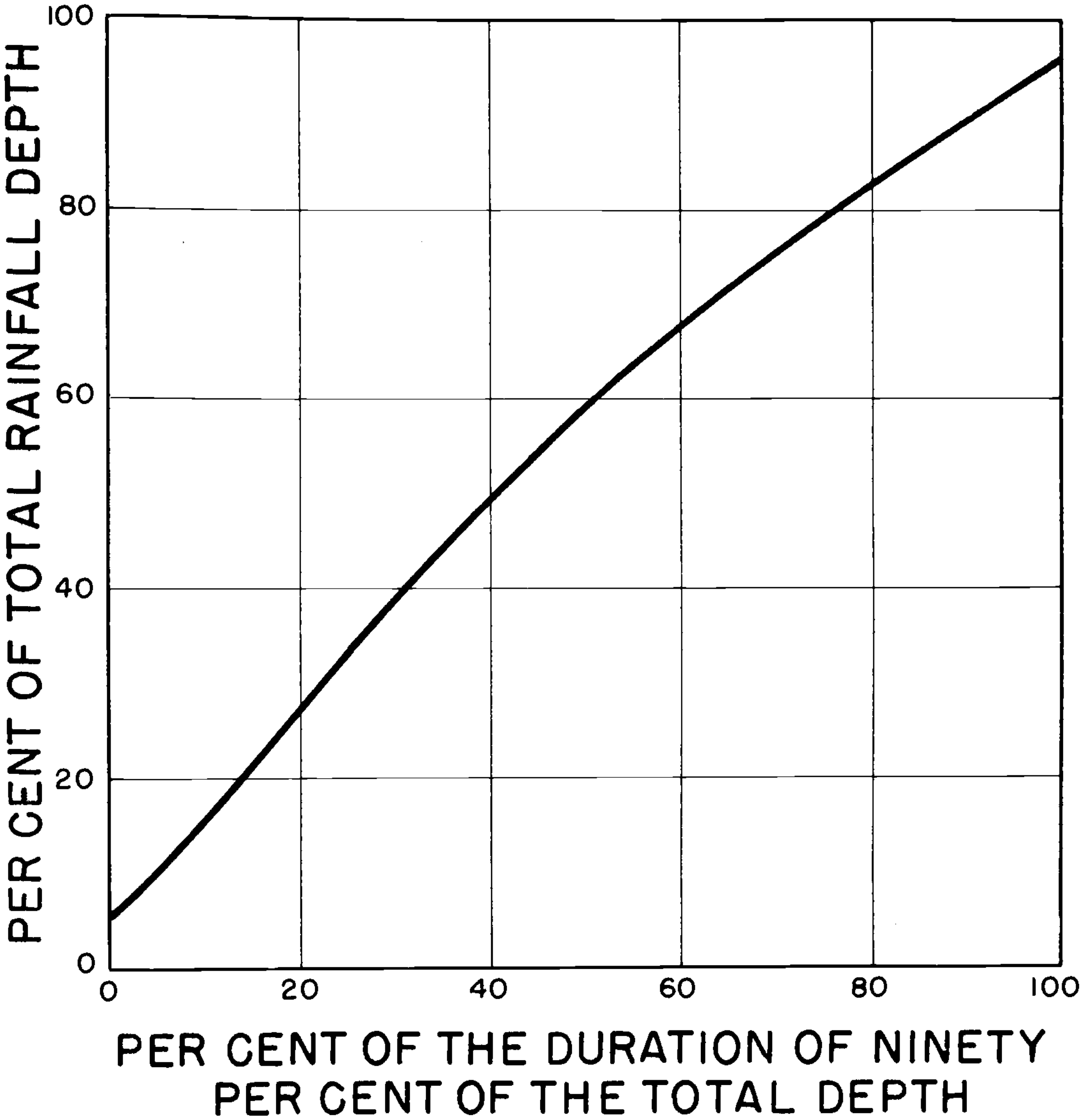

FIGURE 4.10 – AVERAGE DIMENSIONLESS MASS RAINFALL CURVE.

where T_1 is the storm center recurrence interval for a one square mile area, and a_n is the area in square miles within which a storm center of magnitude $M + d_n$ will cause a depth of at least M at the center. Each storm center depth has an associated probability of being equaled or exceeded, P_{m1}, P_{m2}.....P_{mn}. The probability, P_M, of a point rainfall measurement of M inches being equaled or exceeded is then given by the relationship

$$P_M = \frac{1}{T_1} \sum_{n=0}^{n=\infty} P_{mn} \Delta a_n \tag{4.6}$$

which may be expanded in a stepwise approximation by selecting values of $M + d_1$, $M + d_2$.....$M + d_n$ to give the following equation:

$$P_M = \frac{1}{T_1}\Big[a_1(P_{m1} - P_{m2}) + a_2(P_{m2} - P_{m3}) + \ldots$$
$$\ldots + a_n\big(P_{mn} - P_{m(n+1)}\big)\Big] \tag{4.7}$$

Table III shows the recurrence intervals computed from equation (4.7) and the measured recurrence intervals for 24-hour rainfall at the University of Arizona Station. One-half inch increments above the point depth were used in the approximation. Halving this increment had very little effect on the computed recurrence interval. A sample computation is shown in Appendix C.

TABLE III

COMPUTED AND OBSERVED RECURRENCE INTERVALS FOR VARIOUS POINT RAINFALL DEPTHS

Point depth equaled or exceeded (inches)	Computed recurrence interval (years)	Observed* 24-hour recurrence interval (years) 1910-1941
1.5	2.2	1.8
2.0	4.9	5.0
2.5	11.8	14.3
3.0	32	50

*H. C. Schwalen, Rainfall and Runoff in the Upper Santa Cruz River Drainage Basin, Technical Bulletin No. 95, University of Arizona Agricultural Experiment Station (Tucson: 1952), p. 438.

The 24-hour recurrence intervals would be expected to be shorter than the computed values, which are for individual storms. For 1.5 and 2-inch storms the agreement is very good, but the computed recurrence intervals for the 2.5 and 3-inch storms are obviously too low. The reason for these discrepancies may be errors due to extrapolation of the storm center-frequency curve, or to sampling error in computing the storm center recurrence interval, or both. There also is the possibility that for rare storms, the depth-area relationship is different from the relationship for the storms which were analyzed.

The depth-duration curves of Figure 4.9 were plotted on semi-logarithmic paper to compare them with point rainfall depth duration curves for the Tucson area[18] (Figure 4.11). The curves from Figure 4.9 were converted to average rate of rainfall during the duration of ninety percent of the total depth. Since they represent average intensities, they should be lower than the point rainfall intensities for short durations. However, the storm-center intensities should approach point intensities for long durations where maximum point intensities are caused by the occurence of a storm center near the gage. Although the depth-duration data is consistent with measured events on the Atterbury Reservoir Watershed, it appears that the predicted intensities are too low.

4.10 Conclusions

The following conclusions can be made concerning the five hypotheses of section 4.4:

1. The occurrence of a storm center over any location on the Atterbury Reservoir Watershed appears to be governed by chance.
2. The statistical distribution of storm center depths is closely defined by a log-normal distribution.

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[18]Computed by Charles Roarke of the Soil Conservation Service Watershed Planning Unit from 19 yrs. of record.

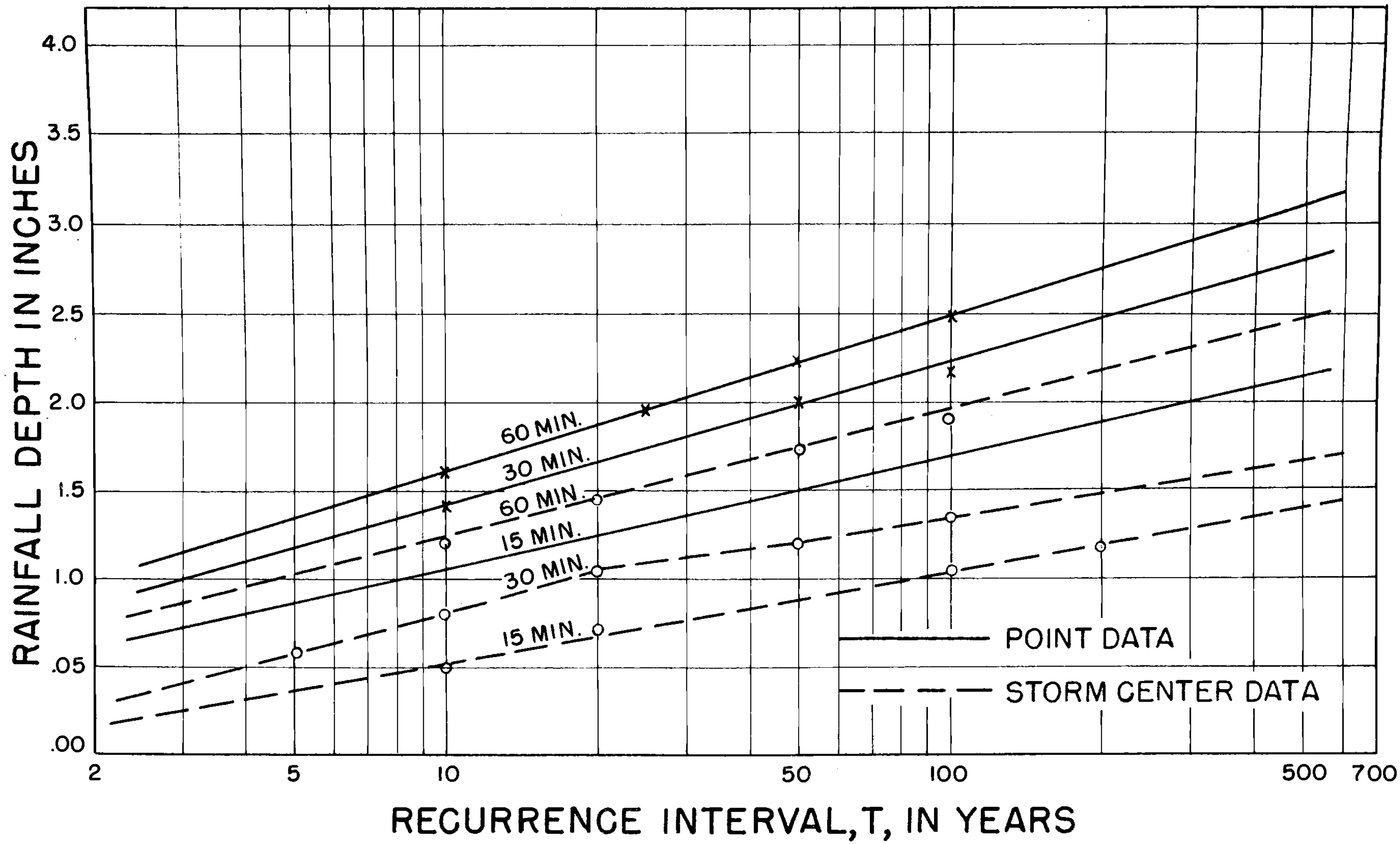

FIGURE 4.11—COMPARISON OF STORM CENTER AND POINT DEPTH-DURATION RELATIONS.

3. Areal storm patterns for the study area can be approximated by equation (4.1).
4. No conclusions can be made regarding the statistical distribution of rainfall duration. There is a fair linear correlation between duration of ninety percent of storm center depth and storm center magnitude.
5. The average dimensionless mass curve does not adequately represent all storm intensity patterns.

Several more years of rainfall data must be analyzed to reduce the errors, and to obviate extrapolation of data.

Chapter 5

RUNOFF

5.1 Annual Volume of Runoff

The annual volume of runoff measured in the three reservoirs for 1956, 1957, and 1958 is shown in Table IV.

TABLE IV

ANNUAL VOLUME OF RUNOFF
ATTERBURY RESERVOIR WATERSHED

	Volume in Acre Feet			
Year	Tank No. 2	Tank No. 3	Atterbury Reservoir	Total
1956	31	7.0	210	248
1957	28	7.3	244	279
1958	24	13.8	242	280
Average	28	9.0	232	269

This runoff represents an average water yield of fifteen acre feet per square mile, or 0.28 inches. Water yield per unit area on a subwatershed basis could not be determined because an unknown amount of water ran over the spillways of Tank Number 2 and Tank Number 3.

The measured runoff showed little variation, although the annual average rainfall varied from 7.30 inches in 1956 to 13.61 inches in 1957. This would indicate that there is a poor correlation between annual runoff and annual rainfall. When more data have been accumulated, it may be found that a better correlation exists between annual runoff and summer rainfall.

5.2 Quality of Runoff

Considerations of the chemical quality and silt content of runoff are important if the water is to be used for artificial recharge. A high silt content is undesirable because the silt lowers percolation rates in spreading basins, and seals the aquifer if the water is recharged through wells. If the runoff has a high sodium content and a high sodium-calcium ratio, the crumb structure of the soil in a spreading basin may be destroyed and percolation rates will decrease.

Water samples were taken from the three reservoirs in 1956 and 1957. The samples were taken from shore and were usually taken three or four days after a period of runoff. The chemical analysis and the sediment content of these samples are shown in Table V.

The total salt content of the runoff is very low and the sodium-calcium ratio is favorable. The silt content of the water in the reservoirs is, of course, variable with time; but for example, the average of 0.026 percent silt in Tank No. 2 represented approximately seven cubic feet of dry silt per acre foot of water.

TABLE V

CHEMICAL ANALYSIS AND SEDIMENT CONTENT OF RUNOFF

Analysis	Tank* No. 2	Tank† No. 3	Atterbury** Reservoir
Carbonate, ppm	0	0	0
Bicarbonate, ppm	105	112	110
Chloride, ppm	11	10	8
Sulphate, ppm	Tr.	Tr.	Tr.
Calcium, ppm	23	29	22
Magnesium, ppm	10	6	9
Sodium, ppm	4	6	7
Total Salts, ppm	153	166	155
Na-Ca ratio	0.17	0.21	0.35
Hardness, gr/gal	5.9	5.7	5.2
Silt, %	0.026	0.032	no determination of silt

*Average of four samples
† Average of three samples
**Average of two samples

5.3 Hydrograph Analysis

The characteristics of the hydrographs of runoff from small, semiarid areas are important in the design of storage reservoirs and storm drainage facilities.

The hydrographs recorded by Flume Number 1 were found to be the only ones accurate enough for analysis. In the three years of record, eight storm centers occurred on Subwatershed I, and there were also eight significant runoff events. It was found that not enough hydrographs were available to prepare a reliable standard dimensionless hydrograph, so no further analysis was attempted. The hydrographs for the largest peak flow for each of the years of record are shown in Appendix D for illustrative purposes.

5.4 Conclusions

The following conclusions appear to be justified by the data:

1. Runoff from the Atterbury Reservoir Watershed is not as variable as the annual rainfall. For this reason, the three year average water yield of 0.28 inches is probably near the true average.
2. Surface runoff water from the watershed has excellent chemical characteristics. It is not known if the silt content is high enough to

warrant treatment if the water is to be recharged through wells.

3. A longer period of record is necessary before hydrograph analysis is feasible, or a rainfall-runoff relationship can be developed.

Chapter 6

DISPOSITION OF RUNOFF

6.1 Introduction

Runoff is a residual of rainfall after certain losses have occurred. These losses are, in order of occurrence: evaporation from rainfall, interception, surface detention, infiltration, and channel losses. For the Atterbury Reservoir Watershed, reservoir losses may be added to this list.

6.2 Rainfall Evaporation and Interception

Evaporation from rainfall may be a large factor in the Southwest, where rain often falls through relatively dry air. No measurements were made of this loss, however, since this project was primarily concerned with rainfall as measured near the surface of the earth.

Interception of rainfall by vegetation is an important loss where vegetation is dense. No attempts were made to measure interception loss on the Atterbury Reservoir Watershed because the investigator did not believe it would be significant, since the vegetation was extremely sparse.

6.3 Infiltration

Surface detention and infiltration have been considered together since they are difficult to measure separately. In humid regions, part of the rainfall which infiltrates moves downward past the plant root zone and eventually becomes ground-water storage. Soil moisture samples taken after prolonged rainy periods on the Atterbury Reservoir Watershed showed that during the period of study, soil moisture did not penetrate beyond the plant root zone. All infiltration, therefore, has been lost by transpiration or evaporation from the soil surface. Since the potential evapo-transpiration is high in this region as compared with the amount of moisture in the soil, the type and density of vegetation has very little effect on water losses.

6.4 Channel Losses

No direct measurements of channel losses have been made on the Atterbury Reservoir Watershed. However, it is the investigator's opinion that a measurement of these losses is an essential part of a study to determine water yield. In this region, storms may cover small portions of a watershed, and the resulting runoff frequently traverses channels which have been dry for several months. It is common for stream flows to be completely absorbed by dry channel beds.

Some of the water which is absorbed by the channel alluvium may escape the root zone and percolate downward to the ground-water table. There have been no authoritative studies which show what proportion of the channel losses eventually becomes ground water.

Channel losses also affect the shape of stream-flow hydrographs. A dry channel causes the rising limb of the hydrograph to be almost vertical, and reduces the peak rate of flow.

6.5 Reservoir Losses

The losses from reservoir storage are by evaporation and percolation. The evaporation losses have been computed for the reservoirs by applying a monthly coefficient[19] to the evaporation measured at the University of Arizona standard U.S. Weather Bureau land pan. For 1956, the evaporation volume in each reservoir was computed from daily evaporation data and the reservoir surface area at noon on that day. For 1957, the average evaporation for a five-day period and the average reservoir surface area for that same period were used to compute the evaporation volume.

Figures 6.1 and 6.2 are mass curves of runoff into, and losses from Atterbury Reservoir and Tank Number 2 for 1956 and 1957. The percolation loss was computed as the difference between storage and mass evaporation at any date.

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[19]Arthur A. Young, _Evaporation Investigations in Southern California_, U.S. Department of Agriculture and Division of Water Resources, Department of Public Works, State of California (Pomona, California: April 1945), p. 72.

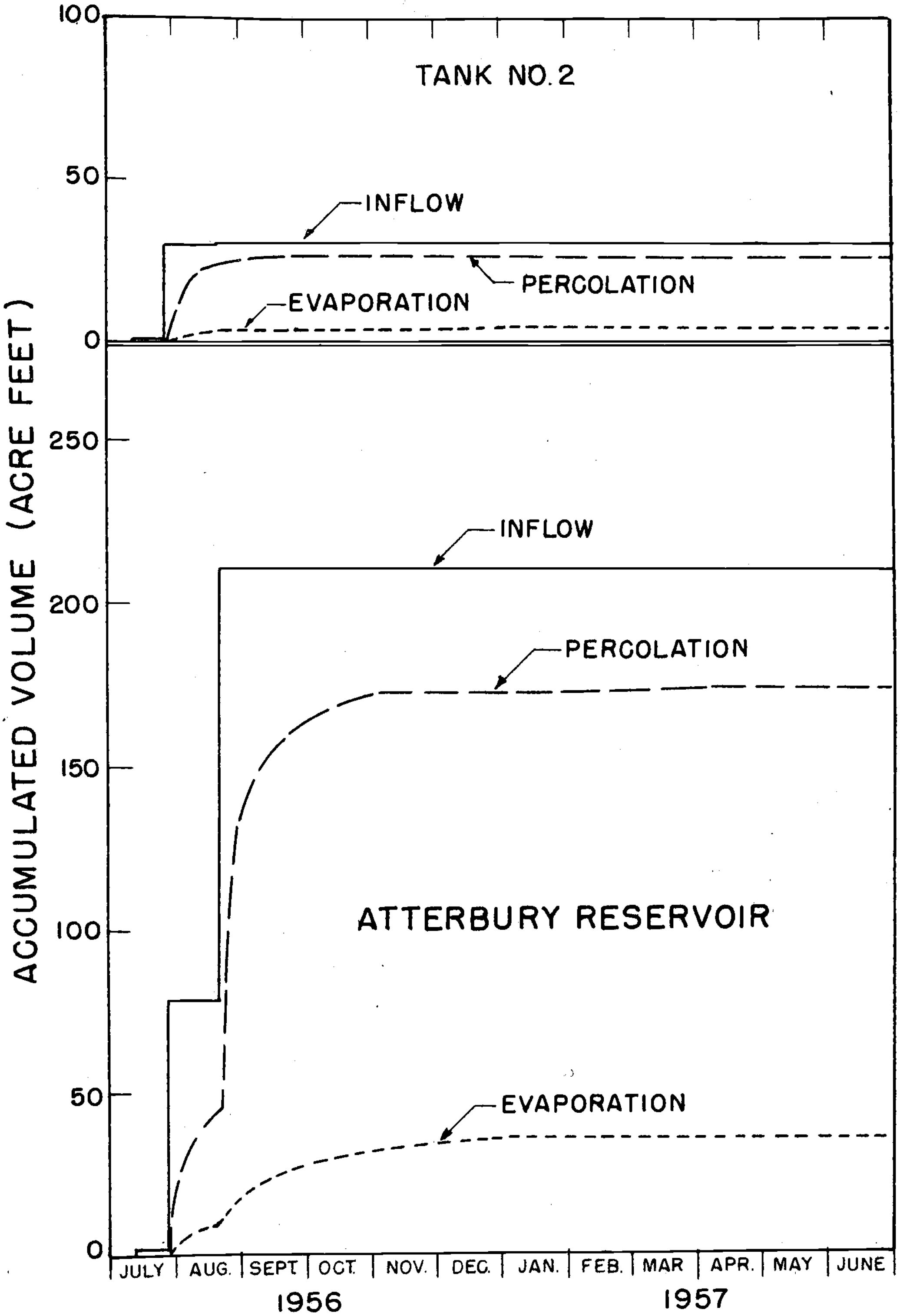


FIGURE 6.1

RESERVOIR LOSSES 1956-57

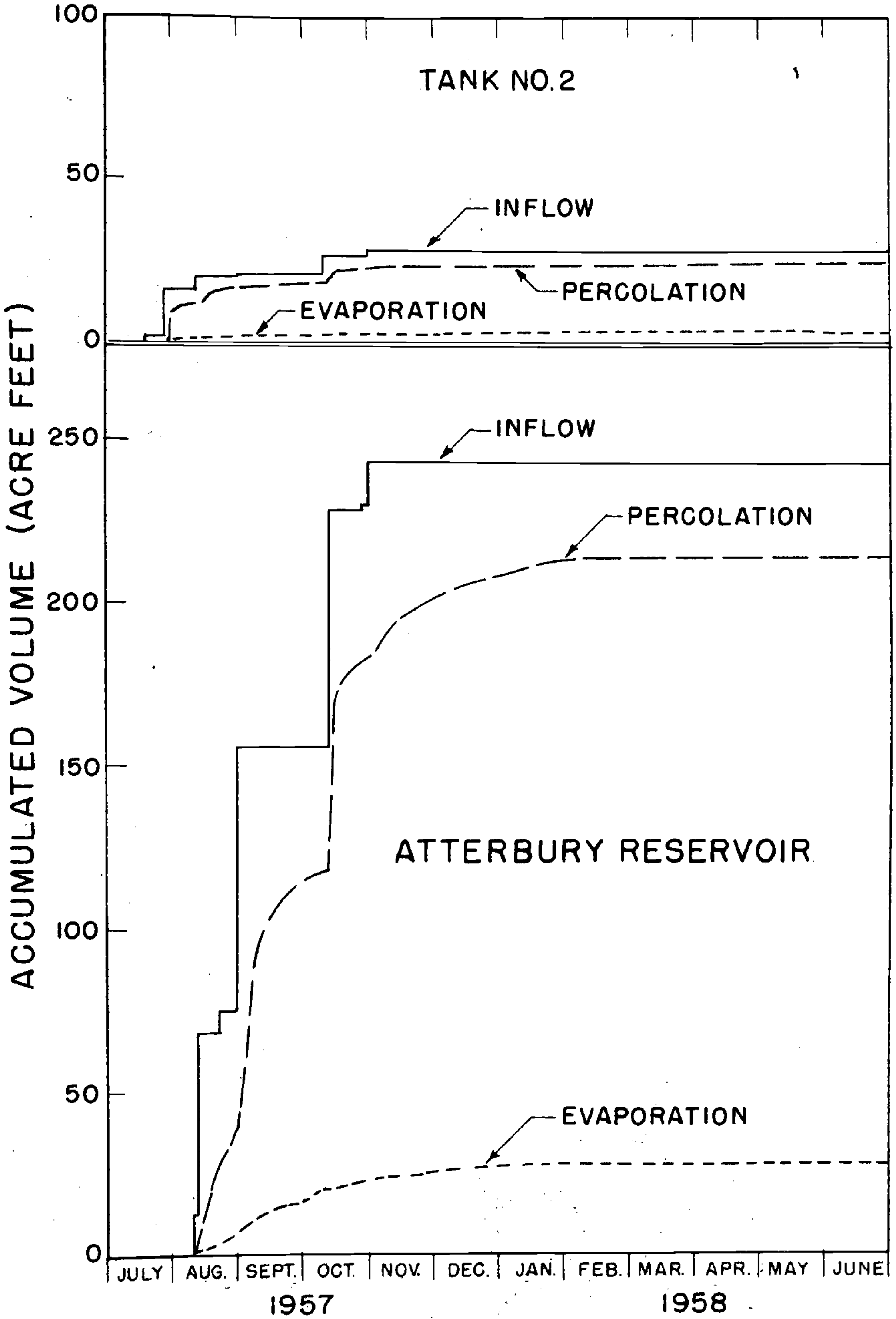


FIGURE 6.2

RESERVOIR LOSSES 1957-58

Most of the loss occurred within a few days after the reservoirs were filled, and much of this percolation went to supply the soil moisture deficit around the periphera of the reservoir. Exploratory soil moisture samples indicated that much of this water is lost by evapo-transpiration. This loss could be reduced by constructing reservoirs with smaller surface area to volume ratios, by sealing the reservoir bottoms, or by removing all vegetation near the reservoir. These practices would cause evaporation losses to become more significant.

6.6 Conclusions

1. Water which penetrated the soil by the process of infiltration was either transpired by plants or evaporated from the soil surface; none of this water was recharged.
2. An unknown amount of the water which was absorbed by the channel alluvium percolated to groundwater.
3. An average of 13.2 percent of the water stored in reservoirs on the watershed was lost by evaporation from the water surface. Of the percolation, an unknown amount was recharged to groundwater, and the remainder was lost to evapo-transpiration.

Chapter 7

CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDY

7.1 Conclusions

In this thesis, the author presented several aspects of a comprehensive study of the hydrologic characteristics of a small, semiarid watershed. While each chapter of this thesis is worthy of more exhaustive study, the investigator felt that because of the short period of record, a general study of the complete project would be more valuable at this time.

Although the period of record was too short to substantiate conclusions regarding quantitative measures of rainfall and water yield over a long period of time, the following general conclusions concerning the instrumentation of the project and the method of study appear to be warranted:

1. In southern Arizona and probably much of the semiarid Southwest, a rain gage density of one per square mile is needed to adequately describe the rainfall patterns resulting from convectional thunderstorms.
2. The areal depth-duration-frequency relations for thunderstorm rainfall developed in

Chapter 4 appear to be promising, and will be a valuable engineering tool when a better method of describing intensity is found.

3. Annual runoff from the Atterbury Reservoir Watershed appears to be less variable than the annual rainfall. For this reason, the three year average runoff of fifteen acre feet per square mile per year is probably not far from the true average.

7.2 Suggestions for Further Study

All of the phases of the project under study at the present time should be continued for at least five more years. A study of stream channel characteristics and the disposition of water that percolates into the channel alluvium, is essential if the project is to fulfill its objectives.

An expanded rain gage network would be of great help in storm center analysis. More recording rain gages would be of value in the development of a method of describing rainfall intensities for individual storms.

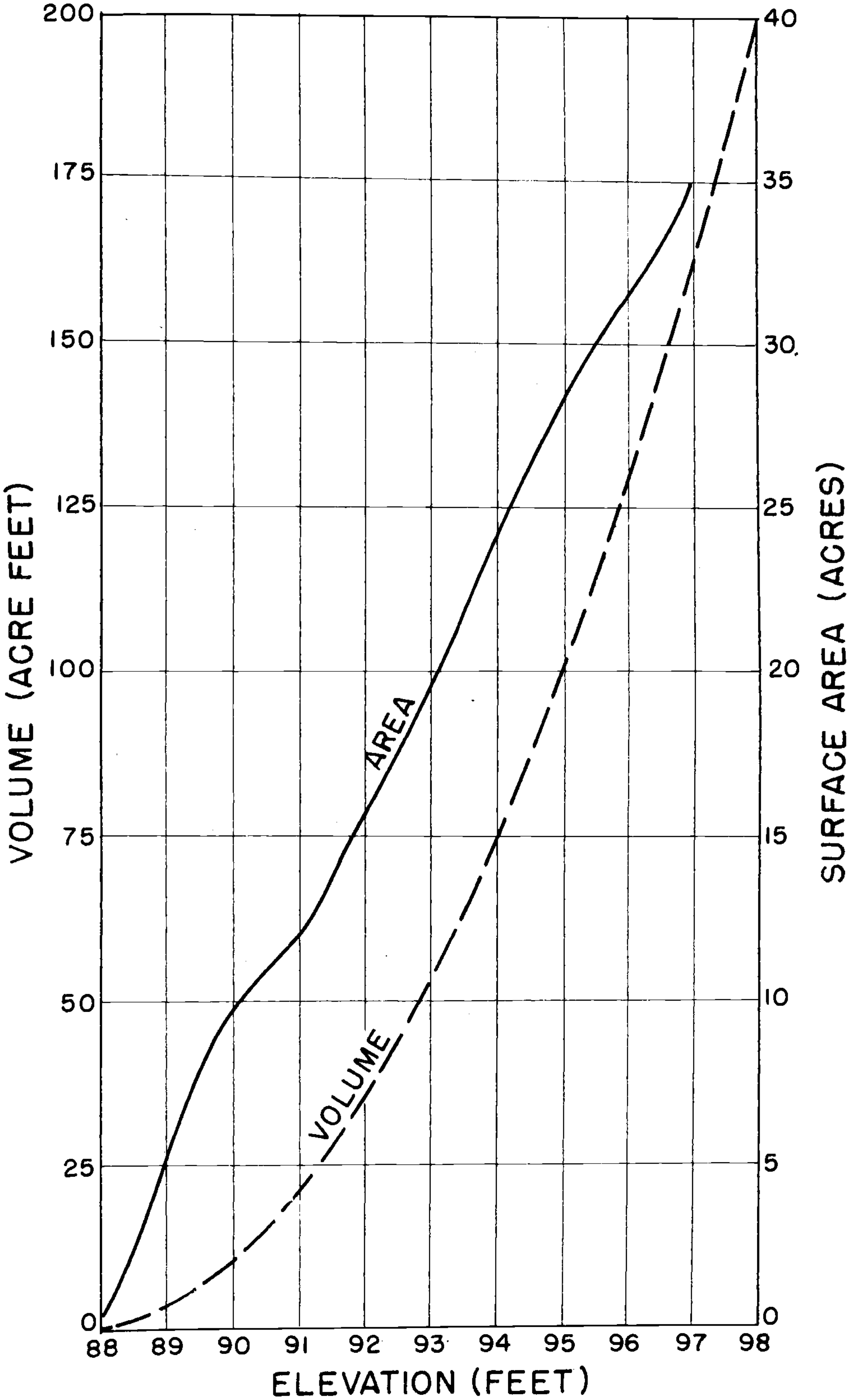
If a reliable rainfall-runoff relationship can be determined, it may be possible to use such a relationship in conjunction with the storm center area depth-duration-frequency analysis to predict water yield.

APPENDIX A

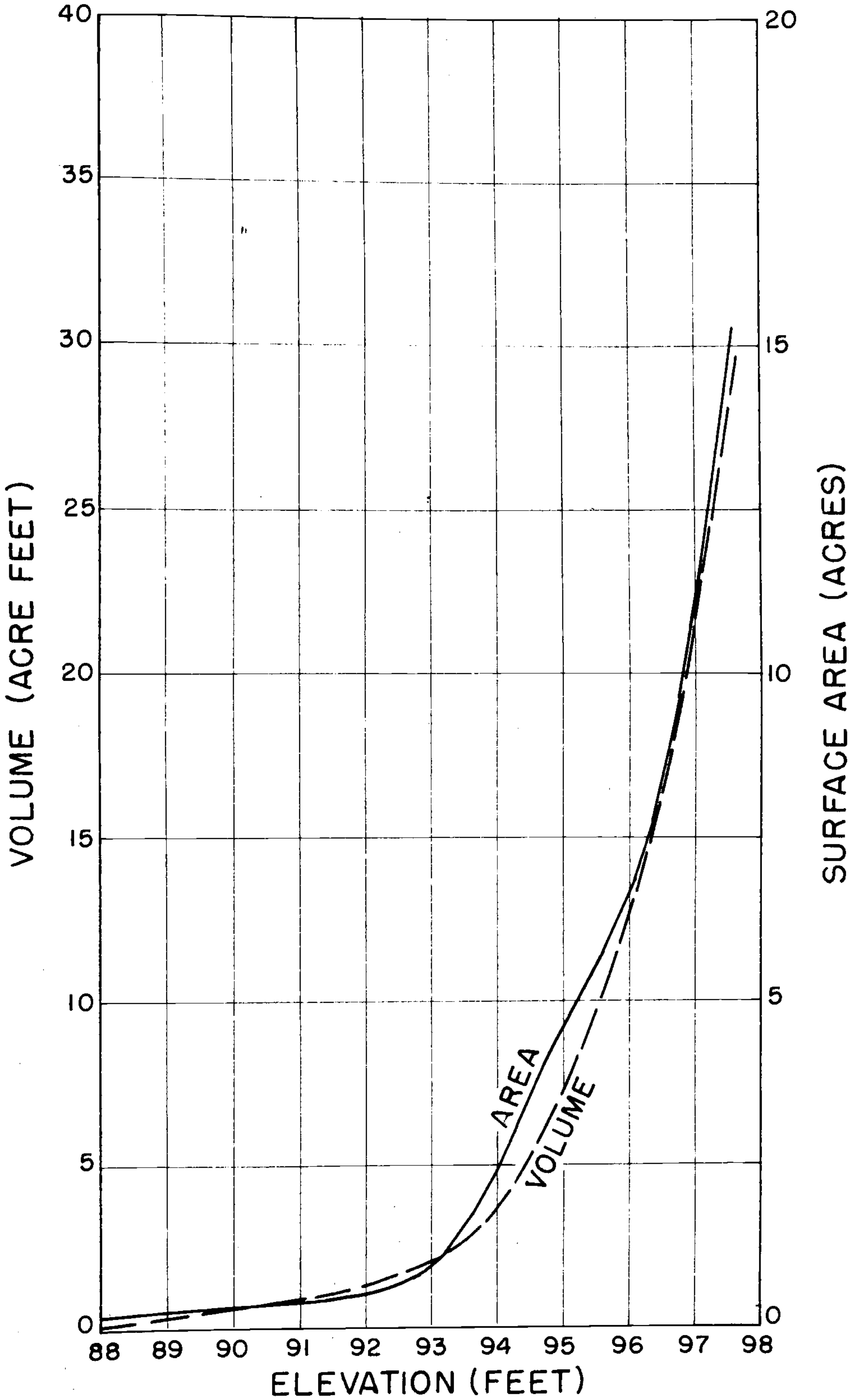
Surface Area and Volume of Reservoirs

v.s.

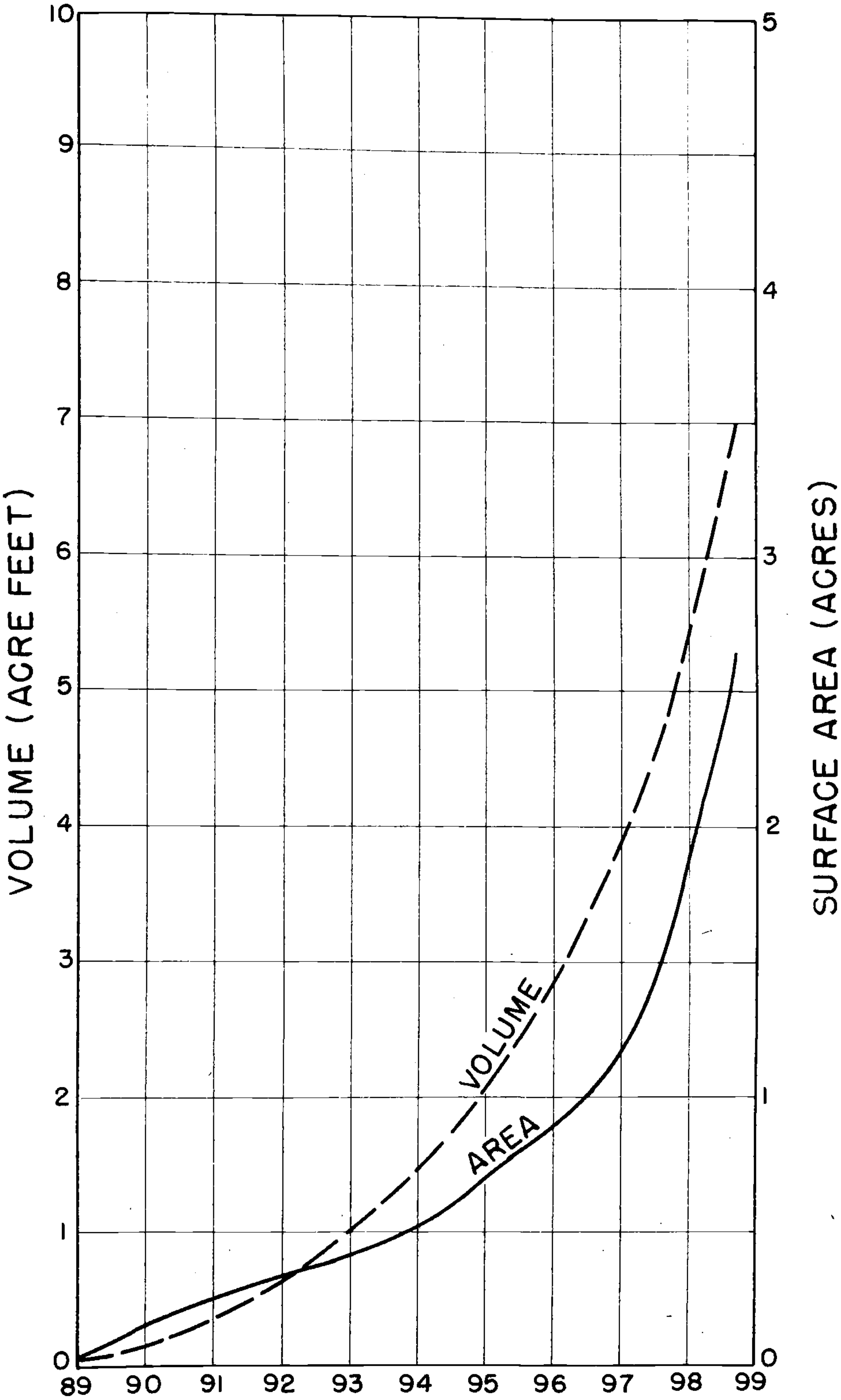
Elevation



SURFACE AREA AND VOLUME V.S. ELEVATION
ATTERBURY RESERVOIR



SURFACE AREA AND VOLUME V.S. ELEVATION
TANK NO. 2



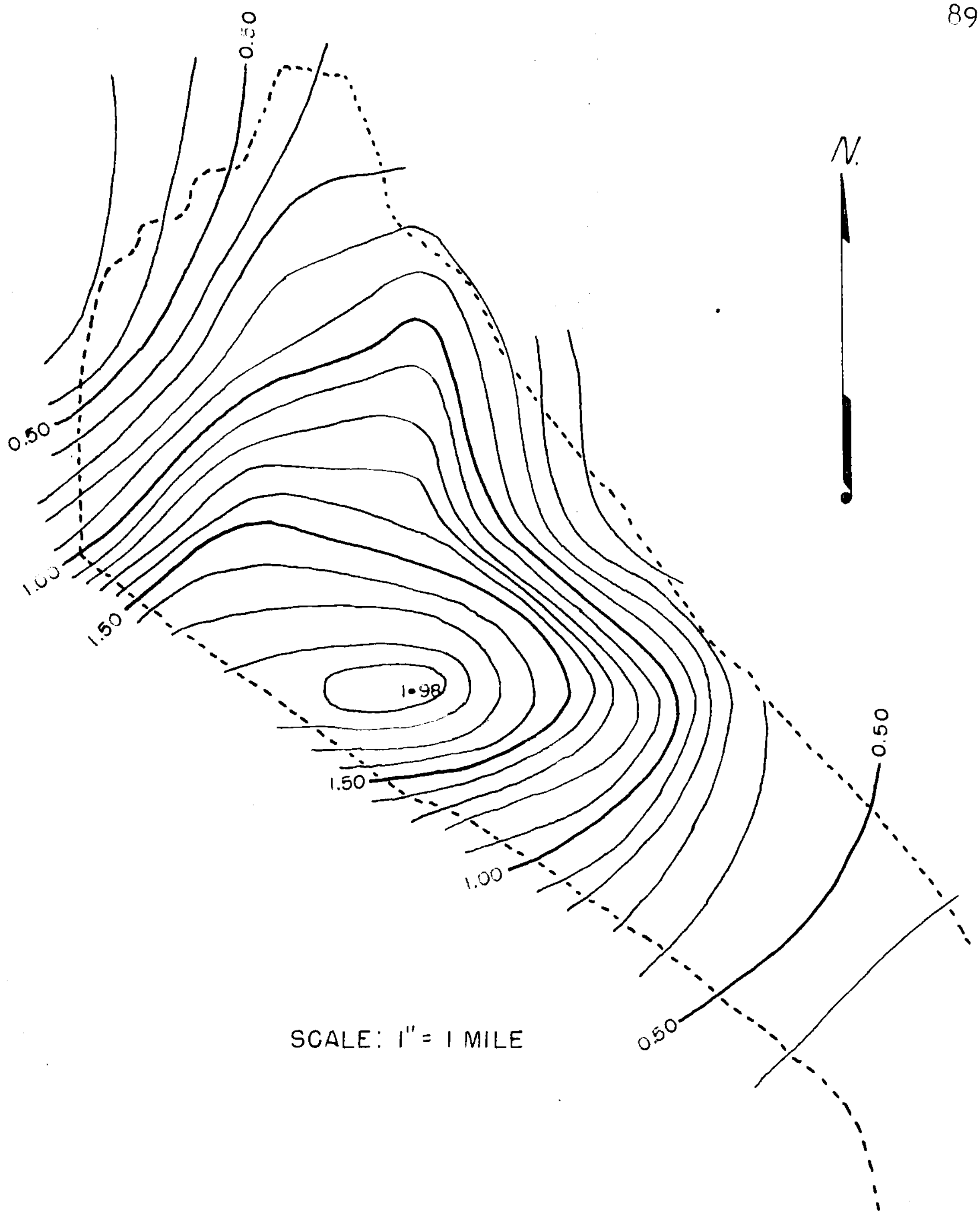
ELEVATION (FEET)

SURFACE AREA AND VOLUME V.S. ELEVATION

TANK NO. 3

APPENDIX B

Isohyetal Map of Typical
Summer Thunderstorm Rainfall



ISOHYETAL MAP
OF
STORM OF AUG. 20, 1956

APPENDIX C

Sample Calculation of the Probability that a Point Rainfall Depth Will Be Equaled or Exceeded

Compute the probability that a two-inch rainfall will be equaled or exceeded at a point near the Atterbury Reservoir Watershed.

From equation (4.7)

$$P_M = \frac{1}{T_1}\left[a_1(P_{m1} - P_{m2}) + a_2(P_{m2} - P_{m3}) + \ldots + a_n\left(P_{mn} - P_{m(n+1)}\right)\right]$$

Select the following increments of rainfall depth above M = 2 inches.

$$M + d_1 = 2.01 \qquad d_1 = 0.01$$
$$M + d_2 = 2.50 \qquad d_2 = 0.50$$
$$M + d_3 = 3.00 \qquad d_3 = 1.00$$
$$M + d_4 = 3.50 \qquad d_4 = 1.50$$
$$M + d_5 = 4.00 \qquad d_5 = 2.00$$

The values of a_n and P_{mn} are found in Figure 4.6 and Figure 4.5 respectively, and are tabulated below.

n	$M+d_n$	P_{mn}	d_n	a_n
1	2.01	0.12	0.01	0.0085
2	2.50	0.054	0.50	4.0
3	3.00	0.025	1.00	12.1
4	3.50	0.012	1.50	22.6
5	4.00	0.006	2.00	36

From section 4.5

$$\frac{1}{T_1} = \frac{1}{3.08} = 0.325$$

Substituting in equation (4.7)

$$P_{2.00} = 0.325\Big[0.0085(0.12 - 0.054) + 4.0(0.054 - 0.025) + $$
$$+ 12.1(0.025 - 0.012) + 22.6(0.012 - 0.006) + 36(0.006)\Big]$$
$$= 0.325\big(0.00056 + 0.116 + 0.157 + 0.1355 + 0.216\big)$$
$$= 0.325(0.625) = 0.203$$

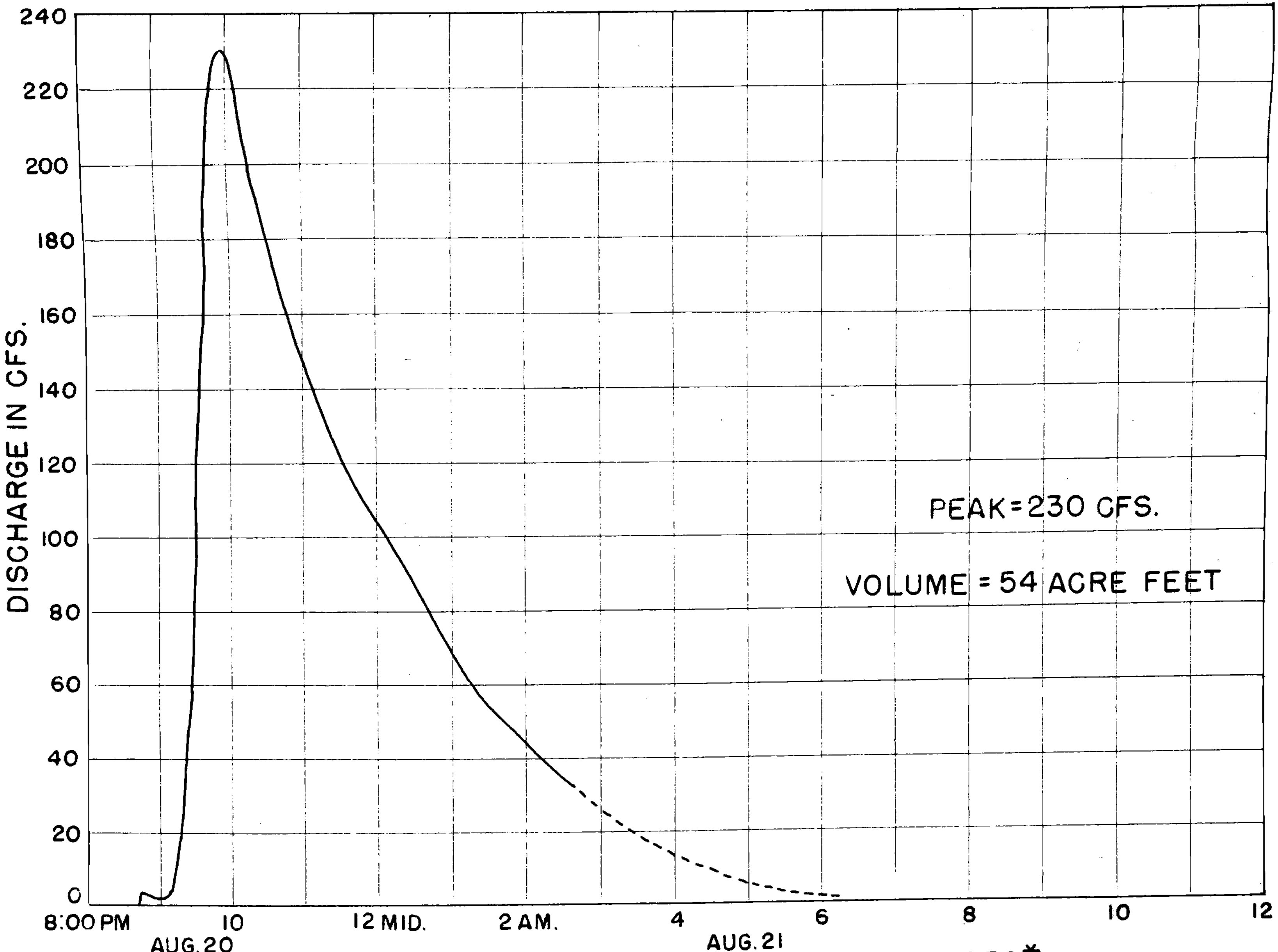
The recurrence interval is then

$$T_{2.00} = \frac{1}{0.203} = 4.92 \text{ years}$$

APPENDIX D

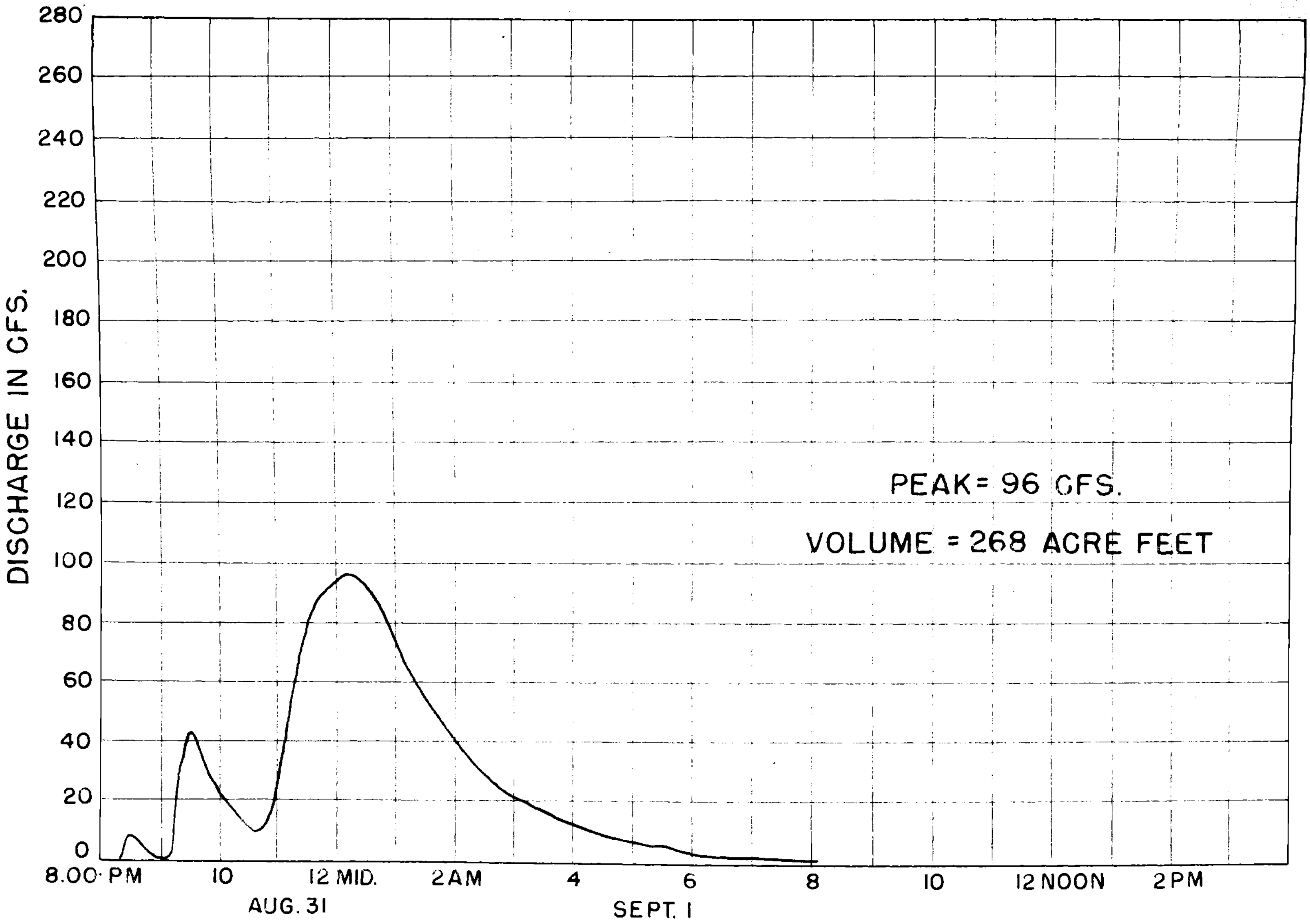
Hydrographs of Annual Peak Flow

Measured at Flume No. 1

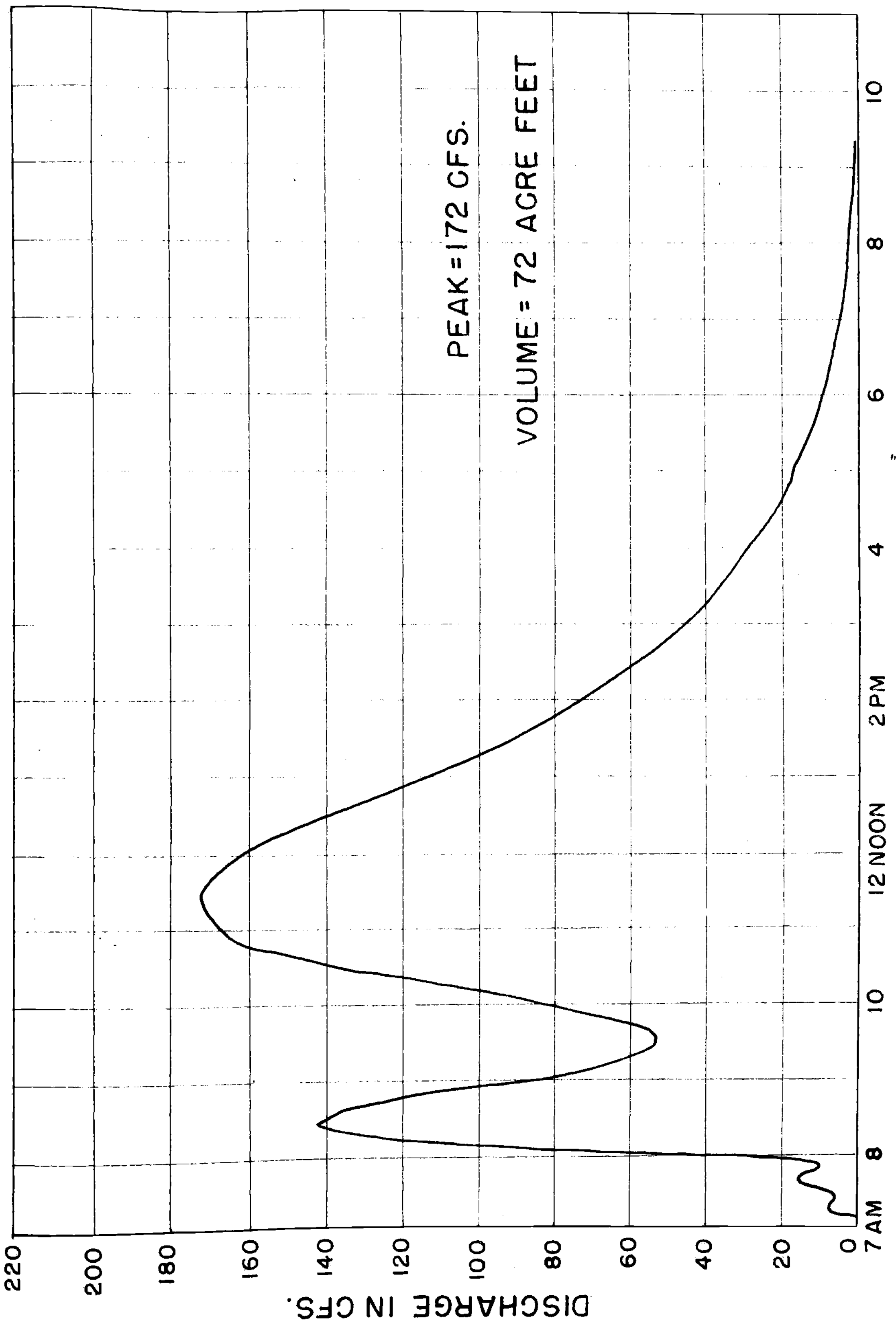


HYDROGRAPH FOR FLUME NO. 1. AUG., 20, 1956*

*FROM VISUAL OBSERVATIONS OF WATER LEVEL.



HYDROGRAPH FOR FLUME NO. 1. AUG., 31, 1957



HYDROGRAPH FOR FLUME NO.1. JULY 29, 1958

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