DESIGN OF BRIDGE AND CULVERT
WATERWAY AREAS FOR RUNOFF
FROM SMALL WATERSHEDS

by

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CHAPTER I

INTRODUCTION AND CONSIDERATIONS

A method of predicting the waterway area of a bridge or culvert required to safely conduct stream flow resulting from intensified rainfall on a small watershed has never been satisfactorily formulated. The vast development, expansion and improvement of the highway system in the United States during the past thirty years has necessitated construction of many small bridges and culverts to carry such stream flow.

Prior to the beginning of this period of sudden growth of the highway system, judgement and rough estimates of expected stream flow were used as a basis for arriving at the required waterway area for a stream crossing. Due to the relatively large number of such installations, it became apparent that a more valid and reliable method was needed, because of the unnecessary expense involved when the waterway area selected was larger than required, and the hazard incurred when it was too small to conduct the peak flows of the stream.

Many methods in the form of empirical equations, charts, diagrams, and tables were prepared as a result in an attempt to solve this problem. Extensive use of these various methods during the past thirty years has shown that they tend to yield fairly good predictions of maximum runoff for large drainage areas, but for areas less than twenty square miles they have been unsatisfactory.

It is the purpose of this thesis to:

(a) Conduct a rainfall-runoff study of a small watershed; and,

(b) Direct attention to various existing methods of determining runoff from drainage areas; and,

(c) Apply several of these methods to the watershed studied, and determine the validity of each method for predicting runoff from a small watershed of the type studied.

Recently much has been written by numerous investigators on this subject. This thesis will summarize some of the different treatments of the subject, and in general, verify or deny the reliability of the methods proposed by these investigators, based on the measurements and observations of the watershed studied. Obviously, such a summary will of necessity include much information which is not new, but which is considered essential for a balanced survey of the subject.

CHAPTER II

HISTORICAL BACKGROUND

In Europe, maximum flood stages were observed as early as 413 B. C. on the Tiber River at Rome, and later in 1000 A. D., records were kept of maximum stages on the River Danube. On the Seine River at Paris, France, records of maximum stages were inaugurated in 1658 A. D. at the La Tournelle Bridge. The longest continuous record of annual flood stages in existence is that of the Roda gauge on the River Nile at Cairo, Egypt. This record was begun in the seventh century.

In America, the longest available record of gage heights for annual floods is that of the Connecticut River at Hartford, Connecticut which was begun in 1828.

The aforementioned records were all those of gage height, and only give an indication of the discharge, due to erosion and shifting of the channel from year to year. The longest record of actual discharge for a river in this country is that of the Merrimack at Lowell, Massachusetts. This record was begun in 1848, and with the exception of five years, has been continuous ever since. However, it was not until 1875 that discharge records were started on other rivers of this country.

Although bridges had been built for centuries, the main design consideration until about 1860 was that of the structural ability of the bridge to support the loads which it was to carry, and the details such as end connections, waterway area, etc., were thought to be less important. At this time, many bridges were being constructed by the railroads and it was seen that these details were often the cause of failure of the bridge in one manner or another. Therefore, they were given more attention. The discovery of theoretical approaches to the design of structural details enabled the solution of many of the problems, but that of the required waterway area remained unsolved. The engineer had been accustomed to merely estimating the elevation above the water surface at which the bridge had to be built in order to allow sufficient area for the flood flows to pass. It became apparent that such a method was in the majority of cases unsatisfactory, and the need for a more reliable method was at hand.

Although rainfall records were being kept in some sections of the country, no correlation was seen between the rainfall on a drainage area and the resulting runoff or discharge of the stream. It was not until 1862 that Nathaniel Beardmore published his "Manual of Hydrology" (2, pp. 5-8) in London, England in which he tabulated the rainfall and runoff of various rivers and streams throughout the world, that such a correlation was indicated. The use of such a relationship in this country was not possible, since there were but few gaging stations and rainfall records in existence.

Nevertheless, the railroad engineer was determined to do something about this problem, and in the eighties many men such as Craig, Burge, Ganguillet, Meyers, Talbot, and others proposed formulas for the discharge of a river or stream based on a coefficient and the drainage area to some power. These formulas were applied wherever discharge records, which were becoming more plentiful, could not be had. The formulas were not entirely satisfactory, and the following was published in 1911 by the American Railway Engineering Association (10, p. 1111):

> "1. There is a general relationship between the best known waterway and runoff formulas. This relationship may be expressed by two terms, a varying coefficient and a varying exponent . . . .
>
> 2. The extent of this relationship for large and small areas is indicated by the Dun Waterway data . . . . ."

The Dun Waterway data was a set of tables based on his experience and records of culverts and bridges which he had installed on the Santa Fe Railway in southwestern United States. This statement by the committee of the association indicated that they were still perplexed as to a solution of the problem.

As more and more rainfall and discharge records for streams and rivers became available, it was observed that the intensity and amount of rainfall had to be considered in predicting the runoff. The formulas were modified to account for intensity of rainfall, but the use of these equations did not, in many cases, yield satisfactory results. As time went on, the problem in the case of large streams and rivers was largely eliminated by the use of discharge records furnished by the United States Geological Survey along with observation of existing bridges. However, a satisfactory method for predicting runoff from drainage areas of less than twenty square miles has not been formulated to date.

CHAPTER III

EXPERIMENTAL STUDY

A fourteen month rainfall versus runoff study was conducted on the North and Middle Forks of Bowers Slough, Benton County, Oregon. The purpose of this study was to obtain the maximum discharge of the stream during this period, and to study the rainfall versus runoff characteristics of a small watershed.

Selection of the Watershed

The first step of the experiment was to select a watershed on which the study could be made. To do this, United States Geologic Survey topographic maps of the Albany and Corvallis Quadrangles were obtained from the U. S. Army Engineers.

The type of watershed desired was one having an area of less than twenty square miles, containing both steep and shallow slopes. The watershed had to be located near Corvallis for transportation reasons, since considerable field work and observations were involved. Also, it was preferable that all parts be accessible from existing roads, since it would be necessary to check actual drainage lines in the field against those indicated by contours of the topographic maps.

After consideration was given to several watersheds, that of Bowers Slough was selected, since it most nearly met the desired qualifications.

Following the selection of the Bowers Slough watershed as the area to be studied, the next step was to establish a gaging station at which gage heights of the stream could be observed. Before such a station could be chosen, many considerations were necessary.

Gaging Stations in General

The place on the stream at which gage heights are observed and where measurements of velocity are made is known as a "gaging station".

Gage heights may be observed by placing a graduated staff near the stream bank, or a weight and chain can be used to measure the distance to the water from a fixed point on a bridge. However, these methods are only satisfactory for a stream whose stage does not change appreciably during a period of one day, since they are read daily. For such a stream as Bowers Slough, these methods are unsatisfactory, since the stream may rise and fall an interval of five feet during one day, and such readings would give no indication as to the true behavior of the stream.

The best method for determining the gage height is by use of a continuous gage height recorder, of which there are many types. A continuous recorder installation consists essentially of a vertical stilling well connected to the stream by an opening or intake pipe, and a recording instrument housed in an instrument shelter. The instrument is actuated by a float on the water surface in the well, to which it is attached by a wire cable. The wire cable is attached through a pulley system to a pen or stylus, which records the fluctuations of the stream on a revolving chart driven by a clock mechanism, thus yielding a gage height-time relationship.

Gage Height-Discharge Relationship

There are two methods of obtaining a gage height-discharge relationship. One of these is to construct a concrete weir across the stream and to correlate the amount of water passing over the weir with the gage height in the reservoir formed by the weir. The other method is by the Velocity-Area Station Method, which is more economical and practical for this type of study than is the former. Briefly, it consists of dividing the cross section of a stream into a number of sections, and for each section the area and velocity are determined, which in turn yields the discharge. For several different gage heights, the discharge is determined, and a curve of discharge versus gage height is plotted. This curve is known as the "station rating curve", or "discharge curve", from which discharges can be obtained for any succeeding gage heights which may occur.

Establishing the Gaging Station

An ideal gaging station should be so located as to conform to the following specifications:

(1) It should be upstream from, but within the range of

influence of the section at which the velocities and gage section areas are to be determined.

(2) Its support should be rigid and immovable so that the elevation of the datum will be unlikely to change.

(3) It should be in a protected spot so that destruction by ice or other floating debris would be improbable.

(4) It should be easily accessible.

(5) It should never be located upstream from a junction with another stream so near as to be affected by backwater from that stream.

(6) The channel of the stream should be stable and permanent, free from vegetation and not subject to overflow.

(7) The measuring section should be on a straight reach of channel, of regular cross section, so that accurate measurements can be made with normal care and without obstruction near the section which might cause undue turbulence, boils, eddies, and negative flow. The maximum velocities should be within the accurate measuring range of a current meter.

Obviously the foregoing conditions are difficult to obtain, and seldom are all found at any one natural gaging site. They are however, conditions which are desirous for an ideal gaging station.

With the foregoing requirements and considerations in mind, a survey of the stream was made, beginning at its mouth on the Willamette River and proceeding upstream.

The most satisfactory site was found at the intersection of Bowers Slough and the Independence Road, approximately 2.75 miles upstream from the mouth on the Willamette River, and 0.4 of a mile below the confluence of its Middle and North Forks. At this location the entire flow of the stream, during flood and normal

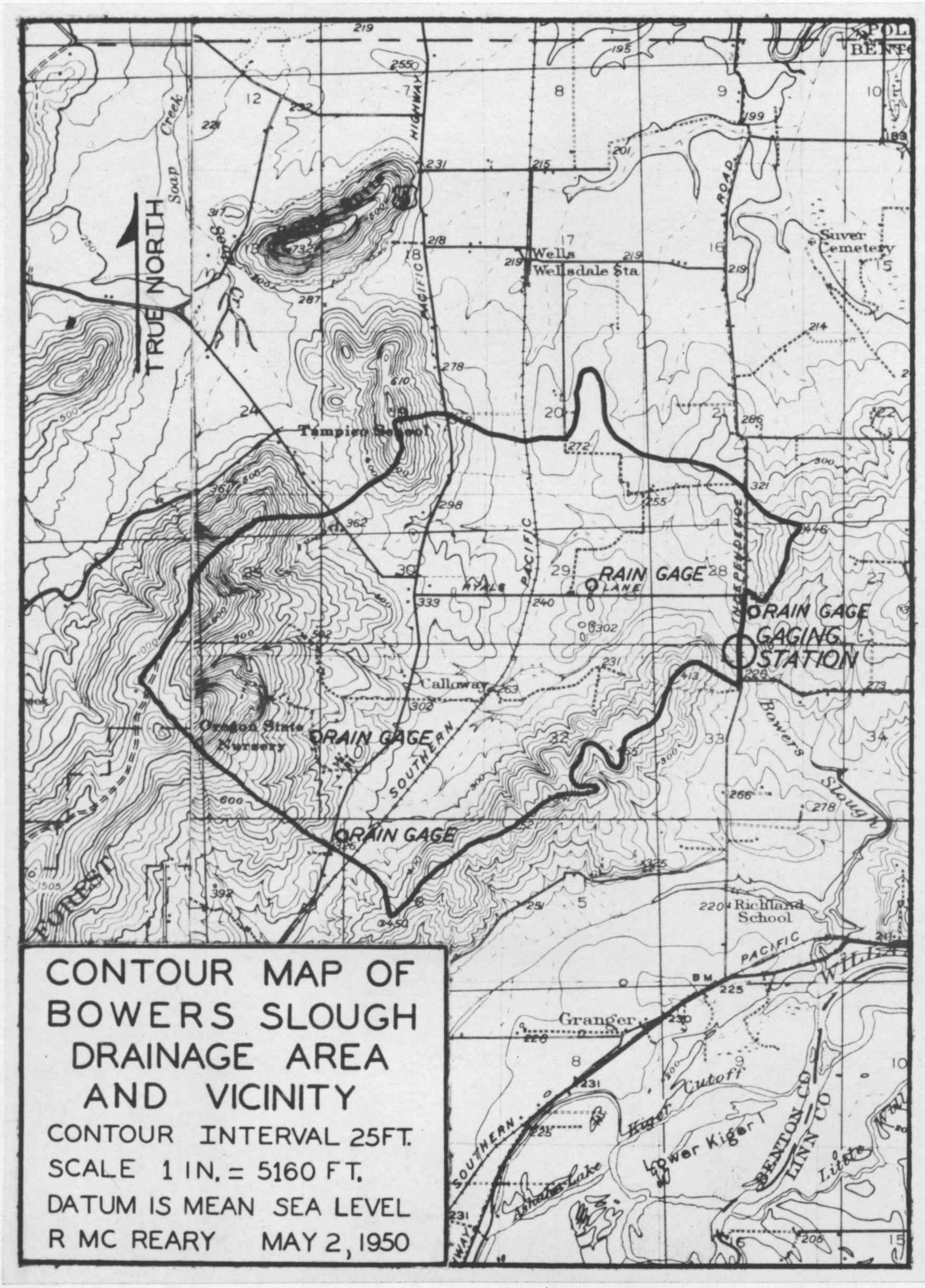


Fig. 1. U. S. Geological Survey Topographic Map of Bowers Slough watershed and vicinity.

stage, according to testimony of residents living near the site, would pass through a 20 foot span concrete bridge. This seemed to be the best location, since at no other point on the stream was there a fixed opening through which all of the water would pass, thereby eliminating the necessity of constructing a weir for a control, which would have been too costly.

The section chosen did not satisfy all of the ideal gaging station requirements, particularly No. 7, since the stream approached the bridge site with a southeasterly bearing, while the bridge lay in an easterly direction. It was apparent that this abrupt change in direction of the stream would bring about a spiral motion at the gage section, which would in turn affect the velocity determinations. However, this condition was unavoidable. In addition to this, there were two timber posts in the center of the stream which were cut off about two feet above the stream bed. They were however, about ten feet downstream from the proposed gage location, and their effect on discharge measurements, though indeterminate,

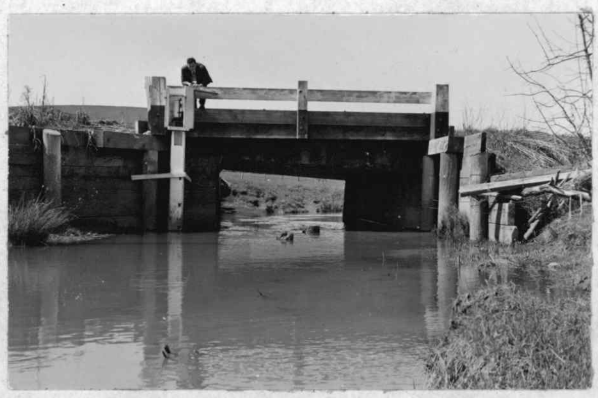


Fig. 2. View of gaging station from point 100 feet downstream.

would be negligible.

So as to prevent damage to the recording instrument and stilling well by debris and ice floes, the recorder was located on the downstream side of the bridge, attached to the bridge rail.

Fig. 3. Closeup of Stevens Type L Gage Height Recorder.

The Gage Proper

The recording gage used in this experiment consisted of a Stevens Type L clock driven recorder, a reduction pulley system and a float. The recorder consists of a base and a double spring seven day clock to which a vertical cylindrical drum is geared such that it makes one complete revolution every two days. A chart, graduated circumferentially into three hour intervals, is held against the drum by two coil spring bands. A pencil stylus, actuated through the pulley system by the float in the stilling well, inscribes the fluctuations of the water surface on the rotating chart.

It was necessary to use a pulley system since a rise of about eight feet was expected in the stream, whereas the recorder, with its own pulleys, could cover only two feet of rise. The pulley system was fabricated in the laboratory, using two aluminum pulleys mounted on a shaft. The pulleys, one four inches in diameter and the other one inch in diameter, were fastened together with small bolts so that they would rotate simultaneously. The pulley system was mounted beneath the platform which supported the recorder.

Each pulley contained a separate wire, fastened to its circumference, and had enough loops of line to satisfy the range over which it was to act. The wire from the larger pulley was attached to the float in the stilling well, while that of the smaller pulley went directly to the movable pulley on the recorder, on which the recording stylus is mounted.

The float used was an elliptical, copper float with a weight attached to the bottom. The weight maintained the float in a partially submerged, vertical position at all times. The entire system was kept taught by means of a counterweight attached to the stylus carriage on the instrument, and the weight of the float in the stilling well.

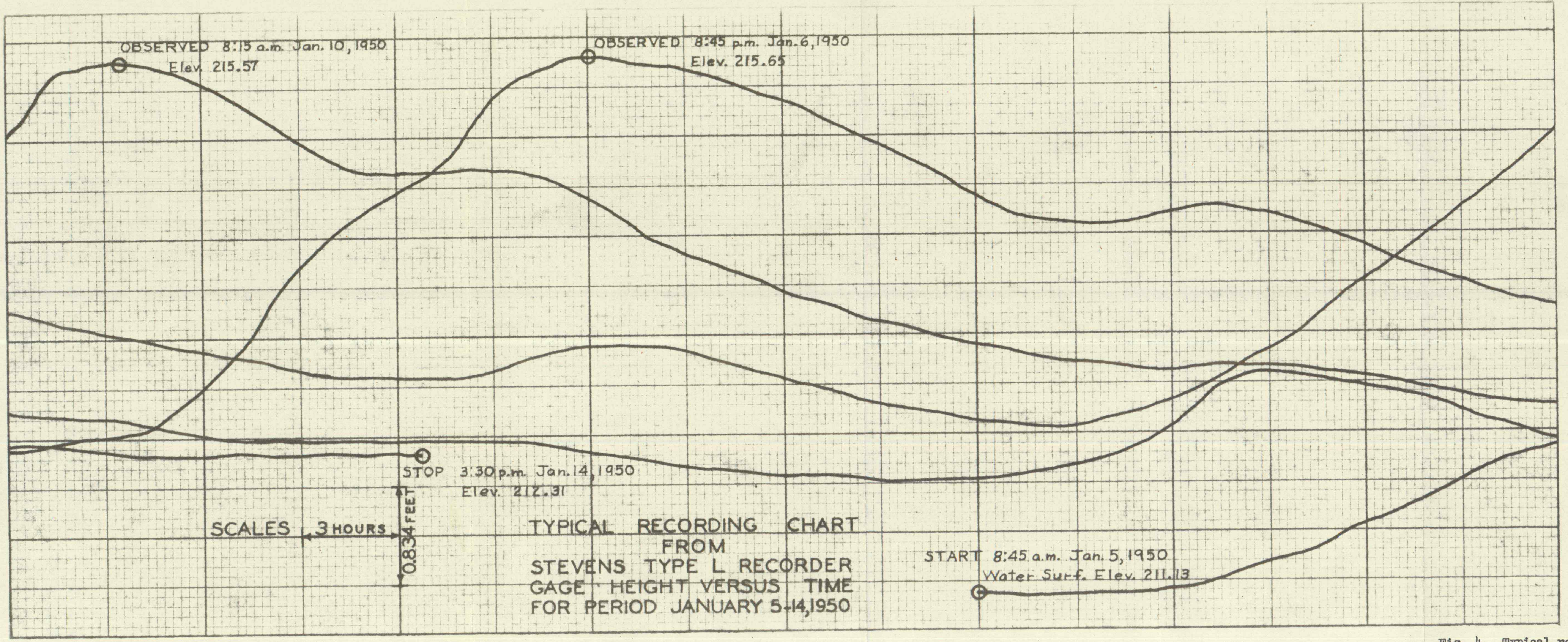


Fig. 4. Typical recording chart from Stevens Type L Recorder used in the experimental study.

Gage Shelter and Stilling Well

The gage shelter was built to conform to the available supports provided by the bridge rail. It consisted of a box with doors, which gave access to the instrument from both side and top. The box was constructed of wood 2" x 4" frame covered with shiplap (see Fig. 3, p. 12). A 6 in. by 6 in. opening was cut in the floor to enable the pulleys to be installed. A platform four inches and one foot square was built in the bottom of the box to provide a means for mounting the pulleys, on top of which rested the recording instrument.

The stilling well was formed of four 2" x 10", eight feet in length, and had an internal area of 64 sq. inches. It was held in position by braces to piling of the wingwalls of the bridge. Holes were cut every two feet vertically in the downstream face to provide means for the water to enter the well. Care was taken to construct the well so that the low flows would not fall below the range of the float. This was accomplished by excavating at the bottom of the well, and digging a small trench two feet long from the bottom of the well to the stream.

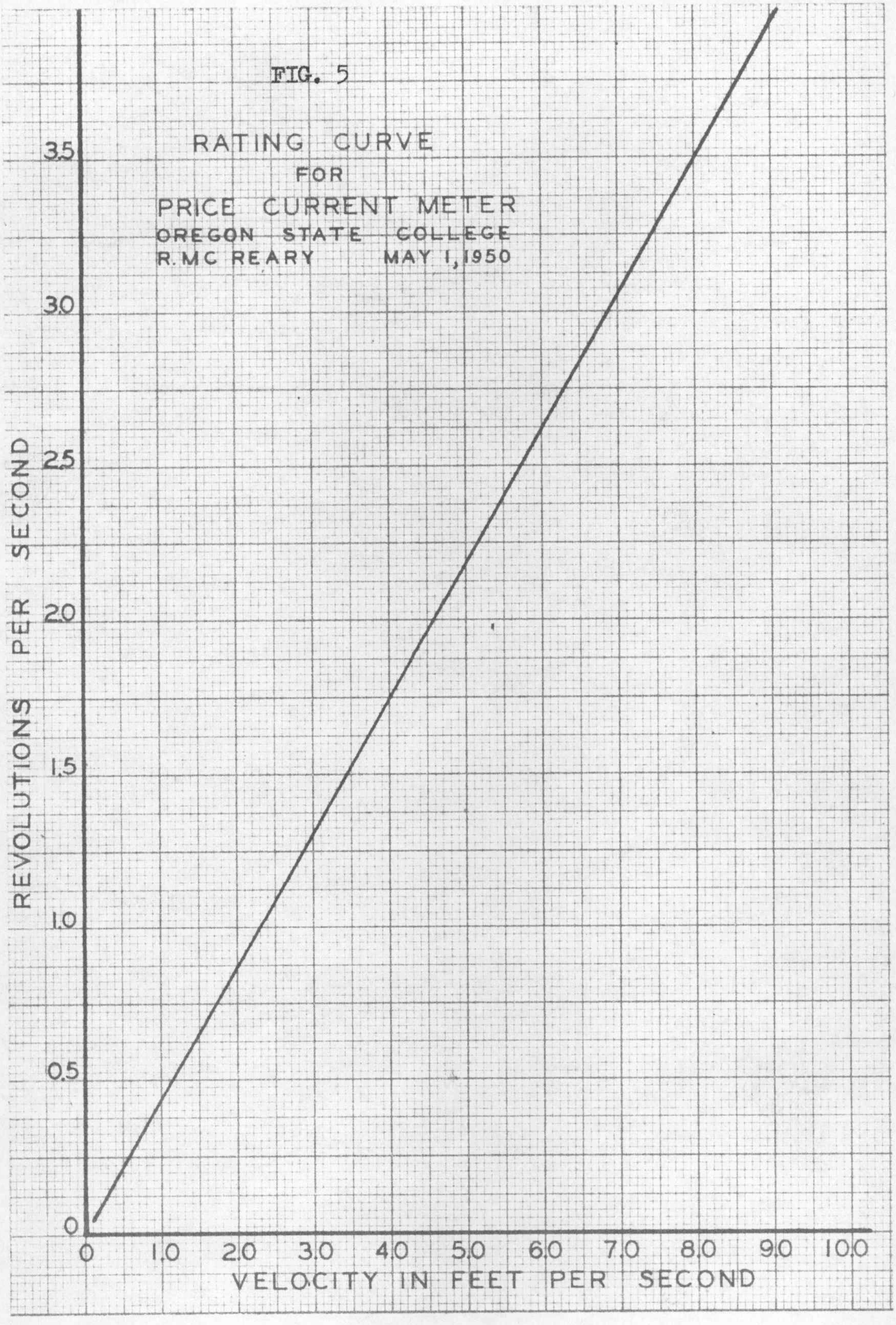
Calibration of Gaging Station

After the gage was installed, it was necessary to make discharge determinations at several stages of the stream, so as to calibrate the gaging station. At various times during a period of two months the stream flow was measured by the velocity area method.

Since the velocity area method was to be used, stations at three foot intervals were established on the felloe guard of the timber bridge deck. By use of a Price current meter, velocities at 0.2 and 0.8 of the sounded depth at each station were found by determining the respective number of revolutions per second of the meter using a stop watch and counter, and referring to a rating curve of revolutions per second versus velocity in feet per second for the current meter. Such a rating curve is shown in Fig. 5, page 17. These velocities were then averaged to give the mean velocity at that station. This procedure was repeated for the several stations. The mean velocities of each succeeding station were then averaged, giving the mean velocity of the section between stations. The area of each section was found by plotting the sounded depths to scale on coordinate paper, and planimetering the individual sections. The product of the section velocity and area gives the discharge in cubic feet per second and the sum of the discharges of the individual sections yields the total discharge of the stream for that particular gage height.

This process was repeated for nine gage heights. The station rating curves shown in Fig. 6 and 7, pages 18 and 19, were plotted from the data provided by the flow determinations (Tables 15-23, Appendix).

FIG. 5


RATING CURVE
FOR
PRICE CURRENT METER
OREGON STATE COLLEGE
R. MC REARY MAY 1, 1950
REVOLUTIONS PER SECOND
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
VELOCITY IN FEET PER SECOND

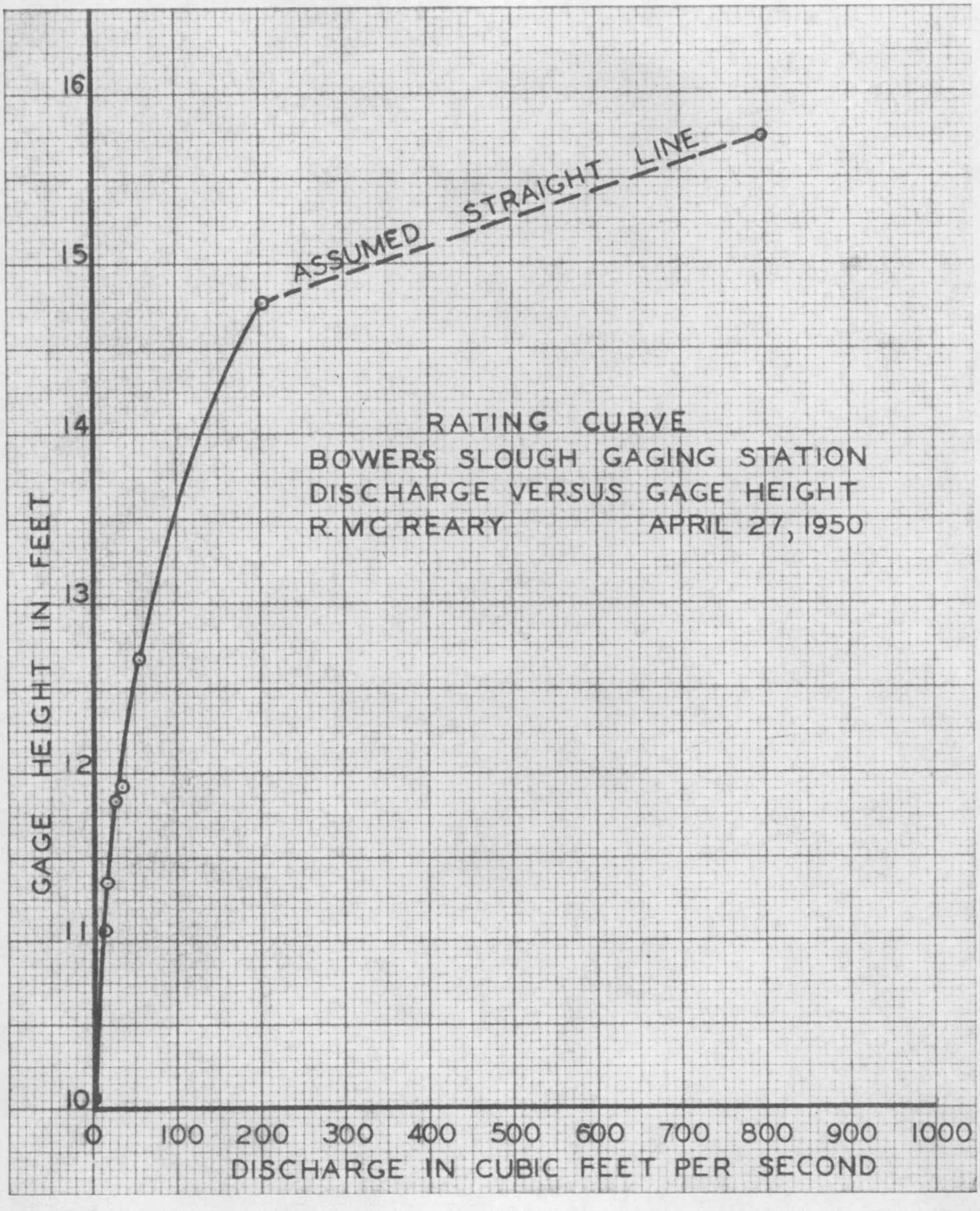


Fig. 6. Gage Height versus Discharge Rating Curve for the gaging station.

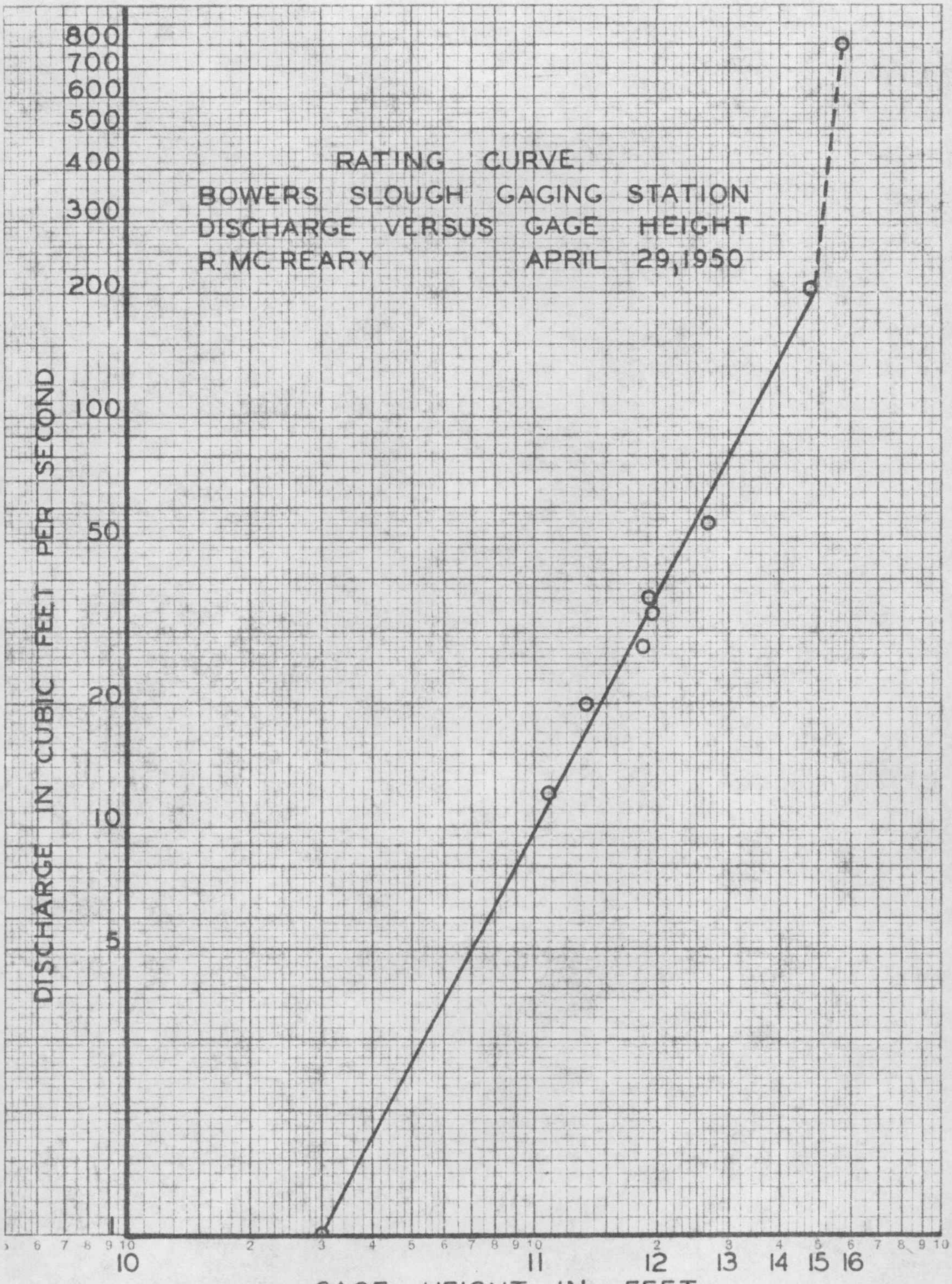


Fig. 7. Gage Height versus Discharge Rating Curve for the gaging station plotted logarithimically.

Maintenance of Gaging Station

Many operational problems were encountered during the period of operation of the gaging station. The maintenance of the recorder and particularly the pulley system required the expenditure of a considerable amount of time at the gaging site. It was necessary to double the counterweight of the pulley-float system in order that sudden rises of the stream would appear on the chart at the time they actually occurred. Also, the only float wire available was three strand steel or copper wire, neither of which are very flexible. Consequently, when a sudden rise would occur, the wire would jump off the pulley, much in the manner of a clock spring, due to the friction of the pulley system. This problem was solved by substituting a linen waxed fish line for the wire. The line was checked periodically for stretching due to moisture, but at the end of a five month period none could be detected.

In addition to maintaining and observing the mechanical effects of the gaging station, it was necessary to rewind the clock and change the recording chart every five days. Each time the chart was changed the gage height of the stream was noted by use of a staff gage mounted on the side of the stilling well, so as to determine the actual reduction ratio of the pulley system.

The gaging station was operated from February 17, 1949 to June 7, 1949, and from November 1, 1949 to April 1, 1950 for this thesis and is still in operation at the present time.

Rain Gages and Stations

In order to get a true picture of the amount and distribution of rainfall in the area, it was necessary to establish four rain gage stations. Although it was desired that continuous recording rain gages be used, it was necessary to use Standard Forest Service gages due to the scarcity and cost of continuous recording gages.

Fig. 8. Standard Forest Service Rain Gage used to measure rainfall.

The standard Forest Service rain gage is shown in Fig. 8. The receiver has a sharp edge eight inches in inside diameter, and is provided with a funnel shaped bottom which conducts the rain to a measuring tube 2.53 inches in diameter, so that the depth of rainfall in the tube is magnified ten times. This tube being five inches high, holds one-half inch of rain. The large outer jacket, known as the overflow attachment, catches the overflow from the measuring tube, in the event that more than one-half inch of rain shall occur between readings. The depth of rainfall is determined by use of a small stick, graduated in inches and tenths (tenths and hundredths of an inch of precipitation) inserted in the measuring tube. If overflow has occurred, the measuring tube is removed and emptied and then refilled with the water from the overflow jacket. This process is repeated as many times as necessary, and the final partial tubeful is measured with the stick.

Some of the considerations involved in the exact location of the rain gages were as follows:

1. The least distance between the gage and any obstruction such as a house or tree should be not less than the height of the obstruction.
2. It should be located behind some barrier such as a fence or a row of shrubbery, so as to minimize the effect of eddy currents caused by the wind, which tend to distort the true values.

Since it was necessary to read the rain gages daily, the expense involved in transportation made it impossible for the writer to observe the readings. Therefore, the gages were set up near selected residences in the drainage area, wherever local residents could be retained to make the readings, since no funds were available for compensation to the readers. A better distribution record of of the rainfall could have been obtained if the gages had been installed at equal intervals across the area. However, due to the aforementioned reasons this could not be done.

The rain gage stations were established at the following locations:

1. Station No. 1 at the Coyle Lumber Mill, near the southwest boundary of the drainage area along the Pacific Highway 99 W at elevation 325.

2. Station No. 2 at the MacDonald Forest Nursery Farm in the central western section of the drainage area at elevation 400.

3. Station No. 3 at the Hilderbrand residence in the central eastern section, at elevation 300.

4. Station No. 4 at the Wilcox residence, slightly outside the eastern boundary, along Independence Road at elevation 270.

Standard laboratory thermometers were placed at Stations 1 and 4. The temperature and rainfall were recorded simultaneously at these two stations. All gage readers took the readings at 5 p.m. daily, so that comparable 24 hour rainfall totals could be obtained.

These rain gage stations were in operation from February 21, 1949 until May 31, 1949, when the stream ceased to flow. When the field experiment was re-activated in September 1949, it was not possible to retain anyone to read these gages. For the duration of the field experiment it was necessary to rely upon the rainfall readings obtained at the U. S. Weather Bureau Station at Corvallis, eight miles south of the drainage area. The method of

Fig. 9. Aerial photograph of Bowers Slough watershed.

correlation of the Corvallis rainfall with that of the drainage area will be discussed at a later point in the thesis.

Location and Characteristics of the Watershed

The watershed of Bowers Slough is located at an approximate longitude of 123° 12' W, and a latitude of 44° 40' N, in Township 10 S Range 5 W, Sections 24, 25, 36 and Range 4 W, Sections 19, 20, 21, 28, 29, 30, 31, 32, 33; in Township 11 S, Range 5 W, Section 1, and Range 4 W, Sections 5 and 6. It lies 7.5 miles North 25° East of the city of Corvallis, Oregon, and 6.5 miles North 25° West of the city of Albany, Oregon. It has an area of 7.34 square miles, and drains an a southeasterly direction toward the Willamette River. It is nearly rectangular in shape, approximately 3.3 miles in a NE-SW direction and 2.3 miles in a SE-NW direction.

Characteristics of Bowers Slough

Bowers Slough may be classed as an intermittent stream, since it flows only part of the year. Its normal period of discharge is from November through May, or about eight months of the year. The average discharge throughout this period is about thirty cubic feet per second (cfs), and its maximum recorded discharge is 796 cfs. Normally, the peak discharges occur during November, December, January, February and March. The normal velocity of the stream is about one foot per second, and a maximum flood velocity of 9.41 feet per second (fps) has been measured.

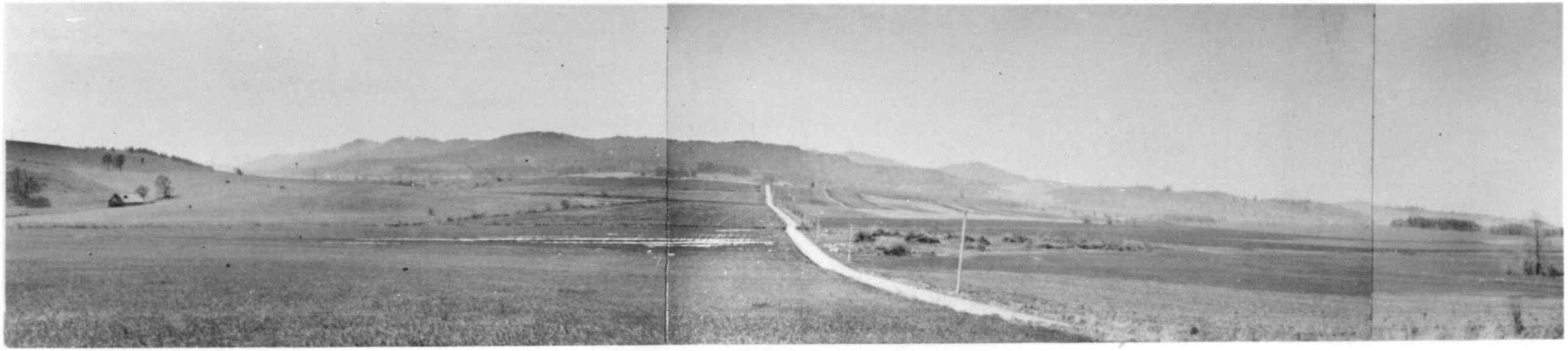


Fig. 10. Looking due west along Ryals Lane from east boundary of drainage area. Junction of the North and Middle Forks of Bowers Slough is at left center. (Taken April 1950)

Fig. 11. Drainage area under one foot snow cover in January 1950. MacDonald Forest appears in the background. Gaging station is off left edge of photograph.

The main branch of the stream is 6.4 miles long from its origin in MacDonald Forest to its mouth at the Willamette River and the total fall of the stream in this distance is 1100 feet. In its upper reaches, it has a slope of 600 feet per mile over a distance of 1.6 miles, and in the remaining length, its slope is 45 feet per mile.

The thread of the stream is fairly straight and uniform, as shown on the contour map in Fig. 1, page 10 and the aerial photograph in Fig. 9, page 24. There are three major forks of Bowers Slough, designated as the North, Middle and South Forks. Of the three, only the North and Middle Forks lie within the drainage area studied. The Middle Fork contributes the major part of the discharge, due to its larger drainage area. The North and Middle Forks join at a point which lies 0.4 of a mile upstream from the gaging station.

The terrain of the watershed might be classed as gently rolling. There are high hills along the western, southeastern, and eastern boundaries, while the central, northern, and southern sections are gently sloping areas. The contour map shown in Fig. 1 gives the topography of the area.

The higher elevations contain dense growths of Douglas Fir, which cover roughly thirty per cent of the area. On the lower slopes of the hills there are a few oak trees, particularly along the northwestern boundary, and some willows are found along the stream banks. The remainder of the drainage area is occupied by

Fig. 12. Looking downstream from gaging station on January 3, 1950.

Fig. 13. Looking upstream from gaging station on January 4, 1950.

Fig. 14. Downstream view in April 1950.

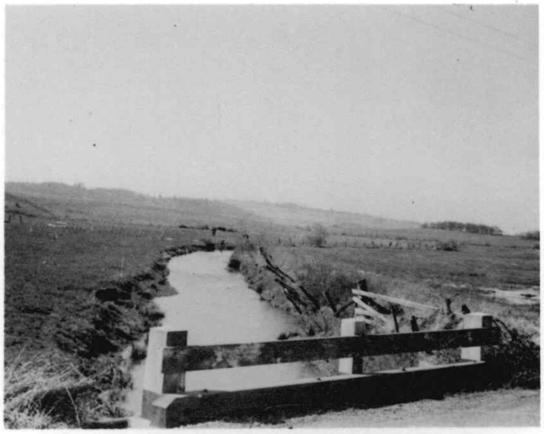


Fig. 15. Upstream view in April 1950.

Fig. 16. View of gaging station during flood of Jan. 5-9, 1950.

Fig. 17. Looking upstream from gaging station on June 15, 1949.

cultivated fields and pasture lands, which make up roughly fifty per cent of the total area. The aerial photograph shown in Fig. 9 shows the distribution of cover over the area.

The geologic cross section of the Willamette Valley in Fig. 18 gives a general picture of the distribution of surface soil types in a vertical plane. The surface soil map of Fig. 19, page 31, gives an aerial distribution of the various soil types found on the drainage area. Table 1, page 30, lists the percentages by area of the individual types of soil. A detailed description of each of these various soils is given in the Appendix.

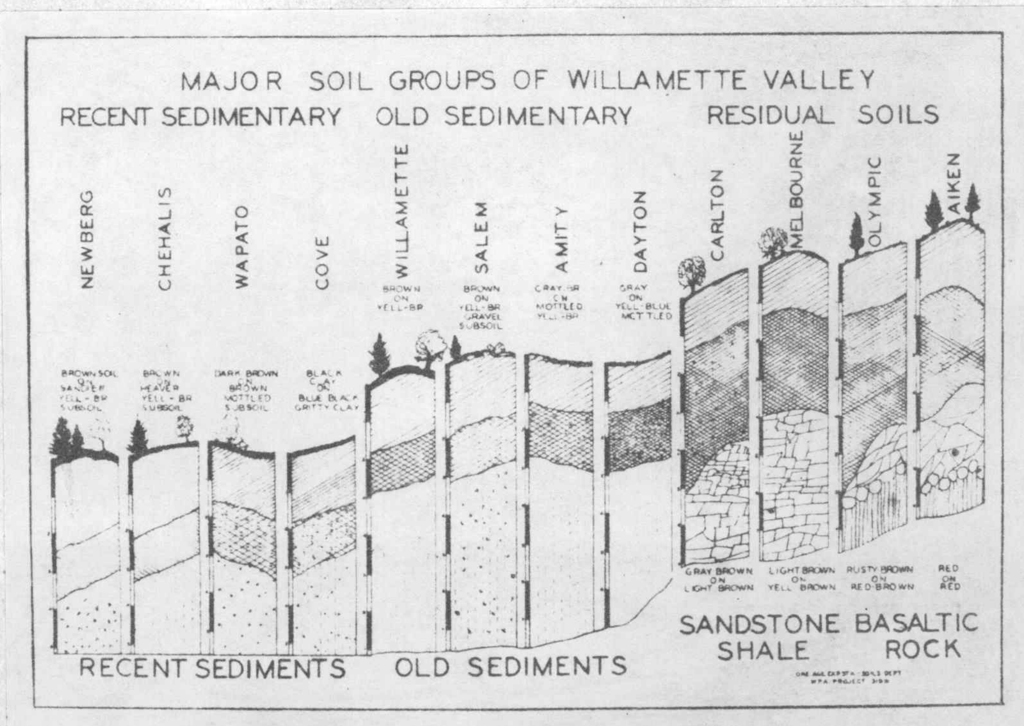


Fig. 18. Geologic Section of Willamette Valley.

Table I

Surface Soils
of Bowers Slough Watershed

Soil Type	Map Symbol	Percent of total area
Aiken Silty Clay Loam	Al	21.3
Amity Silty Clay Loam	Ac	16.9
Olympic Clay Loam	Ol	11.9
Wapato Silty Clay Loam	Wc	10.9
Melbourne Clay Loam	Mc	10.0
Carlton Silty Clay Loam	Cs	8.8
Melbourne Silty Clay Loam	Ms	6.7
Rough Mountainous Land	R	5.6
Willamette Silt Loam	Wl	3.5
Cascade Clay Loam	Ca	2.3
Cove Clay	Cc	2.1

Percentages by Types

Silty Clay Loam	64.6
Clay Loam	24.2
Silt Loam	3.5
Clay	2.1
Rough Mountainous Land	5.6

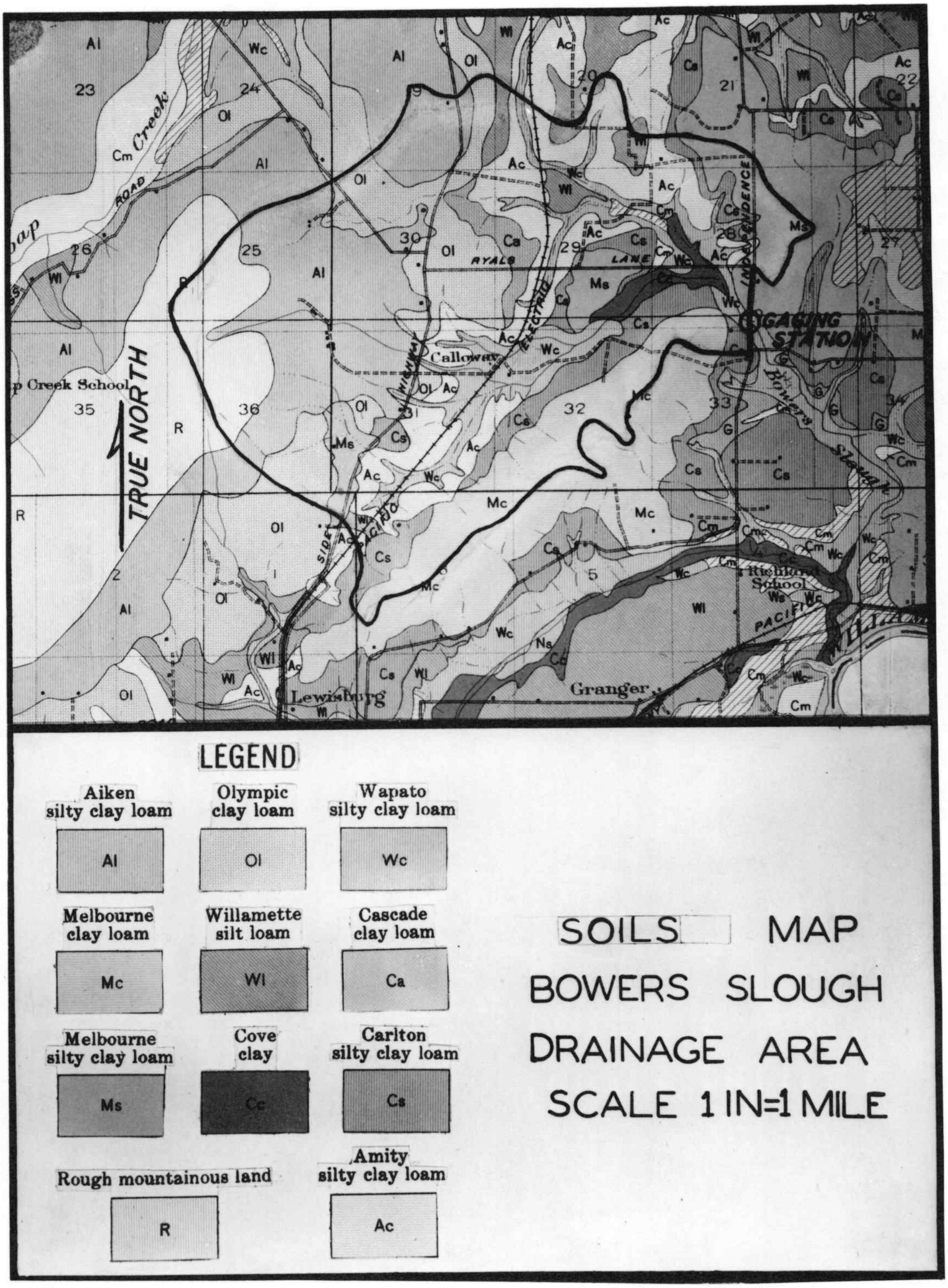


Fig. 19. Soils Map of Bowers Slough Drainage Area.

Rainfall on Bowers Slough Drainage Area

The majority of the rainfall in this region is of the orographic type. Briefly, it is caused by moisture laden air masses being forced by winds to rise over a topographic barrier such as mountains. When such masses rise, they expand and cool, resulting in precipitation. Since Bowers Slough drainage area lies on the leeward side of the Coast Range, fifty miles east of the Pacific Ocean, the moisture laden offshore winds of the Pacific travel into the Willamette Valley through the passes of the mountains. The winds which travel over the mountains are low in moisture content, since most of it has been deposited on the windward side due to orographic action. The winds which come through the passes retain their moisture, and travel in a northerly direction. The hills of Bowers Slough drainage area lie in the path of these winds, and cause them to be deflected upward, producing orographic rainfall.

As mentioned previously, rainfall records were kept on the drainage area proper from February 21 to May 31, 1949, at which time they were discontinued due to lack of observers. Though of relatively short duration, these records, as tabulated in Table 31, Appendix, did yield some interesting tendencies of the rainfall at the drainage area.

It was definitely established that the total rainfall during the period of observation was largest on the upper slopes of the drainage area. This is characteristic of orographic rainfall, the predominant type in this region. The following values of total rainfall illustrate the variation at different locations.

At rain gage Station 1, located at the foot of MacDonald Forest, elevation 325, the total rainfall during the period of observation was 9.41 inches. For this same period, Station 2 in MacDonald Forest, elevation 400, recorded a total of 9.03 inches. At Station 3, elevation 300, the total was 8.58 inches, and at Station 4, elevation 260, 8.15 inches. Also, at the U. S. Weather Bureau Station, Corvallis, Oregon, the total for the period was 6.27 inches. This station is approximately eight miles southwest of Station 1, at an elevation of 270. It is to be noted that the greatest total precipitation did not occur at the highest elevation. This was probably due to the protection afforded the gage at Station 2 by the nearby forest.

Likewise, the daily rainfall was not always greatest at the station of highest elevation. The daily record of rainfall, Table 31, Appendix, shows that occasionally the precipitation at Stations 3 and 4 was larger than that of Stations 1 and 2. The reason for this is unknown, but it does illustrate variation of the rainfall on the area.

Correlation of Rainfall at Bowers Slough Watershed and Corvallis

Since the rain gages were removed from the watershed on May 31, 1949, it was necessary to establish a correlation between the rainfall at the watershed and Corvallis, the location of the nearest rain gage, for the remainder of the study.

The average total rainfall during the period of study was 8.79 inches on the watershed, while that of Corvallis was 6.27 inches. These totals indicate that the rainfall at Bowers Slough is approximately 1.4 times as great as that at Corvallis. Examination of the data of Table 31, Appendix, shows that this is true in many cases, while in others it is in error by 100 per cent. However, since it was not possible to obtain rainfall records for the remainder of the experiment, the rainfall on the watershed was assumed to be 1.4 times that at Corvallis.

Relation of Rainfall to Runoff for the Watershed

Graphs of the average daily rainfall and maximum daily discharge were prepared for the months of February 1949; January, February, and March 1950 (Fig. 20-23, pp. 35-38). The graphs show the general relationship of rainfall to runoff for the watershed. The data used to construct these graphs is contained in the Appendix.

In order to ascertain more closely the relationship between the rainfall and runoff of the watershed, six floods or freshets were selected from the discharge records of the stream. These floods are examined separately for this relationship.

Flood of April 30 to May 4, 1949. The total hydrograph of this flood is shown in Fig. 24, page 40. The flood was caused by a total mean rainfall of 2.61 inches on the watershed during a period of four days. This rainfall was preceded by a relatively

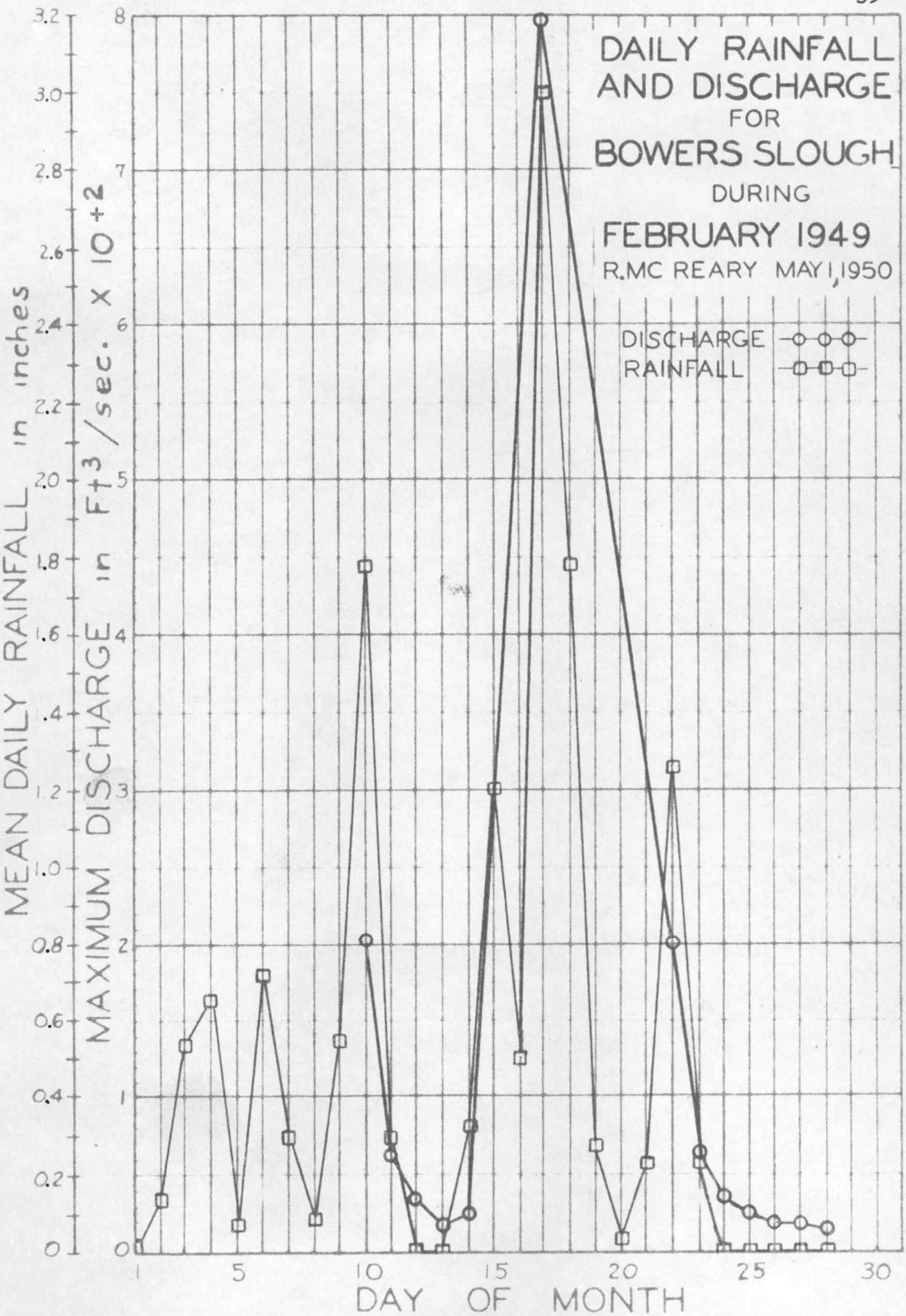


Fig. 20. Graph of rainfall and discharge for February 1949.

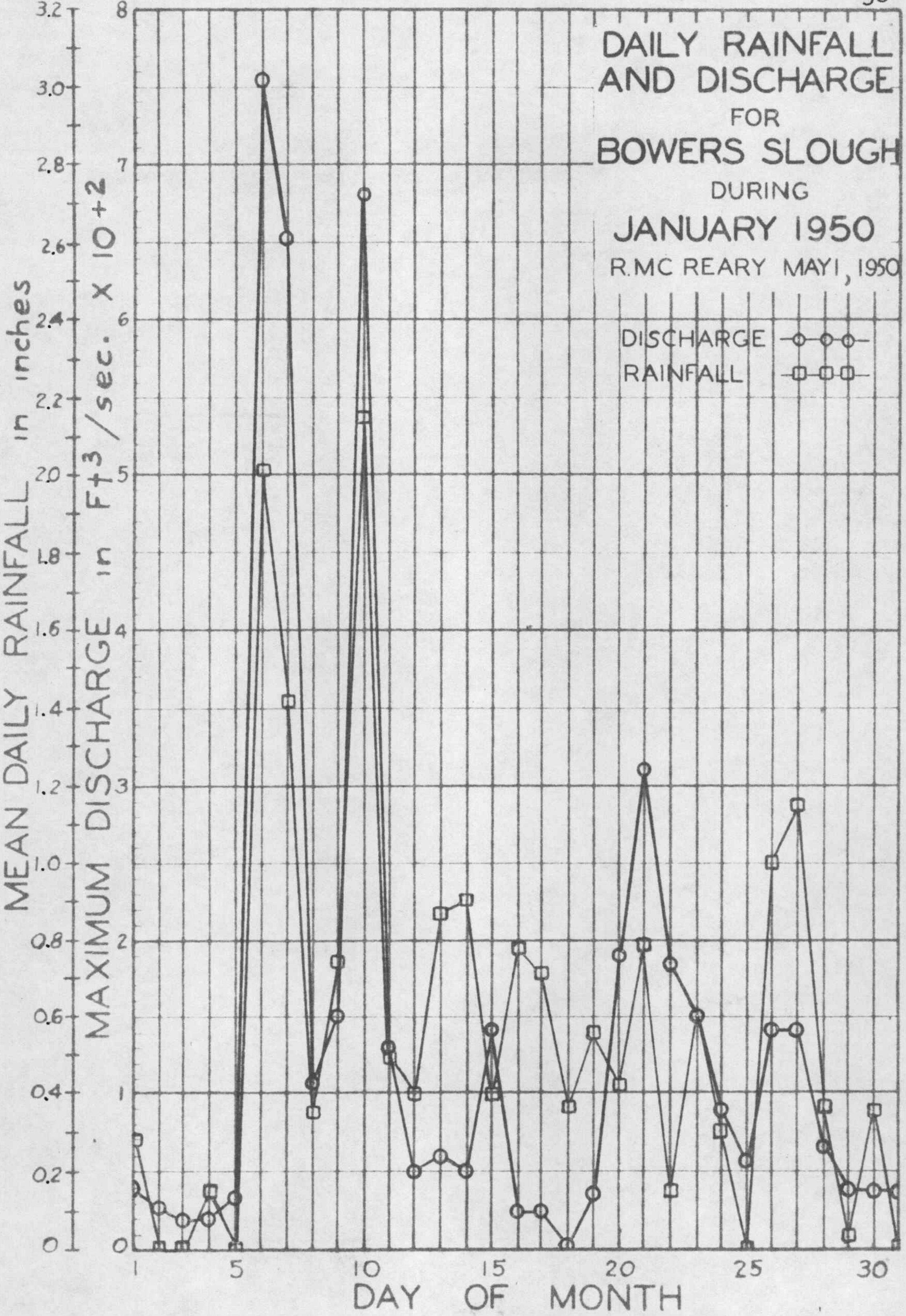


Fig. 21. Graph of rainfall and discharge for January 1950.

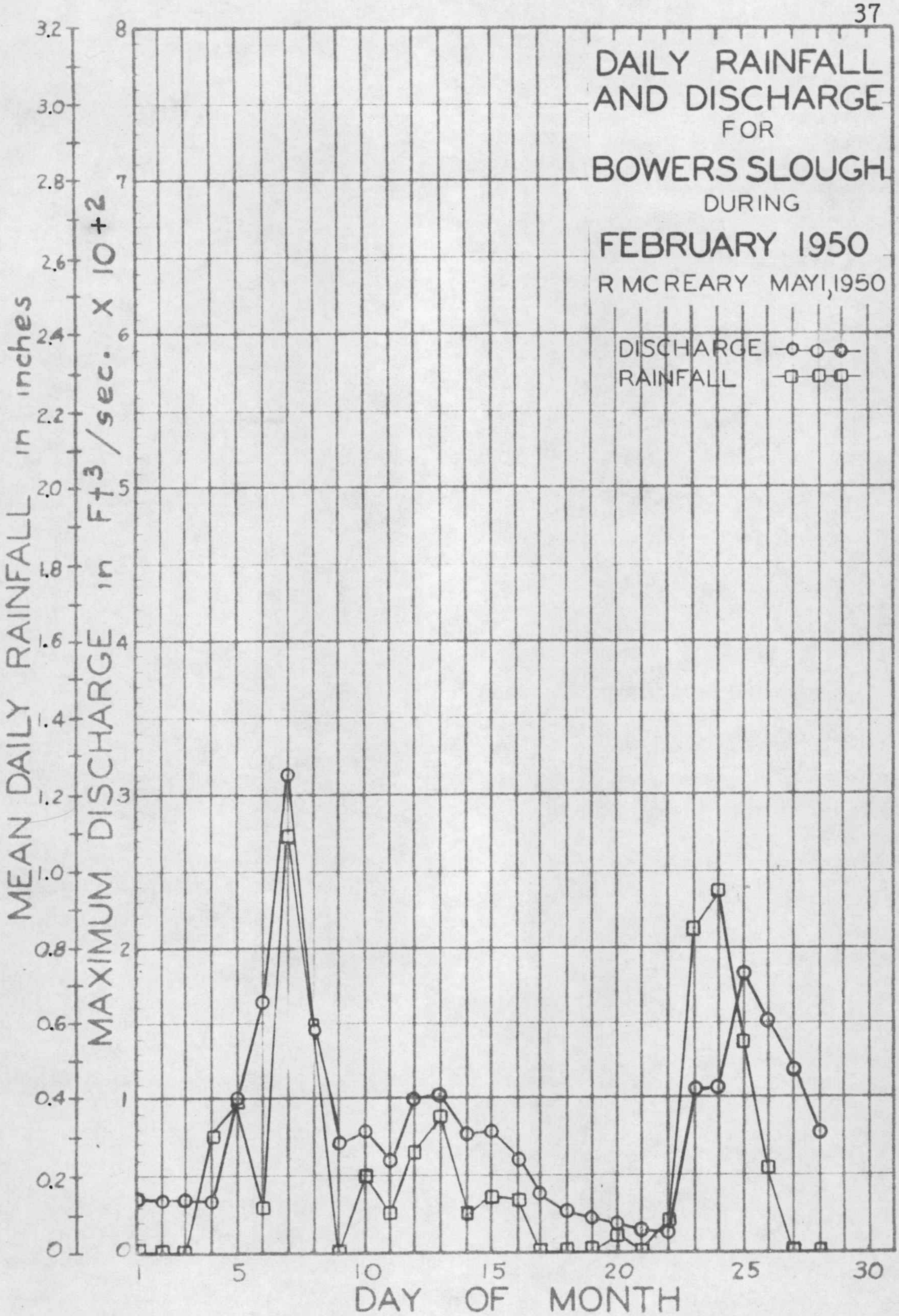


Fig. 22. Graph of rainfall and discharge for February 1950.

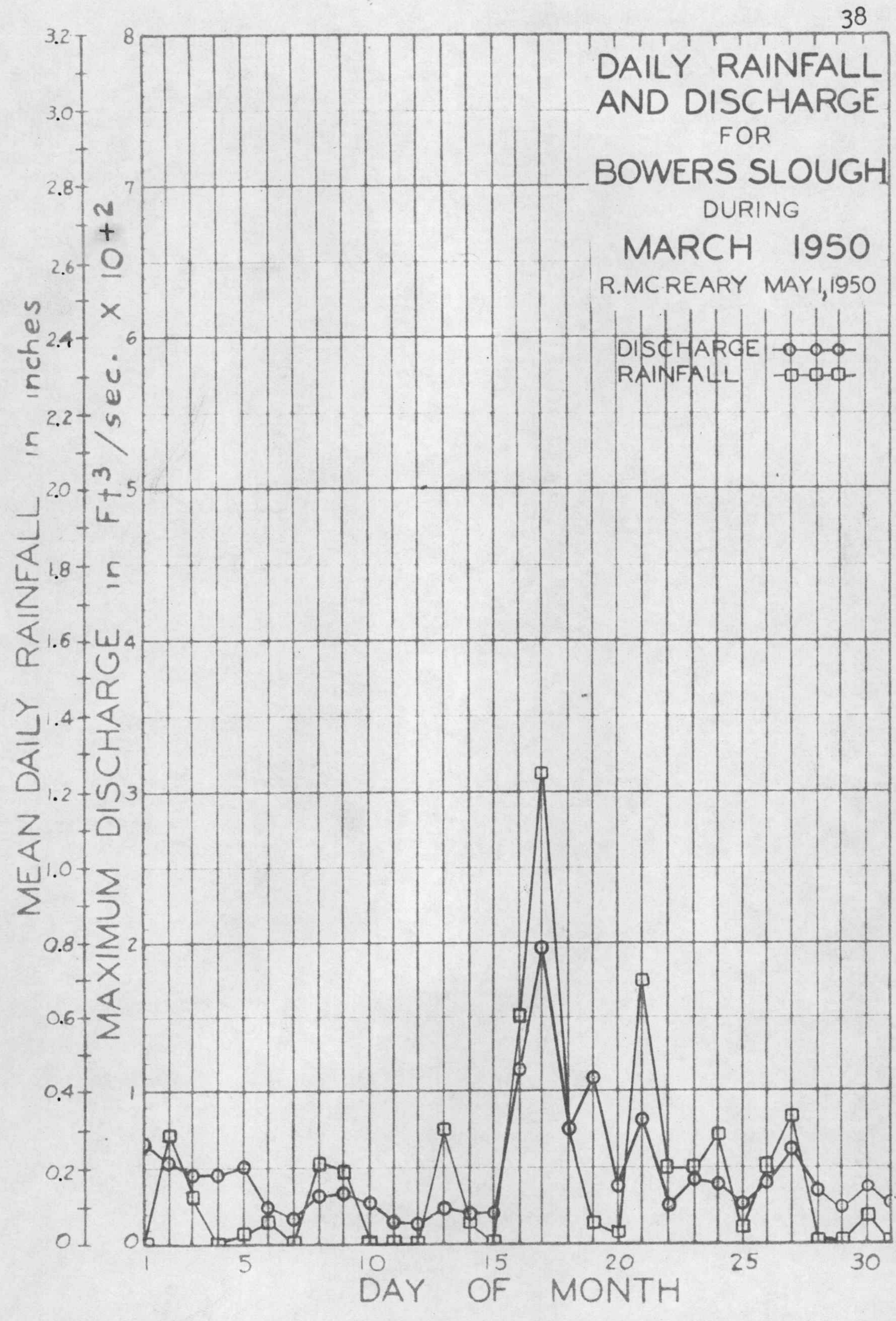


Fig. 23. Graph of rainfall and discharge for March 1950.

dry period of twenty-nine days during which 0.44 inches of rain had fallen. Rain began to fall at 8:00 a.m., April 30, and continued until late afternoon on May 2. The daily record of rainfall, Table 31, shows there was little rainfall between May 2 - May 19.

The flow of the stream at 8:00 a.m., April 30, was 1.6 cfs, at which it remained until 3:45 p.m. of the same day, when the stream began to rise. This indicated a lapse of seven hours and forty-five minutes between the time when the rain began and the time when the stream began to rise. Thirty-one hours later the flood peak of 55 cfs passed the gaging station, indicating a time of concentration of thirty-eight hours, forty-five minutes for this particular flood. Fifty-eight hours after the peak had passed, the discharge was 10.98 cfs and fifteen days elapsed before the stream returned to its discharge at the beginning of the flood.

From the rainfall and discharge data for the flood, the hydrograph of Fig. 24, page 40, was plotted on large coordinate paper. The values for the curve are given in Table 25, Appendix. Using a planimeter, the total volume of discharge above the base flow of the stream was found to be 9,120,000 cubic feet. A method suggested by Wisler and Brater (9, p. 30) was used to separate the discharge above base flow into its components of surface and groundwater runoff. The planimeter was used to find the volumes of these separate parts of the hydrograph. The surface runoff was 3,940,000 cubic feet and the groundwater runoff, 5,180,000 cubic feet.

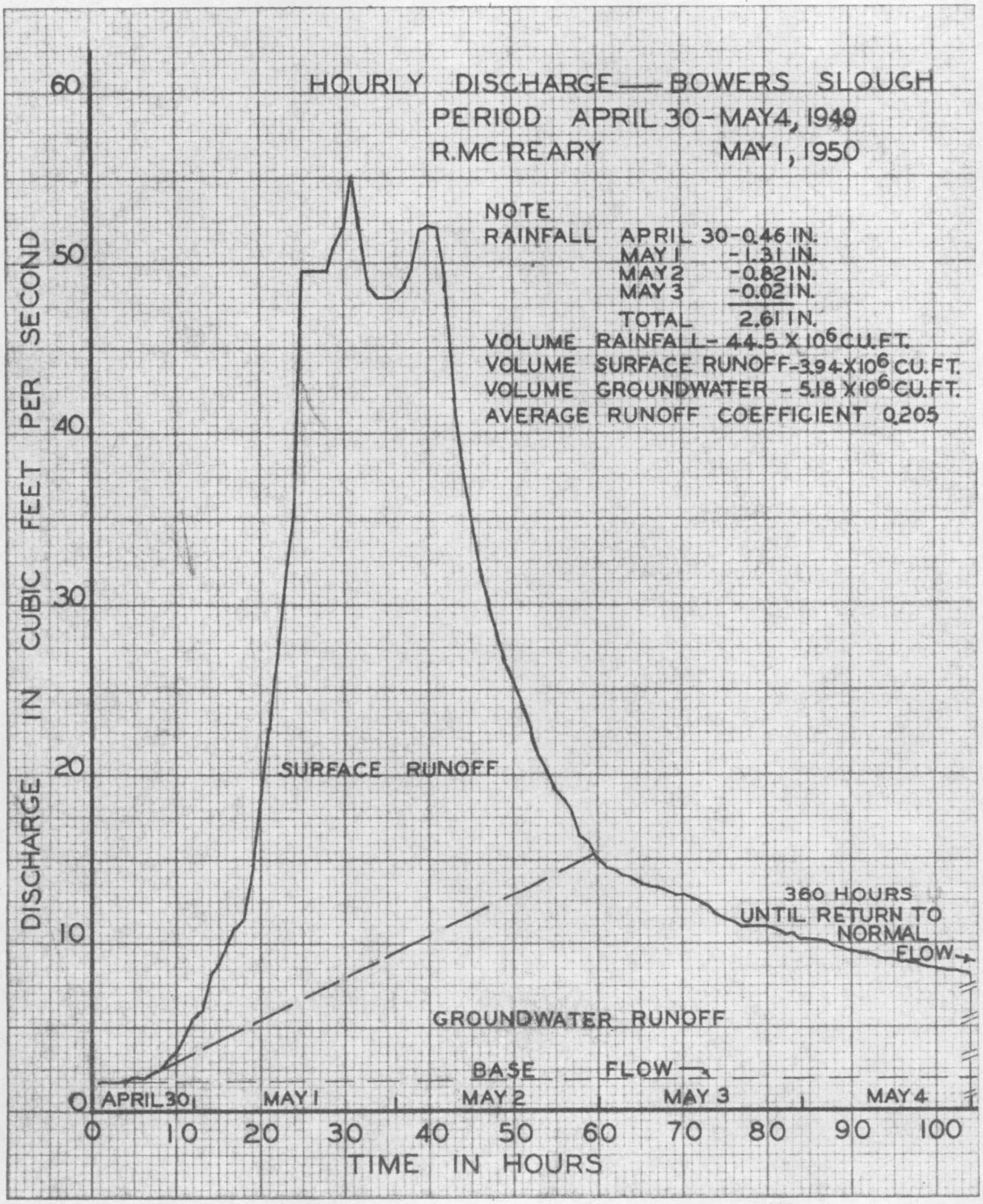


Fig. 24. Total hydrograph for flood of April 30 to May 4, 1949.

The total rainfall of 2.61 inches on the 4700 acre area produced a volume of 44,500,000 cubic feet of which only 9,120,000 cubic feet reached the stream as surface and groundwater runoff. Thus, only 20.5 per cent of the rainfall reached the stream immediately and the remaining 79.5 percent was to disappear in the form of transpiration, percolation, and evaporation losses.

The large percentage of the rainfall which did not reach the stream was to be expected, since the storm was followed by a period of warm weather, during which the evaporation losses were high. Also, the cultivated fields had a growth of about ten inches of grass, and the wooded areas had full foliage, both of which tended to hold back much of the rainfall, allowing it to evaporate. A part of the rainfall was used up in the form of transpiration, since at this time of the year the plants and trees were growing rapidly.

Flood of January 5 to 9, 1950. On January 6, 1950, a gage height corresponding to 757 cfs was recorded at the gaging station. This discharge resulted from a twenty-four hour rainfall of 2.04 inches (1.46 in. at Corvallis) on a snow cover of five inches which had accumulated during the previous four days. The hydrograph of the runoff resulting from this combination of rainfall and snow is shown in Fig. 25, page 42, and the data for the hydrograph is tabulated in Table 26 of the Appendix.

During the month of December 1949, about six inches of rain had fallen on the drainage area. The rain was rather evenly

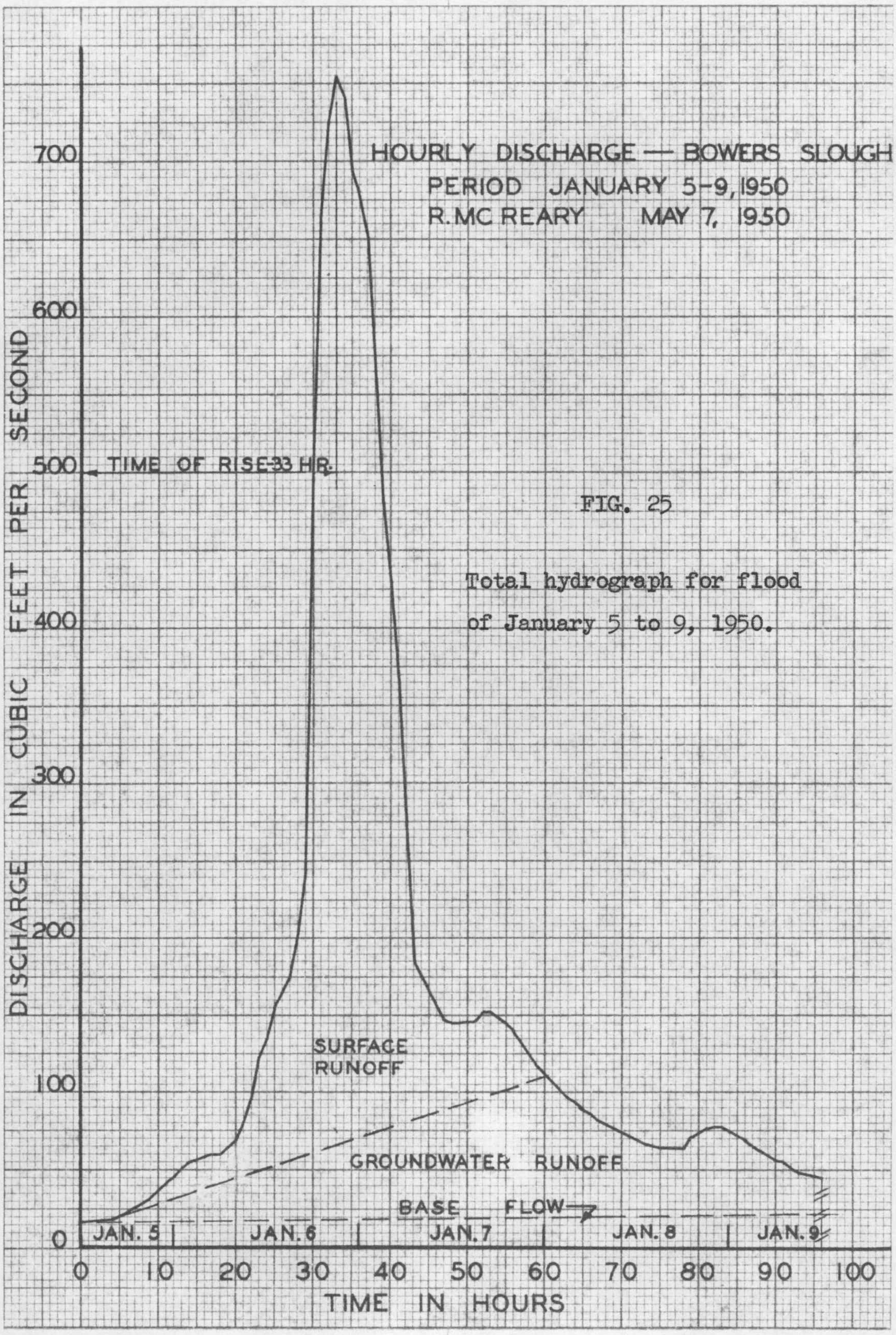


FIG. 25

Total hydrograph for flood of January 5 to 9, 1950.

distributed throughout the month, and the mean temperature was 43 degrees Fahrenheit. On January 1, 1950, three inches of snow fell, followed by two inches more on January 4. Rising temperatures on January 5 caused this snow to melt. The effect of the melting snow on the flow of the stream can be seen on the hydrograph for the flood shown in Fig. 25. It indicates that the first eighteen hours of rise were due to the snow melt, since this part of the curve rises gradually. At 3:00 p.m., January 6, an intense rain began to fall, and the total rainfall for the day was 2.04 inches. The effect of this intense rain was to increase the discharge from about 175 cfs to 757 cfs in a period of six hours; the time of concentration for this particular storm. The time required for the stream to rise from 175 cfs to 757 cfs and fall to 175 cfs was only seventeen hours. The stream flow decreased nearly as fast as it had risen, in spite of a rainfall of 1.41 inches on January 7, which did not affect the discharge appreciably. This additional rain did not have much affect because the snow had been melted by January 7, and it caused only a slight increase in discharge during the fifty-third hour of the flood.

The total volume of discharge above the base flow from January 5 to January 9 was 46,800,000 cubic feet, while the total volume of precipitation for 4.33 inches of rain was 84,700,000 cubic feet, excluding the precipitation in the form of snow on the area when the rain began. Thus, it would appear that 55 per cent of the rain which fell reached the stream, but actually, the percentage is less than this, since a portion of the discharge was due to the snow melt.

The total volume of discharge subdivided into surface and groundwater runoff was 31,100,000 cubic feet and 15,700,000 cubic feet, respectively.

Flood of January 9 to 13, 1950. A rainfall of 0.74 inches on January 9 caused a sudden increase in discharge as shown by the hydrograph of Fig. 26, page 45. The data for this hydrograph is contained in Table 27, Appendix. An additional rainfall of 2.14 inches on January 10 increased the discharge to a peak of 715 cfs. The time required for the stream to rise to its peak was eighteen hours. During a period of twelve hours, the stream rose from 200 cfs to 715 cfs and fell to 200 cfs. Again the rapid behavior of the stream was evidenced. The total discharge above the base flow of the stream from January 9 at 12:00 a.m. to January 13 at 9:00 a.m. (92 hours) was 46,800,000 cubic feet, while the total volume of rainfall was 73,200,000 cubic feet. Therefore, the total runoff was 64 per cent of the total volume of rainfall. Of the total discharge, 36,500,000 cubic feet occurred as surface runoff, and 10,300,000 cubic feet was groundwater discharge. It is to be noted that the coefficient of discharge for this flood is slightly higher than that of the previous storm. This is logical, since the surface soils were saturated by the rain and snow of January 1 to 9, enabling the rainfall to reach the stream

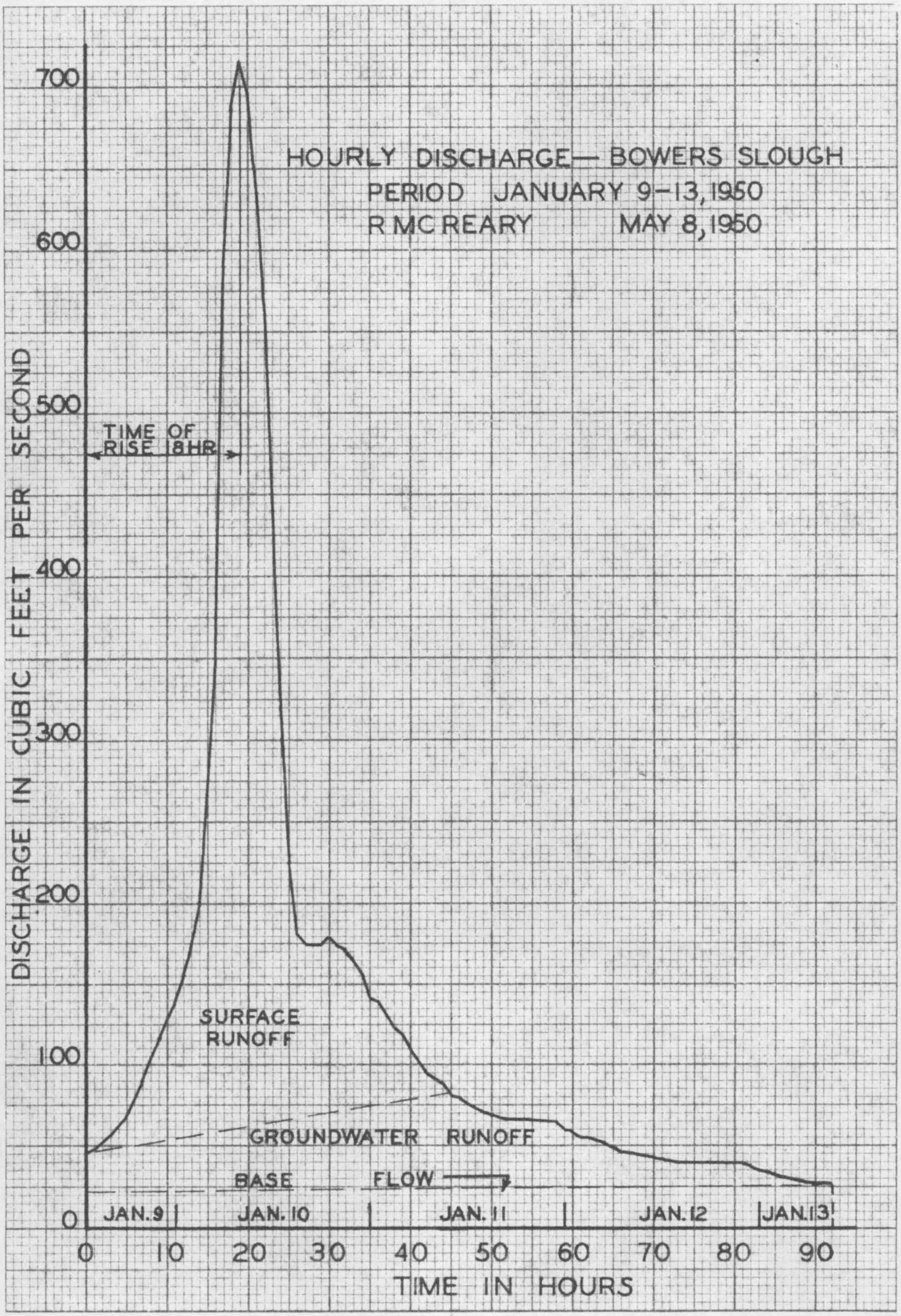


Fig. 26. Total hydrograph for flood of January 9 to 13, 1950.

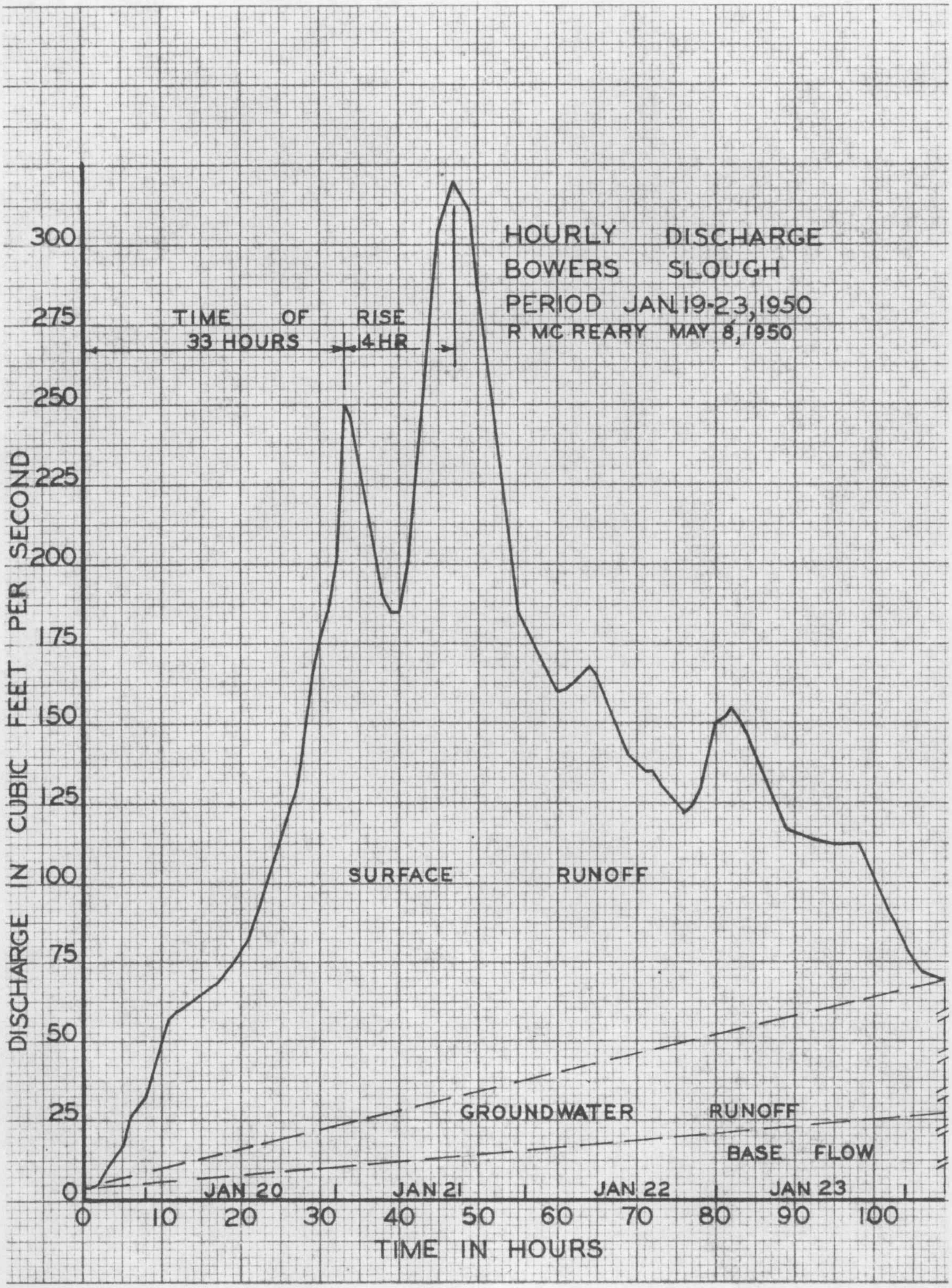


Fig. 27. Total hydrograph for flood of January 19 to 23, 1950.

more rapidly in the form of surface runoff. Had this not been the case, much of the rain would have been required to saturate the surface soil, and would have reached the stream as groundwater discharge.

Flood of January 19 to 23, 1950. The hydrograph of this flood is shown in Fig. 27, page 46. The maximum discharge was 320 cfs, less than half that of the two previous floods. It was produced by a total rainfall of 1.76 inches on January 19, 20 and 21, falling on a snow cover of one foot depth, accompanied by a sudden rise in temperature of twenty degrees Fahrenheit. The total volume of discharge was 50,800,000 cubic feet, while the volume of rainfall was 41,500,000 cubic feet. Of the total discharge 39,300,000 cubic feet occurred as surface runoff, while 11,500,000 cubic feet was groundwater runoff. The average coefficient of runoff for this flood is 1.23. The reason for the large value of the coefficient is that the major part of the runoff was formed by melting snow, and not the rainfall. The amount of this part of the runoff is unknown, since no measurements of the water content of the snow were made, due to lack of equipment. Furthermore, the depth of snow on the area was considerably greater on the higher elevations, which were not accessible at the time. Thus, it is seen that rainfall is not always the criterion in predicting the runoff from small drainage areas of the type studied, since snow cover must be considered.

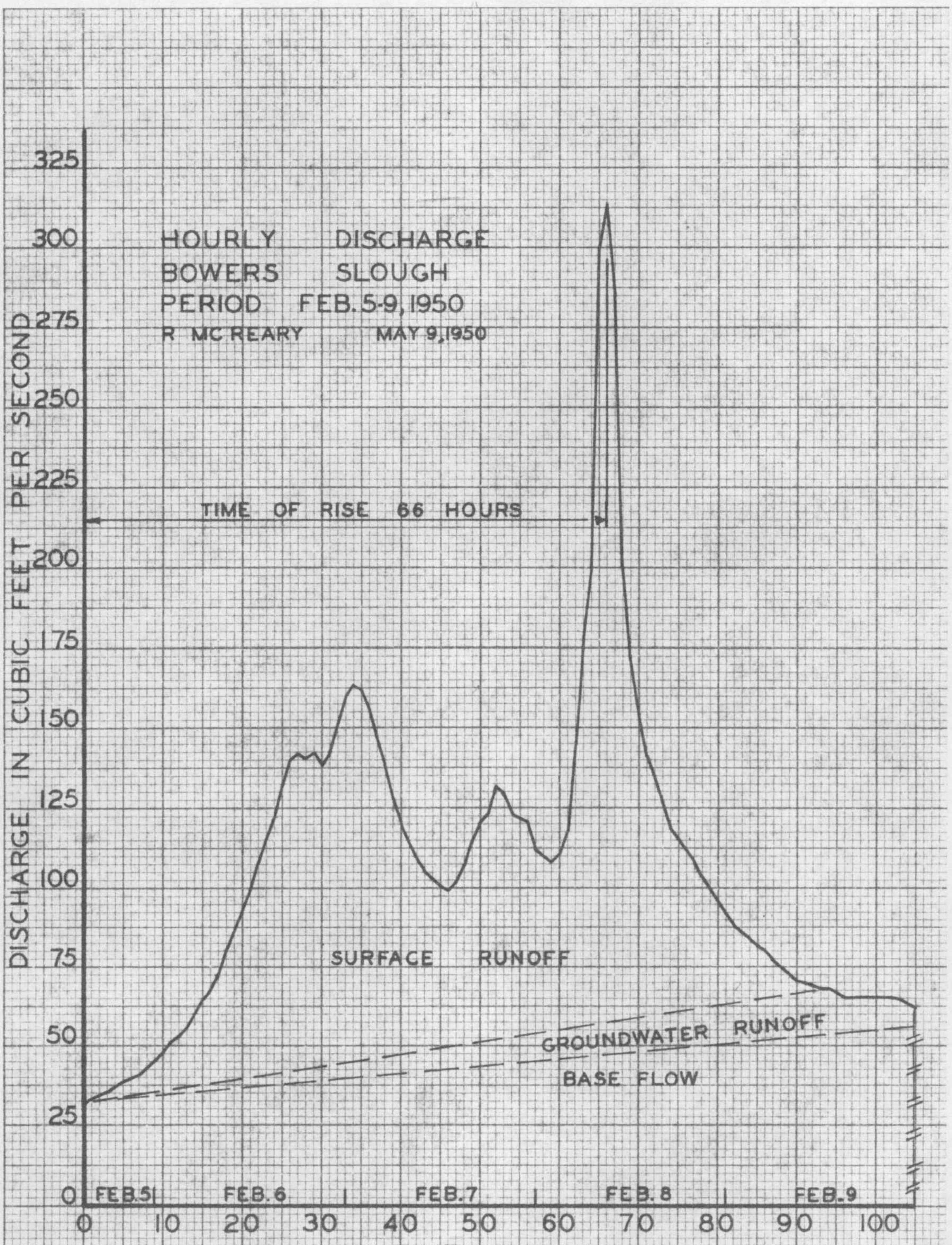


Fig. 28. Total hydrograph for flood of February 5 to 9, 1950.

Flood of February 5 to 9, 1950. The hydrograph of this flood is shown in Fig. 28, page 48. This flood occurred when 2.72 inches of rain fell on a snow cover of two to four feet, accompanied by a simultaneous increase in temperature of about twenty degrees. The total rainfall was 46,300,000 cubic feet, and the total discharge 24,300,000 cubic feet, yielding an average runoff coefficient of 0.52. The major part of the discharge was assumed to be surface runoff, due to the frozen condition of the surface soils, which prevented the rainfall from reaching the water table, through which it might have discharged as groundwater runoff. The coefficient of runoff for this storm was considerably less than that of January 19 to 23, although the conditions producing the flood were similar. The reason for this in unknown, but it may have been due to a much lower water content of the snow cover. Also, the entire snow cover was not removed during this period, due to its greater depth, whereas it was in the previous storm. This remaining snow cover undoubtedly retained some of the rain.

Flood of March 15 to 20, 1950. The last flood observed on Bowers Slough was that of March 15 to 20, 1950. The hydrograph of this flood is shown in Fig. 29, page 50. The flood was caused by a rainfall of 2.22 inches occurring on March 15, 16 and 17. On March 17, 1.23 inches of rain fell, producing the maximum discharge of 197 cfs. The total discharge was 26,500,000 cubic feet, and the volume of rainfall was 37,800,000 cubic feet, giving an average

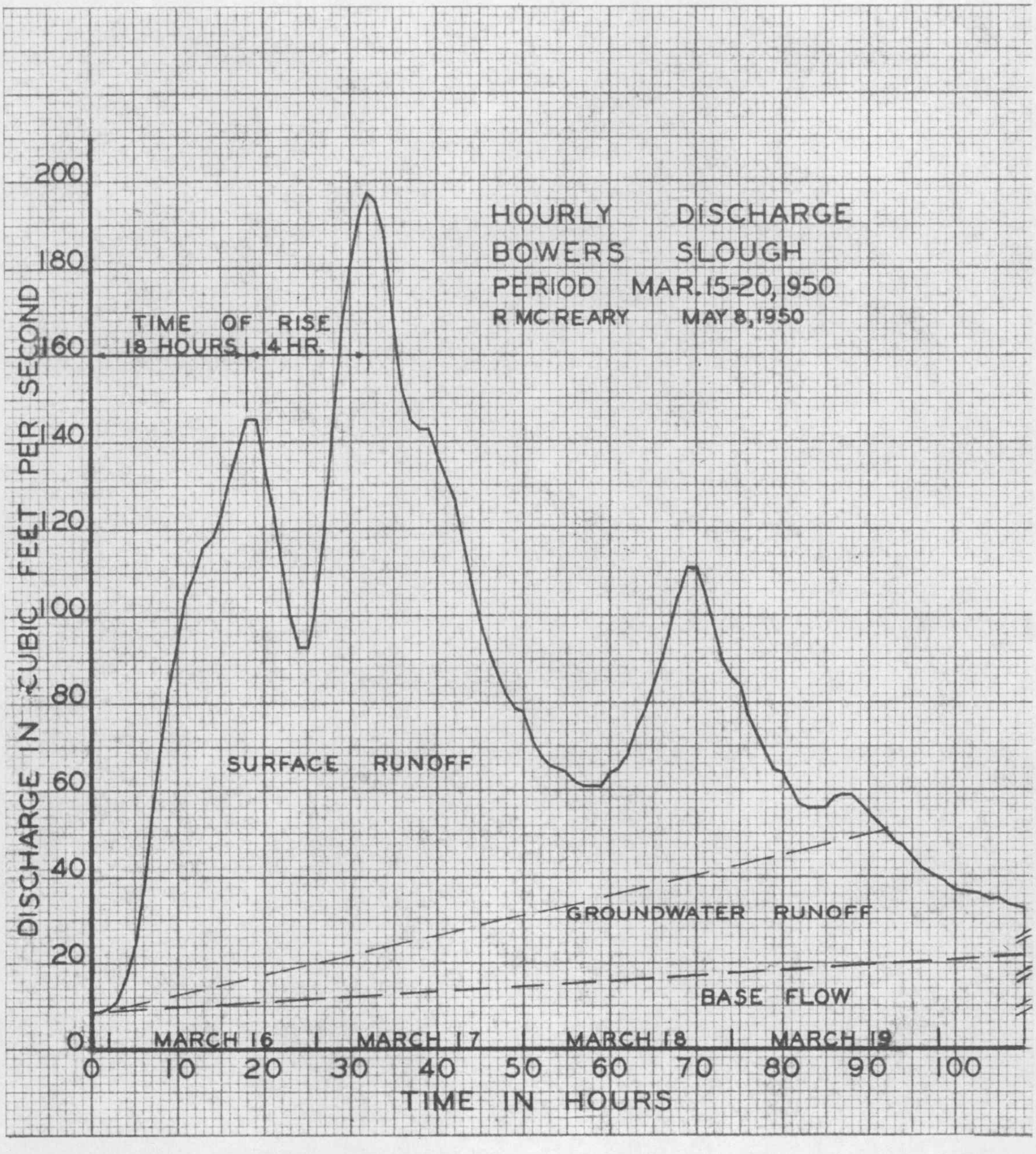


Fig. 29. Total hydrograph for flood of March 15 to 20, 1950.

runoff coefficient of 0.70. Of the total runoff, 19,500,000 cubic feet was surface runoff, and 7,000,000 cubic feet was groundwater discharge.

Greatest Discharge of Stream. The largest known discharge of the stream occurred on February 17, 1949. Unfortunately, the recording gage had not been installed at this time, so that a true hydrograph of the flood was not obtainable. However, the flood was observed during the peak hours of discharge. The discharge of the stream at its peak was determined by the current meter method. The total rainfall recorded at Corvallis on February 17 was 2.13 inches. This rain fell on a snow cover of about four inches of wet snow and resulted in a discharge of 796 cfs. It is believed that approximately this same amount of rain fell on the drainage area, since the storm was one which blanketed the Willamette Valley. At the Eugene Weather Bureau Station, thirty-four miles south of Corvallis, 2.26 inches of rain were recorded on the same day, thus indicating that the rainfall was uniform throughout the valley. Although no intensity recordings were made at Corvallis due to defective apparatus, the maximum intensity at Eugene was 0.19 inches per hour. The intensity record for February 17 at Eugene, beginning at 1:00 a.m. and continuing through 12:00 p.m. on that same day, is as follows: 0.02, 0.03, 0.04, 0.05, 0.05, 0.05, 0.08, 0.12, 0.16, 0.19, 0.16, 0.14, 0.12, 0.10, 0.08, 0.07, 0.10, 0.10, 0.11, 0.12, 0.13, 0.13, 0.06, 0.05 inches per hour. There is reason to believe that similar intensities occurred on the drainage area. Since the path of the storm on that day was in a northerly direction, it travelled directly from Eugene to the drainage area.

The ability of the stream to rise suddenly during a few hours was observed on this day as it was later on January 6 and 10, 1950. The stream rose and fell a foot at the peak discharge in a period of fifteen minutes. The discharge determination at the peak flow by the velocity area method is presented in Table 23, Appendix. According to testimony of the Portland Gas and Coke Company maintenance man, in charge of maintaining a gas line which crosses the stream at the gaging station, this flood was the largest in the past fourteen years. The reason for his certainty was that the stream came within a fraction of an inch of topping the road at the station, which had never occurred since the installation of the gas line. It is believed that this is a true statement, since it is the duty of the maintenance man to check the pipe line whenever possibility of flood occurs. His statement was further corroborated by residents living near the gaging station.

One of the problems which made the determination of the discharge of the drainage area difficult was the ponding action of the stream immediately above the gaging station during the peak of the major floods. Due to insufficient waterway area at the bridge, a reservoir was formed, and the water discharged under a head of 1.65 feet in the case of the flood of February 17, 1949. Obviously, the discharge measured at the downstream side of the bridge where the gaging station was located was not as great as that of the drainage area. Had this been the case, no reservoir action could have existed. However, the storage of the reservoir was unknown, due to the lack of a recording instrument on the upstream side of the bridge. Had one been available, the storage curve of the reservoir might have been plotted, and by combining this curve with that of the outflow curve given by the discharge at the gaging section, the true inflow curve or discharge of the drainage area could have been found.

In an attempt to determine the difference between the discharge on February 17 at the gaging station, and that of the drainage area, the upstream peak flood elevation of 217.33 was plotted on a five foot interval contour map, and the surface area of the water determined by use of a planimeter. This area was found to be 83.68 acres, or 3,645,000 square feet. It was observed that the stream remained within its banks up to an elevation of 214.70. Therefore, the depth of the flooded area at the stream was 2.63 feet. Assuming that the section of the reservoir approached that of a pyramid, the resulting volume of the flooded area would be $\frac{2.63}{3} \times 3,645,000$ or 3,195,540 cubic feet. This volume of water was accumulated during a period of three hours. The average rate of accumulation would then be the total volume divided by the intervening time, or 295.88 cfs. This discharge added to the measured gaging station discharge of 796 cfs gives the maximum drainage area discharge of 1092 cfs. However, this is only an approximate value due to the large number of unknown factors such as exact rate of rise and storage volume, but it does give an indication of the relation of the drainage area discharge to that of the gaging station.

Conclusions of the Experimental Study

The rainfall-runoff study conducted on the Bowers Slough watershed provided the following characteristics and facts:

1. That a study of many years duration is necessary to find true characteristics of a stream and watershed of the type studied.

2. Continuous records of discharge are necessary to determine the peak flows of a small stream due to its ability to rise and fall four times its normal flow during a period of less than one day.

3. The greatest discharge of a small stream in this locality will most likely occur between the months of November and March.

4. The coefficient of runoff varies for different seasons of the year, depending primarily on the amount of moisture in the surface soil, the cover of the forest, and agricultural land, and the amount and intensity of rainfall.

5. The coefficient of runoff may reach a value greater than 1.00 when the watershed has a snow cover at the time of sudden intense rainfall and rise of temperature from below to above freezing.

6. A warm rain falling on a snow cover of a small watershed will most likely produce the maximum discharge from the area.

CHAPTER IV

DETERMINATION OF WATERWAY AREA FOR STRUCTURES

General

The amount of opening required to conduct stream flow through a bridge or culvert is dependent upon the amount of expected flood flow and the permissible velocity through the opening. In determining the amount of flood flow, it is necessary to rely upon direct stream flow measurements, or the existence of high water marks near the structure site, along with existing runoff formulas.

The expense and difficulties involved in stream flow measurement preclude its general use for small streams. In addition, so as to be of most value, stream gaging should be accompanied by an elaborate and well distributed system of rainfall records. This may also prove to be expensive and unfeasible in many cases.

Obviously, when high water marks or formulas are used as criteria, the judgement of the individual is a very important factor. However, the impractibility of stream gaging often makes it necessary to rely upon runoff formulas and high water marks for determination of maximum flow.

The permissible velocity through the opening is a matter of choice, depending on the type of soil and foundation conditions prevailing at the site, and whether or not it is desired to maintain the flow with no appreciable reservoir action upstream from the site.

Dangers of Insufficient Waterway Area

If a culvert or bridge is not provided with sufficient waterway area to conduct peak flood flows, one or more of the following may occur:

1. Backwater may become high enough to cause damage to adjacent property by flooding cultivated areas and destroying crops, or covering adjacent railroad tracks and highways.
2. The lateral restriction of the stream may induce currents parallel to the approach embankments of such magnitude as to render it necessary to riprap or otherwise protect the slopes.
3. The increased currents induced by the restriction may cause erosion of stream banks or bed.
4. Excessive current velocity through the opening may erode and undermine footings of the structure.
5. Drift and debris or ice floes may endanger the superstructure when the flow is under a head or when the water surface approaches the clearance elevation.

Obviously, the determination of the proper proportions of waterway openings must be made with these dangers in mind. The losses which will occur should one or more of the above take place should be balanced against the cost necessary to prevent such losses. If only flooding of land is involved, it is not in many cases necessary to provide for maximum flood flows. However, if danger to life is apparent, provisions for the maximum flood flow must be made.

Factors Affecting Runoff

The maximum flood flow which will occur is dependent upon the amount of runoff from a given amount of precipitation and its concentration in the stream. A portion of the rainfall is absorbed by the soil and cover, a small amount evaporates, and the remainder runs off. Much has been written on the effect of forestation upon runoff, and one can find expressions of opinion to substantiate the contention that it will increase or decrease it. However, it seems reasonable that with a rainfall of given intensity falling on a forested area, the concentration of the runoff will be at a slower rate than it would be if the cover were removed.

The type of surface within the drainage area has much to do with the amount and rate of runoff. Varying degrees of absorptiveness will be found from the gravelly and sandy soils, through the loams and loose soils to the more impervious clays and rocky formations. That part of the rainfall absorbed will of course be greatest on the sandy and gravelly soils and least on the stony, rock surfaces. The surface may be covered with vegetation, which will tend to absorb and retard the movement of the water or it may be barren with this tendency lacking. The effect of absorption on the runoff is dependent to some extent upon the area and length of the storm. The character of surface may be uniform for a small area, but for larger areas it is likely to be a composite of several types. As the effect must be evaluated by judgement, the more thorough the knowledge of the soil conditions, the better the evaluation.

Topography is an important factor since the rate of concentration is largely dependent upon it. Here again it is necessary to have full knowledge of the area and to take into account the variation of the topography over the various parts of the area. It is easy to be misled by rough terrain along a stream, whereas a major part of the drainage area may be of much lighter classification, or the reverse may be true. To supplement impressions of topography obtained in the field there are available topographic maps from various government agencies for most sections of the country. In the selection of quantitative factors for the varying classifications of topography a standard for comparison must first be established. For a given set of conditions, knowing or assuming the adaptability of a runoff formula for one classification of topography, ratio factors can be applied to adapt its use to other classifications. Classifying into mountainous, hilly, rolling, and flat does not mean much without a further basis of measure as these terms are relative only. There appears to be a tendency for individuals to classify terrain according to the character of the region with which they are most familiar.

Another important factor, and one which is often neglected, is the shape of the drainage area. Generally speaking, a fan shaped or somewhat circular area in which the water from all parts of it reach a given point on the stream at about the same time gives the maximum concentration. On the other hand, the long narrow areas give the minimum concentration because the water in the lower part of the area has passed on before that of the upper part has reached the point. However, it should be remembered that storms are always moving and not stationary. If the direction of movement of the storm is the same as that of the stream flow, it may bring about a severe increase in the concentration of the flood. This is particularly true in the case of the long narrow areas.

Storage, which may be in the form of natural or artificial lakes and ponds, has a very important effect upon the amount of runoff. It affects both the time required to reach the maximum rate of runoff and the percentage of the rainfall flowing off at any given time. It delays the peak flow, lengthens its duration, and tends to decrease the intensity of the critical rainfall. This factor may, in some cases, reduce the runoff by as much as fifty per cent or more. The wide, flat type valley will be affected the most, while the narrow V-shaped valleys will show little reduction.

Runoff Formulas

A formula for the determination of runoff, to be all inclusive, should in some manner take into account the factors hereinbefore mentioned. The wide variation in these factors and the numerous combinations possible make it difficult to derive one formula or method of predicting runoff or waterway area for general use.

However, during the past century, many methods have been proposed for use in different areas throughout the United States and other countries. These methods have been applied to a multitude of areas with varying results, particularly in the case of small drainage areas. The exact number of these methods and formulas which have been proposed is unknown, but research done in connection with this thesis indicates that there are about thirty. They are to be found in numerous texts, bulletins and articles, but so far as is known no complete list has ever been published.

In order that such a list might be obtained, a large number of references were consulted and a letter of inquiry concerning methods used to predict runoff and waterway area was sent to each of the forty-eight state highway departments throughout the country. This survey indicated that there were at least ten different methods in use by the twenty-five states replying to the questionnaire. An additional fourteen methods were found by research. Table 2, page 61, shows the methods used by various highway departments as a guide in predicting the required waterway area at a given bridge site where no discharge records are available.

Table 2

Methods Used by State Highway Departments
for Prediction of Waterway Areas Required for Runoff from Small Watersheds

State	Talbot Formula	Rational Method	Kuichling Formula	Adolph Meyer Formula	Burkli-Ziegler Formula	Duns Tables	Drainage Tables	Empirical Stream Data	Own Method
Arizona	X						X		
California									X
Connecticut									X
Delaware	X								
Georgia	X							X	
Illinois	X								
Iowa	X								
Kansas	X	X			X				
Kentucky	X								X
Maryland		X							
Massachusetts									X
Michigan									X
Minnesota				X					
Missouri	X	X	X						
Nebraska	X				X	X			
New Jersey	X							X	X
New Mexico	X								
North Dakota	X					X			
Ohio								X	
Oregon	X								
Pennsylvania									X
South Dakota							X		
Tennessee	X								
Texas		X							X
Wisconsin		X						X	

Application of Runoff Formulas to Bowers Slough Drainage Area

Twenty of the most commonly used methods consisting of runoff and waterway area formulas will be applied to the area studied. The theoretical discharges yielded by the formulas will be compared with a probable maximum twenty-five year discharge of Bowers Slough.

Nomenclature

A = waterway area in square feet (sq. ft.) required to safely conduct peak flows

M = area in acres (ac.) drained by the stream above the point on the stream under consideration

D = area in square miles (sq. mi.) drained by the stream above the point under consideration

W = mean width of drainage area in miles (mi.)

L = mean length of drainage area in miles (mi.)

Q = discharge of stream in cubic feet per second (cfs)

S = average surface slope of drainage area in feet per thousand (fpt)

C,K = coefficients (values given with each formula)

The following values have been determined for the experimental area, and will be used in the formulas to compute the discharge or required waterway area.

M = 4700 ac.

D = 7.34 sq. mi.

W = 1.7 mi.

L = 3.7 mi.

S = 48.5 fpt

Talbots Formula (5, p. 7). In 1887, Professor A. N. Talbot proposed what is now one of the most widely used waterway area formulas. It was developed primarily for drainage areas of less than seventy-seven square miles, and is seldom used on larger areas.

His formula is $A = CM^{3/4}$, in which C is a coefficient depending on the topography. The following values of C are commonly used:

Topography	Value of C
Mountainous	1.00
Very Hilly	0.66
Hilly	0.50
Rolling	0.33
Gently Rolling	0.25
Flat	0.20

So as to apply this formula to the experimental watershed, a value of C = 0.50 is selected, since the topography of the area is best described as hilly. Placing this value of C in the Talbot formula, we have,

$$(1) \qquad A = CM^{3/4}$$

$$A = 0.5\ (4700)^{3/4}$$

$$A = 284 \text{ sq. ft.}$$

Assuming a velocity through the opening of ten feet per second (the maximum measured velocity was 9.41 feet per second), and since the discharge is the product of the area of the opening and the velocity through it, we have,

$$(2) \qquad Q = AV$$

$$Q = 284 \times 10$$

$$Q = 2840 \text{ cfs}$$

This is considerably larger than the maximum measured discharge of 796 cfs. As stated previously, this maximum discharge was that of the gaging station, while the actual drainage area discharge was somewhat greater, approximately 1100 cfs. However, this assumed maximum of 1100 cfs is that for only a two year period of observation. Actually, a structure such as a culvert or small bridge is usually designed for a twenty-five year frequency, i.e., for the maximum flood which will occur once in twenty-five years. There is little reason to believe that Bowers Slough will discharge more than 1500 cfs once every twenty-five years, because of the prevailing weather conditions in this region.

So as to determine the proper twenty-five year coefficient C for the experimental watershed, the twenty-five year maximum of 1500 cfs and an assumed velocity through the opening of 10 fps are placed in equation (2). Solving for the area, we have

$$Q = AV$$

$$1500 = A \times 10$$

$$A = 150 \text{ sq. ft.}$$

This value for A is now placed in equation (1), from which C is determined as follows:

$$A = CM^{3/4}$$

$$150 = C\,(4700)^{3/4}$$

$$C = 0.26$$

Thus it is seen that the normal table of values for the coefficient C is in general too high for areas similar to that of Bowers Slough, and the coefficient should be decreased from ten to fifty per cent, depending on the loss which would be incurred if failure of the structure should occur.

Adolph Meyer Formula (7, p. 369). The Meyer formula, $Q = 100\ D^{0.6}\ C_f C_r$, was developed for the state of Minnesota, and is intended to be used only in that state. However, some engineers have applied it with varying degrees of success to areas in other regions throughout the country.

In the formula, Q is the discharge to be expected once in 10, 25, or 100 years, depending on the desired design frequency. The values of C_f, the coefficient of frequency, and C_r, the runoff coefficient, are given in Table 3, below, and Table 4, page 66.

Table 3

Frequency Coefficient C_f

For a flood of magnitude to be expected	Coefficient
Once in 10 years	0.85
Once in 25 years	1.00
Once in 100 years	1.25

Table 4

Values of Runoff Coefficient C_r in Adolph Meyer Formula

Character of Drainage Basin	Coefficients Sandy Soil	Loam	Clayey Soil
1. Very flat agricultural or timber land with some marshes and swamps . .	0.35	0.40	0.50
2. Relatively flat agricultural or timber land with some marshes and ponds . .	0.45	0.50	0.60
3. Gently rolling agricultural or timber land full of lakes, ponds, and marshes connected by poorly defined water courses	0.50	0.60	0.75
4. Relatively flat agricultural or timber land of fairly uniform slope, without lakes and ponds	0.60	0.70	0.85
5. Slightly undulating agricultural or timber land without lakes or ponds; or distinctly rolling, hilly agricultural or timber land, with lakes and ponds .	0.70	0.80	1.00
6. Gently rolling agricultural or timber land without lakes and ponds	0.85	1.00	1.25
7. Distinctly rolling, hilly agricultural or timber land without lakes and ponds; or hilly agricultural or timber lands with steep slopes and lakes, ponds and marshes in valleys	1.10	1.50	2.00
8. Hilly agricultural or timber land with steep slopes barely admitting of cultivation; without lakes, ponds or marshes	2.25	3.00	4.00
9. Very hilly timber or brush-covered land, slopes too steep for cultivation, ravines and gullies with occasional small ponds or marshes	3.50	4.50	6.00
10. Very hilly timber or brush covered land with some rock outcropping; ravines and gullies and occasional small ponds or marshes	5.00	6.00	8.00
11. Very hilly to rugged country with much rock outcropping; scattered timber; occasional small ponds and marshes .	9.00	10.00	12.00
12. Rugged to precipitous rocky country with practically no soil cover; small timber and brush; ravines and gullies, no lakes, ponds, or marshes to retard runoff	--	15.00	--

For the experimental area, the frequency coefficient is taken as 1.00 corresponding to a recurrence interval of twenty-five years. A runoff coefficient of 3.00 will be used since the soil on the area is largely loam and the description under item No. 8 of Table 4, page 66, most nearly matches that of the drainage area. Placing these values in the formula, we have

$$Q = 100\ D^{0.6} C_f C_r$$

$$Q = 100\ (7.34)^{0.6}\ (100)(3.00)$$

$$Q = 993 \text{ cfs}$$

This is lower than a safe value for the discharge to be expected once in twenty-five years at the gaging station site on Bowers Slough. However, it is believed that this formula could be used on areas of this type if the runoff coefficients were to be adjusted to conditions in this region.

Myers Formula (8, p. 5). The formula developed by Major E. T. W. Myers was probably the first runoff formula to be used extensively in this country for determining waterway areas. It is $A = CM^{1/2}$, in which C is a coefficient to be varied with topographical conditions. This coefficient is to be taken as 1 in ordinary, slightly rolling agricultural land, 1.5 in hilly land, and 4 in rocky mountainous land.

An intermediate value of C = 2.5 is selected for the experimental area, since the topography of the area is partly hilly and partly mountainous.

Using this value of C, the waterway area is:

$$A = CM^{1/2}$$

$$A = (2.5)(4700)^{1/2}$$

$$A = 171 \text{ sq. ft.}$$

Assuming a velocity through the opening of ten feet per second, it would carry a discharge of 1710 cfs, a reasonable value for the expected discharge. It is believed that this formula would be quite reliable for similar small drainage areas if the proper value of C was determined for a particular region.

Wentworth Formula (8, p. 9). In connection with his work on the Norfolk and Western Railway, Wentworth developed the formula $A = M^{2/3}$. It was derived for use on areas along the rail line in southeastern United States.

Applying this formula to the experimental area, we have

$$A = M^{2/3}$$

$$A = (4700)^{2/3}$$

$$A = 289 \text{ sq. ft.}$$

As before, assuming a velocity through the opening of ten feet per second, the discharge would be 2890 cfs. Again, as in the majority of existing runoff formulas, the Wentworth Formula yields a value for discharge which is too high. However, this formula would be usable if a coefficient were to be added to it, and the values of the coefficient found experimentally.

Fanning Formula (10, p. 1114). J. T. Fanning has proposed the formula $Q = 200\ M^{5/6}$, for use on streams in the New England and

Applying this formula to the experimental area, we have

$$Q = 200\ M^{5/6}$$

$$Q = 200\ (4700)^{5/6}$$

$$Q = 1052\ \text{cfs}$$

This is regarded as being slightly low for the maximum flood on the stream studied, and the constant term of the formula must be increased before it can be used on similar areas.

Dickens Formula (6, p. 660). Dickens proposed the formula $Q = 500\ M^{3/4}$, for the Central Provinces of India.

Solving for the discharge by use of this formula, we have

$$Q = 500\ M^{3/4}$$

$$Q = 500\ (4700)^{3/4}$$

$$Q = 2230\ \text{cfs}$$

This is not an unreasonable discharge to be expected from the area studied. However, the equation as it is should not be used in any given region without first regulating or adjusting the coefficient for different types of areas.

Tidewater Railway Formula (10, p. 1113). A formula used for many years by the Tidewater Railway for predicting the waterway area for bridges is $A = 0.62\ M^{0.7}$.

Applying this to the experimental watershed, the area is

$$A = 0.62\ M^{0.7}$$

$$A = 0.62\ (4700)^{0.7}$$

$$A = 231\ \text{sq. ft.}$$

If a velocity through the opening is assumed to be ten feet per second, the discharge would be 2310 cfs. Again, this is a reasonable discharge, but the coefficient would have to be checked before this equation could be used in any given region. With proper modification of the coefficient, it would no doubt be useful for small areas.

Peck Formula (10, p. 1113). The Peck formula is $A = \frac{M}{C}$, where C is a coefficient varying from 4 to 6. If the maximum value of the coefficient C is selected for the experimental watershed, the formula yields an area of 4700/6 or 783 square feet. If the minimum value of C = 4 is used the area required is 1175 square feet. If in both cases a velocity of ten feet per second were assumed, the discharges would be respectively, 7830 and 11,750 cfs, or from ten to fifteen times as great as any discharge recorded for the stream. Both of these values are unreasonably high, and would result in extreme overdesign for areas similar to the one observed in this project.

The Rational Formula (8, p. 9). One of the oldest and best known formulas is the so-called Rational formula. It is based on the assumption that the maximum rate of flow from a certain average rainfall intensity on the drainage area is produced by that rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. This is the time required for the surface runoff from the most remote part of the drainage system to reach the point under consideration. When this runoff reaches the point on the stream under consideration, the peak of the flood will most likely occur. Therefore, the time required for the flood to crest will be the time of concentration. The magnitude of flood being considered is taken into account in the selection of the intensity of the rainfall. These intensities for most parts of the United States have been tabulated in D. L. Yarnell's , "Intensity Frequency Data". (11, pp. 1-68).

This method assumes that greater intensities over periods shorter than the time of concentration, and lesser intensities over periods longer than the time of concentration would not produce a flood crest of greater magnitude than that for the critical period or time of concentration. In the case of the former, only part of the drainage area would be contributing to the flood crest, and for the latter, earlier parts of the rainfall would have passed the point of observation.

The form of the formula is Q = CIM, in which C is a coefficient representing the percentage of average rainfall appearing as runoff at the end of the time of concentration at the point of observation and I is the average rainfall intensity in inches per hour prevailing during the time of concentration. A complete table of values of C is given on page 72 (3, p. 31).

In the formula Q = CIM, then, the value of C is selected from Table 5 as 0.45, since the average slope of the experimental watershed is 4.85 per cent, and the major part of the soil in the

Table 5

Values of C in the Formula Q = CIM

SLOPE	LAND USE	CLASSIFICATION OF SOIL					
		Rolling Plains		Sandy or Sandy Loam soils (pervious)		Black or Loessial soils (impervious	
		Min.	Max.	Min.	Max.	Min.	Max.
Flat (0% to 1%)	Timber			0.15	0.20	0.15	0.20
	Pasture			0.20	0.25	0.25	0.30
	Cultivated			0.25	0.35	0.30	0.40
Rolling (1% to 3.5%)	Timber	0.25	0.30	0.15	0.20	0.18	0.25
	Pasture	0.40	0.45	0.30	0.40	0.35	0.45
	Cultivated			0.45	0.65	0.50	0.70
Hilly 3.5% to 5.5%)	Timber			0.20	0.25	0.25	0.30
	Pasture			0.35	0.45	0.45	0.55
	Cultivated			0.60	0.75	0.70	0.85
Mountainous (5.5%	Timber					0.70	0.80
	Bare					0.80	0.90

watershed is silty clay loam of slightly impervious nature. The value of I is taken as 0.50 inches per hour, since this intensity must be of a duration equal to the time of concentration of the stream above the point under consideration. The maximum intensity to be expected once in twenty-five years in this region as given by Yarnell is 1.00 inches per hour, which would hardly last throughout the concentration period of about five hours. Actually, the maximum intensity recorded by the U. S. Weather Bureau Station at Eugene, Oregon during the two year observation period was only 0.19 inches per hour. Therefore, it is believed that an intensity of 0.50 inches per hour is sufficient.

Placing these values in the equation, we have

$$Q = CIM$$

$$Q = 0.45 \times 0.50 \times 4700$$

$$Q = 1059 \text{ cfs}$$

This is considered a conservative value for the discharge to be expected at the gaging station, since the maximum recorded was 796 cfs, and the probable actual watershed discharge at the time this discharge was measured was about 1100 cfs.

Thus, it is seen that with the proper selection of the coefficient C and the expected rainfall intensity, a satisfactory solution of the maximum discharge can be obtained by use of the Rational Formula.

<u>Burkli-Ziegler Formula</u> (8, p. 6). The Burkli-Ziegler formula was introduced into this country in 1881 by Rudolph Hering. It is based on several heavy storms in Zurich, Switzerland, and was developed in connection with storm sewer design. However, it has been used in this country to some extent for predicting runoff from small drainage areas.

It is the form $Q = MCI\ (S/M)^{1/4}$, in which I is the maximum rate of rainfall in inches per hour, and C is a coefficient having a value of 0.2 for all areas other than residential areas. I is taken as 1.00 inch per hour according to the Yarnell rainfall data.

Substituting these values in the equation, we have

$$Q = MCI\ (S/M)^{1/4}$$

$$Q = 4700 \times 0.2 \times 1.0\ (48.45/4700)^{1/4}$$

$$Q = 3000 \text{ cfs}$$

This is obviously a high value for the drainage area discharge to be expected, but it is on the safe side and would result in overdesign of the waterway area. It is believed that this formula could be applied to small areas if more exact values of the coefficient were obtained experimentally for a given locality.

<u>Craig Formula</u> (8, p. 8). One of the first runoff formulas was that proposed by Craig in 1868 and was based on his observations in India. It takes into account the length and width of the valley as follows:

$$Q = 440\ WC \text{ hyperbolic log } \frac{8L}{W}^{2}$$

The factor C varies from 0.68 to 1.95, depending on surrounding conditions.

For the drainage area studied, a value of C is taken as 1.00. Applying this value to the equation, we have

$$Q = 440 \text{ WC hyperbolic log } \frac{8L}{W}^2$$

$$Q = 440\ (1.7)(1.0) \log 8 \left(\frac{3.7}{1.7}\right)^2$$

$$Q = 3115 \text{ cfs}$$

This is a safe value of discharge for the area studied, but the use of this formula on similar areas would indeed be limited, because of the large number of variables embodied in the factor C. However, it is presumed that this factor could be tied down by applying the formula to several drainage areas for which the maximum discharge is known and comparing these discharges to the theoretical discharges computed by the formula without the factor. In this respect, it possesses little if any advantage over the more simple forms of runoff formulas.

Pettis Formula (8, p. 11). Colonel C. R. Pettis developed the so-called "width" formula in 1927. It has been designed as such because the only characteristic of the drainage area taken into account directly by the formula is the width. It is $Q = C\ (PW)^{5/4}$, in which C is a coefficient which varies for different sections of the country and P is the depth of rainfall in inches effective in producing the crest discharge.

The values of C are as follows:

East of the Mississippi River and the Pacific Coast . . .	310
Arid regions of the Southwest	40
Semi-arid regions of the Rocky Mountains	200

The depth of rainfall may be computed from the following formula:

$$P = KT^{0.23} (0.9956) D^{1/2}$$

K is the maximum theoretical precipitation in inches for one hour on one square mile in a typical storm and T is the length of time in hours that it takes for all of the effective water from the peak of precipitation to pass the point in question.

Assuming K = 1.0 inches per hour for this locality, T = 5 hours as observed, the value of P as computed is 3.91. Substituting this value in the original equation, we have

$$Q = C\ (PW)^{5/4}$$

$$Q = 310\ (3.91 \times 1.70)^{5/4}$$

$$Q = 3300 \text{ cfs}$$

This discharge is likewise larger than would be expected for the area studied. Due to the number of factors which are included in this method, it is highly probable that by adjusting the coefficient C, it could be used successfully for a particular region. The Missouri State Highway Department has used this formula on several hundred crossings and obtained excellent results with it.

<u>McMath Formula</u> (10, p. 1115). The McMath formula has been used extensively for determining expected discharges, particularly by railroad bridge engineers. It is $Q = CVS^{1/5}M^{4/5}$, in which C is the proportion of the rainfall which reaches the stream and V is the volume of water in cubic feet falling on an acre of

surface per second during the period of greatest intensity of rain.

Assuming that C has a value of 0.6 for the area, and V corresponding to an intensity of 1.00 inch per hour is 1.008, the discharge is

$$Q = CV\,S^{1/5}\,M^{4/5}$$

$$Q = (0.6)(1.008)(48.5)^{1/5}(4700)^{4/5}$$

$$Q = 2940 \text{ cfs}$$

In view of the fact that C was taken as only 0.6 in computing the discharge, it is believed that this formula will give values which are too large for areas similar to that studied, since the values of C have been found to be as high as 1.23 in the case of the flood on Bowers Slough of January 19 to 23, 1950.

Burge Formula (6, p. 660). This formula is sometimes referred to as the Dredge Formula. It was used for many years with much success on the Madras Railway line in India. It takes into account the length of drainage area in the following manner:

$$Q = 1300\,\frac{D}{L^{2/3}}$$

Applying this to the experimental area, we have

$$Q = 1300\,\frac{(7.34)}{(3.70)^{2/3}}$$

$$Q = 3970 \text{ cfs}$$

Obviously, this is considerably larger than would be expected to occur in Bowers Slough. However, this formula could be used in a given region if the coefficient and the power of the length were to be adjusted to fit local conditions.

<u>Murphy Formula</u> (6, p. 660). E. C. Murphy developed this formula for streams in the northeastern part of the United States. His formula is

$$Q = \left(\frac{46790}{D + 320} + 15\right) D$$

Applying this to the experimental area, we have

$$Q = \left(\frac{46790}{7.34 + 320} + 15\right) 7.34$$

$$Q = 1160 \text{ cfs}$$

This is slightly low for the maximum discharge to be expected for Bowers Slough. However, this formula has definite possibilities for application to similar small areas. Certainly, the constants could be adjusted to fit areas in this region.

<u>C. B. & Q. Railroad Formula</u> (10, p. 1115). For use in the design of culverts and bridge waterway areas, engineers of the C. B. & Q. Railroad proposed in 1906 the formula:

$$Q = \frac{3000\ D}{3 + 2D^{1/2}}$$

Applying this to the watershed studied, we have

$$Q = \frac{3000 \times 7.34}{3 + 2(7.34)^{1/2}}$$

$$Q = 2620 \text{ cfs}$$

Although this is slightly high for the expected discharge, it is reasonable that this formula likewise could be applicable to areas of the type studied if the constants were properly modified.

Kuichling Formula (8, p. 29). Emil Kuichling has developed many formulas for runoff. These formulas have been derived according to data obtained on various American and European Rivers. The formula which he proposed for areas of less than 100 square miles is

$$Q = \left(\frac{35000}{D + 32} + 10\right) D$$

Applying it to the experimental area, we have

$$Q = \left(\frac{35000}{7.34 + 32} + 10\right) 7.34$$

$$Q = 6600 \text{ cfs}$$

Obviously, this is an extremely high value, but the possibility remains that the constants could be altered so as to make the formula applicable to similar small areas.

Ganguillet Formula (6, p. 660). The Ganguillet formula was proposed for Swiss streams in 1883.

$$Q = \frac{1421\ D}{3.11 + D^{1/2}}$$

$$Q = \frac{1421 \times 7.34}{3.11 + 7.34^{1/2}}$$

$$Q = 1800 \text{ cfs}$$

This is a very reasonable value of discharge for Bowers Slough, and the formula could likely be used on similar small areas in this region.

El Paso & S. W. Railway Formula (5, p. 20). This formula was derived by engineers of the El Paso & S. W. Railway for use along their line in the southwestern states.

It is as follows:

$$Q = 17\,D\,\frac{(8000)^{1/2}}{D^{1/2}}$$

$$Q = 1521\,D^{1/2}, \text{ and applying this, we have}$$

$$Q = 1521\,(7.34)^{1/2}$$

$$Q = 4122 \text{ cfs}$$

Likewise, this is quite high for the area studied, as might be expected, due to the difference between the rainfall-runoff characteristics of southwestern and northwestern states. However, by modifying the constant and the power of D, it could be made applicable to similar small drainage areas.

<u>Elliot Formula</u> (6, p. 666). The Elliot formula was derived for use in northwestern Arkansas. It is

$$Q = 24\,D^{1/2} + 6\,D$$

When applied to the Bowers Slough drainage area, the discharge is as follows:

$$Q = 24\,(7.34)^{1/2} + 6(7.34)$$

$$Q = 1090 \text{ cfs}$$

This is slightly low for Bowers Slough, but with slight changes in the constant terms, it would be applicable to areas of the type studied in this region.

Table 6

Values of Discharge

from Formulas Applied to Bowers Slough

Formula	Discharge in cu. ft. per sec.
Talbot	2840
Adolph Meyer	993
Myers	1710
Wentworth	2890
Fanning	1052
Dickens	2230
Tidewater Railway	2310
Peck	7830
Rational	1059
Burkli-Ziegler	3000
Craig	3115
Pettis	3300
McMath	2940
Burge	3970
Murphy	1160
C. B. & Q. Railway	2620
Kuichling	6600
Ganguillet	1800
El Paso & S. W. Railway	4122
Elliot	1090
Average discharge excluding Kuichling and Peck formulas	2345

Conclusions

It is to be noted that the discharges obtained by applying the formulas to the experimental drainage area were varied. However, only two of those used gave unreasonable values, namely the Kuichling and Peck formulas. The average discharge given by the remaining eighteen formulas was 2345 cubic feet per second, 56 per cent greater than the probable twenty-five year flood for Bowers Slough of 1500 cubic feet per second. This average would more than likely correspond to the probable fifty year flood of the stream. A waterway area selected according to this average would be more than ample for a Bowers Slough crossing.

Those formulas which have been used in the past by railroad engineers such as the Wentworth, Tidewater Railway, McMath, Burge, C. B. & Q. Railway, and El Paso & S. W. Railway all tend to yield values which are more than sufficient. This is not surprising, in view of the conservative attitude of the railroads toward design, since they cannot risk possibility of loss of life due to derailment or wrecks caused by failure of bridges or culverts.

The average discharge given by the railroad formulas is 3142 cubic feet per second. A bridge or culvert for Bowers Slough having a waterway area selected according to this value of discharge would be twice as large as need be for an ordinary highway crossing.

The waterway formulas of the form $A = CM^n$, in which A is the waterway area required to pass maximum flood flows, C is a coefficient, M is the drainage area, and n is a power varying from 0.5 to 1.0, consistently yielded discharges which were too large. Those formulas which are of this type are the Talbot, Myers, Wentworth, Tidewater Railway, and Peck. The average discharge given by these formulas is 3516 cubic feet per second, approximately 2.3 times as great as the maximum probable twenty-five year discharge of 1500 cubic feet per second. It will be recalled that in the determination of the discharges using the waterway area formulas a velocity through the structure of ten feet per second was assumed.

The discharge formulas of the type $Q = CM^n$, in which Q is the discharge, C is a coefficient, M is the drainage area, and n is a power which varies from 0.5 to 0.75, yielded an average discharge of 2100 cubic feet per second. This group consists of the Adolph Meyer, Fanning, Dickens, and El Paso & S. W. Railway formulas. Of this group, probably the most reliable for small areas in this locality are the Adolph Meyer and Fanning formulas.

The remaining formulas do not group themselves into any definite category. Of these formulas, those giving the best results were the Rational, Murphy, Ganguillet, and Elliot formulas. The average discharge given by these was approximately 1280 cubic feet per second, slightly less than the probable twenty-five year maximum. However, these relationships are recommended for use on small drainage areas in the Willamette Valley which are similar to the type studied because their coefficients would require only slight modification.

Of all the formulas, the Rational formula seems to have the best possibilities, since it takes into account the intensity of rainfall as well as the drainage area. The Texas Highway Department has used this formula with much success, by determining proper values for the intensities and the coefficient for all regions of Texas.

However, any one of the twenty formulas could doubtless be made applicable to small areas if the coefficients and constants were to be modified by checking the values of discharge given by the formula against actual discharge records covering a period of years for a groups of areas within a region such as the Willamette Valley.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

The relatively short observation period of fourteen months during which the stream and drainage area were studied prevent any definite or absolute conclusions being reached concerning the applicability of the various runoff formulas to small watersheds. However, the data collected from this study did yield a basis upon which a reasonable maximum twenty-five year discharge could be selected for comparison with those given by the formulas. It is believed that the correlation established between the computed discharges and the maximum twenty-five year discharge gives a fair indication of the reliability of the individual formulas when applied to small drainage areas.

In order to evaluate more soundly the usefullness of the formulas, a study should be of about ten years duration. Not only should the present study be continued, but additional studies should be inaugurated on a number of watersheds varying in size from one to twenty square miles. At first, the studies should be confined to the Willamette Valley, and later expanded to cover other areas of the state of Oregon.

Obviously, the cost of such a study would be prohibitive if an individual or single agency were to undertake it. However, a cooperative survey sponsored by those to whom the resulting data might be of value would be economically feasible. The possibility exists that many of the rain gages and water level recorders required for the study could be obtained on a loan basis from such government agencies as the U. S. Weather Bureau, the U. S. Geological Survey, the U. S. Army Engineers, the Soil Conservation Service, and the Bureau of Reclamation. In addition, substantial aid might be obtained from the Public Roads Administration, the Oregon State Highway Department, and the Oregon State Engineers Office, all of whom have expressed much interest in such a study.

A cooperative study of runoff from small watersheds seems to be the only answer to the problems of engineers concerned with it. The data extracted from the study would largely eliminate the hazard of underdesign and the expense of overdesign of bridge and culvert waterway areas, as well as being beneficial to anyone concerned with the relationship of rainfall to runoff.

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APPENDIX

Description and Mechanical Analysis of Various Soil Types on Bowers Slough Watershed

Aiken Silty Clay Loam. The surface soil of the Aiken silty clay loam in its typical development consists of 10 to 12 inches of red to brownish-red silty clay loam. The subsoil is red in color, generally of about the same texture or slightly heavier than the soil, and compact. In the virgin state the surface soil has a high content of organic matter, and locally contains numerous round, partly cemented brown or rusty-brown iron concretions. In places the soil contains angular fragments of basalt and the bedrock is found at shallow depths though it rarely outcrops. The soil is friable and easily worked, and even when wet a granular structure largely counteracts the heavy structure. For the most part this soil is deeply weathered, bedrock being reached ordinarily at 4 to 6 feet from the surface. The Aiken silty clay loam is a residual soil derived from the weathering in place of basalt and to some extent from coarser grained basis igneous rocks. The drainage of this soil in general is good.

Amity Silty Clay Loam. The surface soil of Amity silty clay loam, from 7 to 12 inches deep, consists of light brown to light grayish-brown silty clay loam mottled with dark rusty-brown, being decidedly grayish in color when dry and a pronounced brown when moist. The subsoil usually consists of two distinct layers, one which may consist of a grayish-brown, dark-brown, or drab-colored material, moderately compact, and silty clay loam in texture, slightly mottled with rusty-brown to a depth of 20 inches, and a lower layer to a depth of 40 inches which consists of a friable, light grayish-brown silty clay loam material mottled with yellow, red, and brown stains. The upper layers are in general quite impervious, which somewhat restricts drainage.

Table 7

Mechanical Analysis of Amity Silty Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Surface, 0-2 in.	0.2	1.0	1.0	2.6	10.8	65.0	19.4
Subsurface, 2-12 in.	0.0	0.1	0.2	0.8	9.6	63.6	25.6
Subsoil, 12-18 in.	0.0	0.2	0.2	0.8	10.4	61.9	26.6
Subsoil, 18-36 in.	0.0	0.0	0.2	1.1	10.8	70.0	18.0

Olympic Clay Loam. The surface soil of Olympic clay loam consists of an 8 to 10 inch layer of chocolate-brown or slightly reddish-brown friable finely granular clay loam or clay containing shot-like pellets or concretions. In the virgin state this layer is well supplied with organic matter. It is underlain by rich-brown or reddish-brown clay or heavy clay loam. Below this, where well weathered, the material is reddish-brown smooth-textured moderately compact granular clay, or clay loam containing partly weathered rock fragments and in many places mottled with yellowish-brown material. Bedrock may occur within this zone, usually at a depth ranging from 4 to 6 feet. In general, it drains readily.

Table 8

Mechanical Analysis of Olympic Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Surface, 0-10 in.	2.7	4.4	3.6	6.6	6.4	34.2	42.2
Subsurface, 10-28 in.	2.7	4.2	3.3	6.1	6.0	32.7	45.2
Subsoil, 28-51 in.	2.0	3.3	2.6	5.3	6.4	33.6	46.8
Subsoil, 51-60 in.	2.0	5.1	4.4	11.5	15.7	35.7	25.5

Wapato Silty Clay Loam. The surface soil of typical Wapato silty clay loam consists of about 8 inches of brown or dark dull brown, heavy silty clay loam, in many places mottled with rusty-brown iron stains. The subsurface soil to a depth of 20 inches is underlain by a dull-brown heavy silty clay loam material, and deeper by a dark grayish-brown, drab, or bluish-gray clay mottled with gray and rusty-brown. Wapato silty clay loam is confined to the overflow lands along the smaller streams, to areas in local basins, and on alluvial-fan slopes. Surface drainage in places is fairly well developed, but subdrainage is poor.

Table 9

Mechanical Analysis of Wapato Silty Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Surface, 0-8 in.	0.3	1.1	1.0	6.6	11.2	50.8	29.1
Subsoil, 8-20 in.	0.2	1.0	1.0	4.9	14.6	49.2	29.2
Subsoil, 20-36 in.	0.0	0.5	0.7	9.2	12.2	22.3	54.9

Melbourne Clay Loam. The surface soil of the Melbourne clay loam consists of 8 to 10 inches of a brown to light-brown clay loam, containing sufficient fine and very fine sand to give it a comparatively friable structure. The subsoil has two sections, an upper layer consisting of a brown to reddish-brown friable heavy clay loam or clay, and a lower layer beginning at 20 to 24 inches, consisting of yellow or brownish-yellow moderately compact clay loam. The sandstone or shale from which the soil is derived is encountered at an average depth of 3 feet, although on the more gentle slopes the depth to bedrock is greater. Locally small fragments of these rocks are mixed with the soil and subsoil, though such areas are not large except where the rock formation outcrops. The drainage of this soil is good to excessive.

Table 10

Mechanical Analysis of Melbourne Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Soil, 0-8 in.	1.0	3.6	2.6	14.2	18.2	36.3	24.1
Subsurface, 8-20 in.	0.6	2.0	1.7	11.7	16.8	32.8	34.2
Subsoil, 20-36 in.	0.2	0.6	1.7	14.8	20.7	37.5	24.6

<u>Carlton Silty Clay Loam</u>. The surface soil of the Carlton silty clay loam is a light grayish-brown to grayish-brown smooth silty clay loam of friable structure, 8 to 13 inches deep. The subsoil to a depth of 36 inches or more is a dull brown to grayish-brown compact clay loam or silty clay loam which is mottled with gray or yellow in the lower part, especially in the lower more poorly drained areas. Partly weathered shale, from which rock the type is derived, is commonly not encountered above the depth of 4 feet, and cuts several feet deeper often fail to expose the bedrock. The type has a gently rolling or hilly to smoothly sloping surface. Surface drainage is well developed, though underdrainage is not good in all places.

Table 11

Mechanical Analysis of Carlton Silty Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Soil, 0-10 in.	0.4	0.5	0.4	2.4	7.8	66.0	22.6
Subsoil, 10-36 in.	0.2	0.9	0.8	4.2	7.2	64.7	22.1

Melbourne Silty Clay Loam. The Melbourne silty clay loam is a brown to light-brown friable silty clay loam, underlain by a subsoil of moderately compact yellow to brownish-yellow clay loam or silty clay loam, which at depths of 3 to 6 feet grades into partly weathered parent sandstone or shale. In places the surface soil contains varying quantities of sandstone or shale fragments, and in the virgin state it has a good supply of organic matter. The Melbourne silty clay loam commonly occupies the lower foothills bordering the valley and lower slopes of the more mountainous areas. Drainage is well developed. The type is of residual origin and derived principally from sandstone formations, some of which are very fine grained and hard and closely resemble the lightly colored igneous rocks.

Rough Mountainous Land. The rough mountainous land consists mainly of areas in the mountainous parts of the area which are undeveloped because of their steep and broken topography. The soils are of residual origin and derived either from igneous or sedimentary rocks. Bedrock is encountered at depths of 6 to 36 inches, and detached rock fragments or boulders are numerous on the surface. Rock outcrops are common along the breaks and steeper mountain sides. Except for a few small areas which have been burnt over, this land is heavily forested with fir and supports a dense growth of underbrush.

Willamette Silt Loam. The surface of the Willamette silt loam consists of 10 to 14 inches of dull-brown to light-brown, smooth friable silt loam or silty-clay loam. The subsoil is a brown to light brown moderately compact silty clay loam. The type has a gently sloping to slightly undulating surface, broken here and there by the steep banks of drainage ways. It occupies positions slightly higher than the surrounding soils, or else better drained areas adjacent to the streams. Drainage is well developed. About 80 per cent is under cultivation and the rest supports a heavy growth of fir.

Table 12

Mechanical Analysis of Willamette Silt Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Soil, 0-10 in.	0.6	0.9	0.5	3.4	6.6	64.6	23.4
Subsoil, 10-36 in.	0.2	0.5	0.6	4.0	7.0	62.3	25.3

Cascade Clay Loam. The surface soil of the Cascade clay loam typically consists of 12 to 14 inches of brown to light-brown clay loam, but in small included spots of deficient drainage the color of the surface material is dark brown to grayish-black and in places the texture is somewhat light for a clay loam. The subsoil is a yellow or brownish-yellow clay loam of compact structure, which becomes lighter in texture and less compact with increasing depth. Bedrock is reached at depths of 4 to 6 feet. The type is a residual soil derived from the coarser grained basic igneous rock. It occupies the crests of the flat or plateau-like lower hills or areas of gently sloping and rolling topography. In all cases the relief is sufficient to afford good drainage. The soil is retentive of moisture and is covered with a good growth of fir or oak.

Table 13

Mechanical Analysis of Cascade Clay Loam

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Soil, 0-12 in.	1.2	2.4	2.6	16.1	14.1	44.3	19.2
Subsoil, 12-36 in.	1.6	5.6	5.2	16.3	10.0	34.2	27.2

Cove Clay. Cove clay consists of a black or dark grayish-black or very dark dull-brown material from 8 to 15 inches deep and generally high in organic matter, underlain by a heavy-textured, waxy, bluish-gray or drab-colored clay, in many places mottled with rusty-brown. This soil occurs on gently sloping to flat or almost level areas. It occurs in low basin like areas and on gently slopes bordering higher-lying soils. Owing to its heavy subsoil, drainage is poorly developed.

Table 14

Mechanical Analysis of Cove Clay

Description	Fine Gravel %	Coarse Sand %	Medium Sand %	Fine Sand %	Very Fine Sand %	Silt %	Clay %
Surface soil, 0-10 in.	0.3	1.2	0.5	4.0	8.2	50.0	35.9
Subsoil, 10-36 in.	0.5	1.4	0.7	3.4	7.4	44.8	41.9

Table 15

Bowers Slough Discharge Measurement

Weather Rain

Date March 1, 1949

Gage Height 11.09

Quantity 12.15 cu.ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (seconds)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	0.9	0.2	0.7	--	--	--	--	--	--	--	--	--	--	--	--
3.0	1.8	0.4	1.4	--	24	--	127	--	0.19	--	0.47	0.24	0.12	3.36	0.40
6.0	2.7	0.5	2.2	47	23	123	121	0.38	0.19	0.90	0.47	0.68	0.47	5.02	2.36
9.0	2.5	0.5	2.0	27	25	139	128	0.19	0.20	0.48	0.48	0.48	0.58	7.80	4.60
12.0	2.4	0.4	1.9	23	22	121	120	0.19	0.18	0.47	0.45	0.46	0.47	4.12	1.94
15.0	2.0	0.3	1.6	14	10	121	130	0.11	0.08	0.30	0.21	0.26	0.36	5.10	1.84
18.0	1.6	0.3	1.3	3	3	120	120	0.03	0.03	0.10	0.10	0.10	0.18	5.10	0.92
18.6	1.6	0.3	1.3	3	3	120	120	0.03	0.03	0.10	0.10	0.10	0.10	0.92	0.09
												Total			12.15

Table 16

Bowers Slough Discharge Measurement

Weather Clear - Warm Date February 26, 1949

Gage Height 11.34 Quantity 19.25 cu.ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	1.0	0.2	0.8	--	--	--	--	--	--	--	--	--	--	--	--
3.0	2.0	0.4	1.6	16	34	130	129	0.12	0.29	0.32	0.69	0.50	0.25	3.96	0.99
6.0	2.1	0.4	1.7	80	56	154	135	0.52	0.41	1.22	0.98	1.10	0.80	5.77	4.62
9.0	2.8	0.6	2.2	38	32	137	144	0.67	0.54	0.28	0.22	0.61	0.85	8.55	7.27
12.0	2.0	0.4	1.6	24	27	136	140	0.18	0.19	0.44	0.48	0.46	0.54	4.57	2.47
15.0	2.5	0.5	2.0	18	18	140	140	0.13	0.11	0.33	0.28	0.30	0.38	5.85	2.22
18.0	1.9	0.4	1.5	8	10	130	115	0.06	0.09	0.18	0.24	0.21	0.25	5.85	1.46
18.6	1.9	0.4	1.5	8	10	130	115	0.06	0.09	0.18	0.24	0.21	0.21	1.07	0.22
												Total			19.25

Table 17

Bowers Slough Discharge Measurement

Weather Cold - Cloudy

Date February 13, 1949

Gage Height 11.34

Quantity 19.78 cu.ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	1.0	0.2	0.8	20	10	132	126	0.15	0.08	0.38	0.23	0.26	--	--	--
3.0	1.7	0.3	1.4	30	21	123	125	0.24	0.17	0.60	0.43	0.52	0.39	3.96	1.54
6.0	3.0	0.6	2.4	61	28	123	123	0.50	0.23	1.17	0.57	0.87	0.70	5.77	4.04
9.0	3.0	0.6	2.4	36	34	121	127	0.30	0.27	0.72	0.65	0.69	0.78	8.55	6.67
12.0	2.0	0.4	1.6	25	27	124	120	0.20	0.23	0.50	0.55	0.53	0.61	4.57	2.79
15.0	2.3	0.5	1.8	21	17	124	121	0.17	0.14	0.43	0.35	0.39	0.46	5.85	2.69
18.0	1.8	0.4	1.4	5	6	68	75	0.07	0.08	0.21	0.23	0.22	0.31	5.85	1.81
18.6	1.8	0.4	1.4	5	6	68	75	0.07	0.08	0.21	0.23	0.22	0.22	1.07	0.24
													Total		19.78

Table 18

Bowers Slough Discharge Measurement

Weather Cold - Cloudy

Date February 14, 1949

Gage Height 11.84

Quantity 27.29 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	1.4	--	--	--	--	--	--	--	--	--	--	--	--	--	--
3.0	1.9	0.4	1.5	9	9	63	67	0.14	0.13	0.36	0.34	0.35	0.18	5.16	0.93
6.0	3.6	0.7	2.9	37	24	62	63	0.60	0.38	1.40	0.90	1.15	0.75	7.27	5.45
9.0	3.4	0.7	2.7	23	18	63	61	0.37	0.30	0.87	0.71	0.79	0.97	10.05	9.75
12.0	2.5	0.5	2.0	20	20	80	62	0.25	0.32	0.61	0.78	0.70	0.75	5.89	4.42
15.0	3.0	0.6	2.4	11	7	60	72	0.18	0.10	0.46	0.26	0.36	0.53	7.35	3.90
18.0	2.3	0.5	1.8	7	7	61	61	0.11	0.11	0.30	0.30	0.30	0.33	7.35	2.43
18.6	2.3	0.5	1.8	7	7	61	61	0.11	0.11	0.30	0.30	0.30	0.30	1.37	0.41
												Total			27.29

Table 19

Bowers Slough Discharge Measurement

Weather Cold - Cloudy

Date February 12, 1949

Gage Height 11.92

Quantity 32.88 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	1.6	0.3	1.3	9	4	124	120	0.73	0.33	0.17	0.08	0.12	--	--	--
3.0	2.6	0.5	2.1	23	27	120	121	0.19	0.22	0.45	0.52	0.49	0.31	5.35	1.66
6.0	3.6	0.7	2.9	80	76	121	120	0.66	0.63	1.55	1.49	1.53	1.01	7.51	7.59
9.0	3.7	0.7	3.0	50	39	122	122	0.41	0.32	0.96	0.75	0.86	1.19	10.29	12.25
12.0	2.7	0.5	2.2	27	39	120	120	0.23	0.33	0.53	0.76	0.65	0.72	6.13	4.41
15.0	3.0	0.6	2.4	25	16	124	128	0.20	0.13	0.47	0.29	0.38	0.52	7.59	3.95
18.0	3.2	0.6	2.6	20	12	126	116	0.16	0.10	0.37	0.24	0.31	0.34	7.59	2.58
18.6	3.2	0.6	2.6	20	12	126	116	--	--	0.37	0.24	0.31	0.31	1.41	0.44
												Total			32.88

Table 20

Bowers Slough Discharge Measurement

Weather Clear - Warm

Date February 24, 1949

Gage Height 11.89

Quantity 36.16 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	1.6	0.3	1.3	--	--	--	--	--	--	--	--	--	--	--	--
3.0	2.6	0.5	2.1	25	44	122	121	0.21	0.36	0.51	0.87	0.69	0.35	5.28	1.85
6.0	3.6	0.7	2.9	87	61	124	122	0.70	0.50	1.63	1.18	1.40	1.05	7.42	7.79
9.0	3.5	0.7	2.8	61	50	135	124	0.45	0.40	1.07	0.96	1.02	1.21	10.20	12.34
12.0	2.6	0.5	2.1	35	46	124	122	0.28	0.38	0.68	0.90	0.79	0.90	6.04	5.44
15.0	2.8	0.6	2.2	31	18	125	128	0.25	0.14	0.61	0.36	0.49	0.64	7.50	4.80
18.0	2.3	0.5	1.8	23	20	125	151	0.18	0.13	0.46	0.34	0.40	0.45	7.50	3.38
18.6	2.3	0.5	1.8	23	20	125	151	0.18	0.13	0.46	0.34	0.40	0.40	1.40	0.56
													Total		36.16

Table 21

Bowers Slough Discharge Measurement

Weather Clear - Cold

Date February 11, 1949

Gage Height 12.67

Quantity 55.26 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	2.5	0.5	2.0	10	12	120	120	0.08	0.10	0.20	0.24	0.22	--	--	--
3.0	3.0	0.6	2.4	17	21	117	117	0.15	0.18	0.34	0.42	0.38	0.30	7.15	2.15
6.0	4.5	0.9	3.6	101	88	120	120	0.84	0.73	1.98	1.73	1.86	1.22	9.76	11.91
9.0	4.2	0.8	3.4	61	45	120	118	0.51	0.38	1.19	0.88	1.04	1.45	12.54	18.18
12.0	3.4	0.7	2.7	40	66	118	120	0.34	0.55	0.80	1.29	1.05	1.05	8.38	8.80
15.0	3.7	0.7	3.0	30	38	120	120	0.32	0.17	0.75	0.39	0.57	0.81	9.84	7.97
18.0	3.0	0.6	2.4	32	18	118	117	0.27	0.15	0.64	0.36	0.50	0.54	9.84	5.31
18.6	3.0	0.6	2.4	32	18	118	117	0.27	0.15	0.64	0.36	0.50	0.50	1.87	0.94
												Total			55.26

Table 22

Bowers Slough Discharge Measurement

Weather Cold - Rain and Snow

Date February 10, 1949

Gage Height 14.76

Quantity 202.10 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	4.5	0.9	3.6	--	--	--	--	--	--	--	--	--	--	--	--
3.0	5.2	1.2	4.2	45	70	125	125	0.43	0.56	1.01	1.32	1.17	0.59	12.17	7.18
6.0	6.4	1.3	5.1	200	185	121	122	1.65	1.52	3.88	3.57	3.73	2.45	16.03	39.27
9.0	7.0	1.4	5.6	145	135	121	122	1.20	1.11	2.82	2.61	2.72	3.23	18.81	60.76
12.0	6.7	1.3	5.4	120	135	122	121	0.98	1.12	2.30	2.64	2.47	2.60	14.65	38.09
15.0	6.0	1.2	4.8	90	60	125	121	0.72	0.48	1.69	1.13	1.41	1.94	16.11	31.25
18.0	5.4	1.1	4.3	85	50	124	129	0.69	0.39	1.61	0.92	1.27	1.34	16.11	21.59
18.6	5.4	1.1	4.3	85	50	124	129	0.69	0.39	1.61	0.92	1.27	1.27	3.12	3.96
												Total			202.10

Table 23

Bowers Slough Discharge Measurement

Weather Rain

Date February 17, 1949

Gage Height 15.74

Quantity 795.84 cu. ft. per sec.

Station	Depth Sound (ft.)	Depth Meter (ft.)		Total Rev.		Time (sec.)		Rev. per second		Velocity (ft./sec.)			Velocity Section (ft./sec.)	Area (sq. ft.)	Flow (cfs)
		0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	0.2D	0.8D	Vert.			
0.0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
0.6	5.8	1.2	3.6	--	--	--	--	--	--	--	--	--	--	--	--
3.0	6.5	1.3	5.2	235	385	122	122	1.93	3.16	4.41	7.72	5.84	2.92	14.52	42.40
6.0	7.7	1.5	6.2	510	500	125	125	4.08	4.00	9.34	9.16	9.25	7.55	18.97	143.22
9.0	8.3	1.7	6.6	450	580	128	121	3.52	4.79	8.19	10.95	9.57	9.41	21.75	204.67
12.0	8.0	1.6	6.4	460	465	128	135	3.59	3.44	8.22	7.87	8.05	7.96	17.59	140.02
15.0	7.3	1.5	5.8	275	435	124	125	2.22	3.48	5.08	7.97	6.53	7.29	19.05	138.87
18.0	6.7	1.3	5.4	240	320	141	125	1.70	2.56	3.90	5.84	4.87	5.70	19.05	108.59
18.6	6.7	1.3	5.4	240	320	141	125	1.70	2.56	3.90	5.84	4.87	4.87	3.71	18.07

Total 795.84

Bowers Slough Discharge Record

Table 24

Daily Discharge Record

Month & Day (1949)		Gage Ht.	Maximum Discharge (cfs)	Month & Day (1949)		Gage Ht.	Maximum Discharge (cfs)
Feb.	22	14.76	200.0	Apr.	1	10.89	8.0
	23	12.62	61.5		2	10.84	7.2
	24	11.92	34.0		3	10.79	6.3
	25	11.64	25.3		4	10.78	6.2
	26	11.46	20.2		5	10.69	4.9
	27	11.32	16.7		6	10.64	4.3
	28	11.19	14.0		7	10.64	4.3
					8	10.64	4.3
Mar.	1	11.09	11.7		9	10.60	3.8
	2	11.02	10.2		10	10.56	3.3
	3	10.96	9.2		11	10.56	3.3
	4	10.89	8.0		12	10.53	3.0
	5	10.84	7.2		13	10.52	2.9
	6	10.79	6.3		14	10.51	2.8
	7	10.74	5.7		15	10.47	2.4
	8	10.69	5.0		16	10.47	2.4
	9	10.64	4.3		17	10.48	2.5
	10	10.59	3.6		18	10.49	2.6
	11	10.55	3.2		19	10.49	2.6
	12	10.52	2.9		20	10.49	2.6
	13	10.51	2.8		21	10.51	2.8
	14	10.53	3.0		22	10.53	3.0
	15	10.58	3.5		23	10.48	2.5
	16	10.61	3.9		24	10.46	2.3
	17	10.66	4.5		25	10.43	2.0
	18	10.71	5.1		26	10.42	1.9
	19	10.76	5.9		27	10.42	1.9
	20	10.87	7.5		28	10.35	1.4
	21	10.97	9.2		29	10.40	1.8
	22	11.26	15.1		30	10.57	3.4
	23	11.06	11.0				
	24	11.03	10.7				
	25	11.14	12.7				
	26	11.24	14.8				
	27	11.56	22.8				
	28	11.24	14.8				
	29	11.14	12.7				
	30	11.04	10.6				
	31	10.94	8.8				

Table 24 cont.

Daily Discharge Record

Month & Day (1949)		Gage Ht.	Maximum Discharge (cfs)	Month & Day (1949)		Gage Ht.	Maximum Discharge (cfs)
May	1	12.47	55.0	June	1	10.44	2.1
	2	11.14	12.7		2	10.36	1.4
	3	10.94	8.8		3	10.30	1.0
	4	10.89	8.0		4	10.28	0.9
	5	10.82	6.8		5	10.20	0.6
	6	10.77	6.0		6	10.14	0.3
	7	10.74	5.7		7	10.11	0.2
	8	10.71	5.1		8	10.02	0.0*
	9	10.65	4.4				
	10	10.62	4.0	Nov.	23	11.10	12.0
	11	10.57	3.5		24	10.40	1.8
	12	10.54	3.1		25	10.50	2.7
	13	10.50	2.7		26	10.92	8.5
	14	10.47	2.4		27	11.82	30.8
	15	10.47	2.4		28	11.25	15.2
	16	10.45	2.2		29	11.34	15.3
	17	10.44	2.1		30	11.34	15.3
	18	10.44	2.1				
	19	10.38	1.6	Dec.	1	10.70	5.1
	20	10.54	3.1		2	11.02	10.1
	21	10.59	3.7		3	10.70	5.1
	22	10.58	3.5		4	10.63	4.1
	23	10.51	2.8		5	11.93	34.5
	24	10.44	2.1		6	11.93	34.5
	25	10.40	1.8		7	11.10	12.0
	26	10.34	1.3		8	10.91	8.2
	27	10.33	1.2		9	10.91	8.2
	28	10.33	1.2		10	11.03	10.6
	29	10.32	1.1		11	10.80	6.5
	30	10.31	1.0		12	10.73	5.5
	31	10.34	1.3		13	10.73	5.5

* Discharge 0.0 from June 8, 1949 to November 1, 1949.

Table 24 cont.

Daily Discharge Record

Month & Day (1949)		Gage Ht.	Maximum Discharge (cfs)	Month & Day (1950)		Gage Ht.	Maximum Discharge (cfs)
Dec.	14	10.73	5.5	Jan.	1	12.11	41.0
	15	11.34	15.3		2	11.74	28.5
	16	11.41	19.2		3	11.30	16.3
	17	12.80	70.0		4	11.27	15.5
	18	13.37	100.0		5	12.06	39.0
	19	12.30	48.5		6	15.65	757.0
	20	11.52	21.9		7	15.49	652.0
	21	11.27	15.1		8	13.52	108.0
	22	11.96	35.5		9	14.24	150.3
	23	13.25	92.0		10	15.59	715.0
	24	13.11	86.0		11	13.84	128.0
	25	12.96	78.0		12	12.42	53.0
	26	12.34	50.0		13	12.68	62.2
	27	11.88	33.0		14	12.32	49.0
	28	11.53	22.5		15	14.04	140.0
	29	12.11	41.0		16	11.71	27.5
	30	12.88	74.0		17	11.63	25.2
	31	13.00	80.0		18	10.63	4.2
					19	11.88	33.0
					20	14.71	186.0
					21	14.96	320.0
					22	14.63	182.0
					23	14.29	150.6
					24	13.13	88.5
					25	12.25	56.1
					26	14.05	140.2
					27	14.05	140.2
					28	12.76	68.0
					29	11.92	34.3
					30	11.92	34.3
					31	11.92	34.3

Table 24 cont.

Daily Discharge Record

Month & Day (1950)		Gage Ht.	Maximum Discharge (cfs)	Month & Day (1950)		Gage Ht.	Maximum Discharge (cfs)
Feb.	1	11.92	34.3	Mar.	1	12.72	66.0
	2	11.92	34.3		2	12.40	52.2
	3	11.92	34.3		3	12.00	37.0
	4	11.90	33.5		4	12.00	37.0
	5	13.39	100.0		5	12.34	50.0
	6	14.37	162.0		6	11.53	22.5
	7	14.95	312.0		7	11.36	17.8
	8	14.18	148.0		8	11.83	31.3
	9	12.84	70.7		9	11.88	33.0
	10	12.96	78.0		10	11.74	28.3
	11	12.59	60.0		11	11.15	13.0
	12	13.45	104.0		12	11.15	13.0
	13	13.45	104.0		13	11.47	20.5
	14	12.98	79.0		14	11.44	19.6
	15	12.98	79.0		15	11.44	19.6
	16	12.59	60.0		16	13.68	117.9
	17	12.02	37.8		17	14.85	197.0
	18	11.74	28.3		18	12.85	72.5
	19	11.57	23.3		19	13.57	112.0
	20	11.40	18.7		20	12.05	38.7
	21	11.32	16.8		21	13.02	81.0
	22	11.19	13.8		22	12.70	65.5
	23	13.50	107.0		23	12.14	42.0
	24	14.85	197.0		24	12.03	38.0
	25	14.72	188.0		25	11.68	26.5
	26	14.28	150.5		26	12.07	39.0
	27	13.75	122.0		27	12.60	60.5
	28	13.00	80.0		28	11.94	34.8
					29	11.44	19.7

Table 25

Hourly Discharge for
Flood of April 30-May 4, 1949

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Apr. 30	1	10.39	1.6	May 2	37	12.32	48.6
	2	10.39	1.6		38	12.34	49.5
	3	10.39	1.6		39	12.39	52.0
	4	10.41	1.8		40	12.40	52.0
	5	10.43	2.0		41	12.40	52.0
	6	10.43	2.0		42	12.32	48.6
	7	10.45	2.2		43	12.14	42.0
	8	10.47	2.4		44	12.03	38.0
	9	10.53	3.0		45	11.95	35.4
	10	10.57	3.4		46	11.87	32.0
	11	10.65	4.4		47	11.74	28.2
	12	10.73	5.5		48	11.69	26.8
					49	11.65	25.5
May 1	13	10.77	5.9		50	11.60	24.0
	14	10.89	8.1		51	11.57	23.0
	15	10.93	8.7		52	11.49	21.1
	16	11.01	10.0		53	11.46	20.2
	17	11.06	11.0		54	11.42	19.0
	18	11.08	11.5		55	11.39	18.5
	19	11.24	14.8		56	11.37	17.8
	20	11.39	18.5		57	11.31	16.4
	21	11.55	22.7		58	11.28	16.0
	22	11.72	27.2		59	11.24	14.8
	23	11.85	31.7				
	24	11.97	35.5	May 3	60	11.22	14.5
	25	12.34	49.5		61	11.22	14.5
	26	12.34	49.5		62	11.21	14.2
	27	12.34	49.5		63	11.20	14.0
	28	12.34	49.5		64	11.19	13.8
	29	12.37	51.0		65	11.18	13.6
	30	12.39	52.0		66	11.17	13.4
	31	12.47	55.0		67	11.16	13.2
	32	12.40	52.0		68	11.15	13.0
	33	12.32	48.6		69	11.15	13.0
	34	12.29	48.0		70	11.14	12.7
	35	12.29	48.0		71	11.13	12.5
	36	12.29	48.0		72	11.13	12.5

Table 25 cont.

Hourly Discharge for
Flood of April 30-May 4, 1949

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
May 3	73	11.11	12.2	May 4	85	11.01	10.1
	74	11.10	11.9		86	11.01	10.1
	75	11.08	11.5		87	11.00	10.0
	76	11.07	11.3		88	10.99	9.8
	77	11.06	11.0		89	10.98	9.6
	78	11.06	11.0				
	79	11.05	10.9				
	80	11.05	10.9				
	81	11.04	10.8				
	82	11.03	10.6				
	83	11.03	10.6				
	84	11.02	11.03				

Table 26

Hourly Discharge for
Flood of January 5-9, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 5	0	11.26	15.5	Jan. 7	37	15.49	650.0
	1	11.26	15.5		38	15.35	565.0
	2	11.28	16.0		39	15.22	480.0
	3	11.30	16.3		40	15.14	430.0
	4	11.34	17.5		41	15.03	365.0
	5	11.43	19.5		42	14.88	275.0
	6	11.53	22.5		43	14.72	185.0
	7	11.63	25.1		44	14.57	175.0
	8	11.75	28.7		45	14.41	165.0
	9	11.84	31.5		46	14.26	155.0
	10	11.99	36.5		47	14.15	147.0
	11	12.12	41.0		48	14.11	145.0
	12	12.22	45.0		49	14.11	145.0
					50	14.13	146.0
Jan. 6	13	12.36	51.0		51	14.14	146.5
	14	12.44	54.0		52	14.21	152.0
	15	12.48	55.5		53	14.21	152.0
	16	12.52	57.0		54	14.17	148.0
	17	12.56	59.0		55	14.11	145.0
	18	12.57	59.0		56	14.02	140.0
	19	12.64	63.0		57	13.91	132.0
	20	12.76	68.0		58	13.80	125.0
	21	12.98	79.0		59	13.68	117.0
	22	13.29	95.0		60	13.57	112.0
	23	13.76	122.0				
	24	13.98	135.0	Jan. 8	61	13.49	107.0
	25	14.22	155.0		62	13.40	102.0
	26	14.40	165.0		63	13.33	97.0
	27	14.58	175.0		64	13.25	93.0
	28	14.74	200.0		65	13.18	88.0
	29	14.85	245.0		66	13.11	86.0
	30	15.25	500.0		67	13.05	82.0
	31	15.52	665.0		68	12.98	79.0
	32	15.61	725.0		69	12.93	76.0
	33	15.65	755.0		70	12.88	74.0
	34	15.63	740.0		71	12.82	71.0
	35	15.56	695.0		72	12.77	68.0
	36	15.53	680.0		73	12.73	66.0

Table 26 cont.

Hourly Discharge for

Flood of January 5-9, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 8	74	12.70	65.0	Jan. 9	85	12.85	72.0
	75	12.69	64.0		86	12.77	68.0
	76	12.68	64.0		87	12.70	65.0
	77	12.66	64.0		88	12.63	62.0
	78	12.68	64.0		89	12.54	58.0
	79	12.82	71.0		90	12.49	56.0
	80	12.88	74.0		91	12.42	55.0
	81	12.93	76.0		92	12.36	51.0
	82	12.95	77.0		93	12.31	48.0
	83	12.94	77.0		94	13.27	47.0
	84	12.88	74.0		95	12.23	46.5
					96	12.21	45.0

Table 27

Hourly Discharge for
Flood of January 9-13, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 9	0	12.21	45.0
	1	12.27	47.0
	2	12.39	52.0
	3	12.49	56.0
	4	12.63	62.0
	5	12.78	68.0
	6	12.98	79.0
	7	13.20	90.0
	8	13.42	102.0
	9	13.63	115.0
	10	13.86	128.0
	11	14.02	138.0
Jan. 10	12	14.28	155.0
	13	14.52	173.0
	14	14.75	200.0
	15	15.01	350.0
	16	15.38	590.0
	17	15.53	685.0
	18	15.58	715.0
	19	15.55	695.0
	20	15.46	635.0
	21	15.35	565.0
	22	15.17	450.0
	23	14.96	320.0
	24	14.79	225.0
	25	14.63	182.0
	26	14.57	175.0
	27	14.54	173.0
	28	14.55	174.0
	29	14.59	180.0
	30	14.55	175.0
	31	14.52	172.0
	32	14.42	165.0
	33	14.28	155.0
	34	14.06	142.0
	35	13.99	138.0
Jan. 11	36	13.77	123.0
	37	13.63	118.0
	38	13.53	108.0
	39	13.42	102.0
	40	13.31	96.0
	41	13.23	92.0
	42	13.14	87.0
	43	13.05	82.0
	44	13.00	80.0
	45	12.93	76.0
	46	12.88	73.0
	47	12.82	71.0
	48	12.78	69.0
	49	12.77	68.0
	50	12.76	67.0
	51	12.76	67.0
	52	12.76	67.0
	53	12.75	67.0
	54	12.73	66.0
	55	12.70	65.0
	56	12.67	64.0
	57	12.62	61.0
	58	12.54	58.0
	59	12.50	56.0
Jan. 12	60	12.46	55.0
	61	12.43	53.0
	62	12.40	52.0
	63	12.33	49.0
	64	12.31	47.5
	65	12.27	47.0
	66	12.23	45.5
	67	12.20	44.0
	68	12.16	43.0
	69	12.14	42.0
	70	12.11	41.0
	71	12.08	40.0

Table 27 cont.

Hourly Discharge for
Flood of January 9-13, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 12	72	12.08	40.0	Jan. 13	84	11.82	31.0
	73	12.08	40.0		85	11.80	30.5
	74	12.08	40.0		86	11.77	28.0
	75	12.08	40.0		87	11.75	27.5
	76	12.08	40.0		88	11.74	27.5
	77	12.08	40.0		89	11.71	27.5
	78	12.08	40.0		90	11.70	27.0
	79	12.06	39.0				
	80	12.01	37.0				
	81	11.94	35.0				
	82	11.92	34.0				
	83	11.87	32.5				

Table 28

Hourly Discharge for
Flood of January 19-23, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 19	0	10.63	4.2	Jan. 21	33	14.85	250.0
	1	10.63	4.2		34	14.84	245.0
	2	10.73	5.5		35	14.81	231.0
	3	10.97	9.4		36	14.78	217.0
	4	11.14	13.0		37	14.74	203.0
	5	11.31	16.5		38	14.71	190.0
	6	11.65	25.8		39	14.70	185.0
	7	11.75	28.9		40	14.70	185.0
	8	11.86	32.0		41	14.75	200.0
					42	14.80	225.0
Jan. 20	9	12.08	40.3		43	14.85	250.0
	10	12.30	48.6		44	14.89	280.0
	11	12.52	57.0		45	14.93	305.0
	12	12.56	59.0		46	14.94	313.0
	13	12.60	60.8		47	14.96	320.0
	14	12.64	62.6		48	14.95	315.0
	15	12.68	64.4		49	14.94	322.0
	16	12.72	66.0		50	14.90	285.0
	17	12.77	68.0		51	14.86	265.0
	18	12.82	71.0		52	14.81	245.0
	19	12.88	74.0		53	14.77	226.0
	20	12.96	78.0		54	14.73	207.0
	21	13.03	82.0		55	14.68	185.0
	22	13.17	90.0		56	14.63	180.0
	23	13.31	98.0				
	24	13.45	106.0	Jan. 22	57	14.55	178.0
	25	13.60	114.0		58	14.48	170.0
	26	13.74	120.0		59	14.40	165.0
	27	13.89	130.0		60	14.33	160.0
	28	14.16	147.0		61	14.36	162.0
	29	14.41	165.0		62	14.39	164.0
	30	14.58	177.0		63	14.43	166.0
	31	14.68	185.0		64	14.46	168.0
	32	14.75	200.0		65	14.40	165.0

Table 28 cont.

Hourly Discharge for
Flood of January 19-23, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)
Jan. 22	66	14.31	159.0
	67	14.22	153.0
	68	14.12	145.0
	69	14.05	140.0
	70	14.02	158.0
	71	13.98	135.0
	72	13.98	135.0
	73	13.95	131.0
	74	13.89	128.0
	75	13.83	125.0
	76	13.77	122.0
	77	13.79	124.0
	78	13.89	130.0
	79	14.04	140.0
	80	14.19	150.0
Jan. 23	81	14.25	152.0
	82	14.29	155.0
	83	14.23	151.0
	84	14.15	147.0
	85	14.02	140.0
	86	13.94	134.0
	87	13.86	129.0
	88	13.78	123.0
	89	13.69	117.0
	90	13.63	112.0
	91	13.48	107.0
	92	13.42	102.0
	93	13.41	102.0
	94	13.39	101.0
	95	13.38	100.0
	96	13.38	100.0
	97	13.38	100.0
	98	13.38	100.0
	99	13.32	97.0
	100	13.26	94.0
Jan. 23	101	13.20	91.0
	102	13.14	88.0
	103	13.07	84.0
	104	13.00	80.0
Jan. 24	105	12.92	76.0
	106	12.84	72.0
	107	12.82	71.0
	108	12.81	70.0
	109	12.79	69.0
	110	12.78	68.0
	111	12.77	67.0
	112	12.76	66.0
	113	12.74	65.0
	114	12.72	65.0
	115	12.69	64.0
	116	12.65	63.0
	117	12.63	62.0
	118	12.61	61.5
	119	12.59	61.0
	120	12.57	60.5
	121	12.55	59.0
	122	12.53	58.0
	123	12.49	56.5
	124	12.45	55.0
	125	12.41	53.5
	126	12.37	52.0
	127	12.33	50.5
	128	12.29	49.0

Table 29

Hourly Discharge for
Flood of February 5-9, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Feb. 5	0	11.86	32.0	Feb. 7	34	14.37	163.0
	1	11.89	33.2		35	14.35	161.0
	2	11.93	34.1		36	14.28	155.0
	3	11.97	36.0		37	14.15	147.0
	4	12.02	37.0		38	14.02	138.0
	5	12.05	38.8		39	13.87	128.0
	6	12.08	40.0		40	13.73	121.0
	7	12.11	41.0		41	13.63	115.0
	8	12.16	43.0		42	13.55	109.0
	9	12.24	45.5		43	13.50	106.0
					44	13.44	103.0
Feb. 6	10	12.30	48.0		45	13.40	101.0
	11	12.36	51.0		46	13.37	99.0
	12	12.43	53.0		47	13.41	102.0
	13	12.49	56.0		48	13.50	107.0
	14	12.57	59.0		49	13.62	114.0
	15	12.66	64.0		50	13.73	121.0
	16	12.75	67.0		51	13.78	123.0
	17	12.86	73.0		52	13.91	132.0
	18	12.96	78.0		53	13.89	130.0
	19	13.12	86.0		54	13.78	123.0
	20	13.27	93.0		55	13.73	122.0
	21	13.38	100.0		56	13.66	121.0
	22	13.52	107.0		57	13.61	112.0
	23	13.62	115.0				
	24	13.74	122.0	Feb. 8	58	13.56	110.0
	25	13.91	132.0		59	13.55	108.0
	26	14.05	140.0		60	13.57	111.0
	27	14.08	142.0		61	13.70	118.0
	28	14.05	140.0		62	14.11	145.0
	29	14.08	142.0		63	14.57	177.0
	30	14.02	138.0		64	14.73	200.0
	31	14.08	142.0		65	14.92	300.0
	32	14.20	151.9		66	14.95	314.0
	33	14.33	160.0		67	14.90	285.0

Table 29 cont.

Hourly Discharge for
Flood of February 5-9, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)
Feb. 8	68	14.73	200.0
	69	14.52	172.0
	70	14.27	154.0
	71	14.06	141.0
	72	13.95	134.0
	73	13.85	127.0
	74	13.70	118.0
	75	13.65	116.0
	76	13.60	112.0
	77	13.54	108.0
	78	13.45	103.0
	79	13.38	100.0
	80	13.32	96.0
	81	13.25	92.0
Feb. 9	82	13.18	88.0
	83	13.13	86.0
	84	13.08	84.0
	85	13.04	82.0
	86	13.00	80.0
	87	12.94	77.0
	88	12.90	75.0
	89	12.87	73.0
	90	12.83	71.0
	91	12.80	70.0
	92	12.78	69.0
	93	12.77	68.0
	94	12.76	68.0
	95	12.75	67.0
	96	12.73	66.0
	97	12.73	66.0
	98	12.73	66.0
	99	12.73	66.0
Feb. 9	100	12.73	66.0
	101	12.73	66.0
	102	12.73	66.0
	103	12.70	65.0
	104	12.67	63.0
	105	12.63	62.0
Feb. 10	106	12.61	61.0
	107	12.58	59.5
	108	12.54	58.0
	109	12.53	57.5
	110	12.50	56.5
	111	12.48	55.5

Table 30

Hourly Discharge for
Flood of March 15-20, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)
Mar. 15	0	10.94	8.9
	1	10.94	8.9
	2	10.97	9.4
Mar. 16	3	11.05	11.0
	4	11.30	16.3
	5	11.59	24.0
	6	11.99	37.0
	7	12.46	54.5
	8	12.84	69.0
	9	13.09	84.0
	10	13.27	94.0
	11	13.49	105.0
	12	13.57	110.0
	13	13.65	116.0
	14	13.68	118.0
	15	13.78	123.0
	16	13.90	132.0
	17	14.02	138.0
	18	14.12	145.0
	19	14.12	145.0
	20	13.98	135.0
	21	13.81	125.0
	22	13.60	112.0
	23	13.39	100.0
	24	13.27	93.0
	25	13.27	93.0
	26	13.42	102.0
Mar. 17	27	13.72	120.0
	28	14.12	145.0
	29	14.44	167.0
	30	14.64	180.0
	31	14.79	192.0
	32	14.85	197.0
	33	14.83	195.0
	34	14.73	187.0
	35	14.48	168.0
Mar. 17	36	14.24	152.0
	37	14.12	145.0
	38	14.10	143.0
	39	14.10	143.0
	40	14.00	137.0
	41	13.93	132.0
	42	13.84	127.0
	43	13.66	117.0
	44	13.51	108.0
	45	13.36	99.0
	46	13.23	92.0
	47	13.14	87.0
	48	13.04	82.0
	49	12.99	79.0
	50	12.92	78.0
Mar. 18	51	12.83	72.0
	52	12.77	68.0
	53	12.72	66.0
	54	12.69	65.0
	55	12.66	64.0
	56	12.63	62.0
	57	12.60	61.0
	58	12.61	61.0
	59	12.62	61.0
	60	12.66	64.0
	61	12.70	65.0
	62	12.76	68.0
	63	12.89	74.0
	64	12.97	78.0
	65	13.09	84.0
	66	13.20	90.0
	67	13.35	98.0
	68	13.48	105.0
	69	13.57	111.0
	70	13.57	111.0
	71	13.49	105.0
	72	13.35	98.0
	73	13.20	90.0
	74	13.13	86.0

Table 30 cont.

Hourly Discharge for
Flood of March 15-20, 1950

Month & Day	Hour	Gage Ht.	Flow (cfs)	Month & Day	Hour	Gage Ht.	Flow (cfs)
Mar. 19	75	13.08	84.0	Mar. 20	111	11.86	32.1
	76	12.96	77.0		112	11.86	32.1
	77	12.86	73.0		113	11.87	32.3
	78	12.78	68.0		114	11.88	32.5
	79	12.70	65.0		115	11.87	32.3
	80	12.66	64.0		116	11.86	32.1
	81	12.59	60.0		117	11.85	32.0
	82	12.52	57.0		118	11.83	31.2
	83	12.50	56.0		119	11.79	30.0
	84	12.49	56.0		120	11.75	28.8
	85	12.50	56.0		121	11.72	27.5
	86	12.54	58.5		122	11.69	27.0
	87	12.57	59.0				
	88	12.57	59.0	Mar. 21	123	11.67	26.5
	89	12.52	57.0		124	11.66	26.0
	90	12.46	55.0		125	11.65	25.5
	91	12.41	52.5		126	11.64	25.2
	92	12.36	51.0		127	11.63	24.9
	93	12.30	48.0		128	11.62	24.5
	94	12.26	47.0		129	11.61	24.4
	95	12.20	44.5				
	96	12.15	42.5				
	97	12.11	41.0				
	98	12.08	40.0				
Mar. 20	99	12.05	38.8				
	100	12.01	37.0				
	101	12.00	36.8				
	102	11.99	36.6				
	103	11.97	36.0				
	104	11.95	35.1				
	105	11.94	35.0				
	106	11.92	33.5				
	107	11.90	33.0				
	108	11.88	32.3				
	109	11.87	32.2				
	110	11.86	32.1				

Weather Records

Table 31

Daily Weather Record
Bowers Slough Watershed and Corvallis
February 21, 1949 - June 1, 1949

Date		Station 1	2	3	4	CORVALLIS	CORVALLIS Wind	Temperature (degrees Fahr.) Max.	Min.
		Precipitation in inches							
Feb.	21	0.23	0.24	0.24	0.23	0.25	S	59	45
	22	1.24	1.24	1.20	1.24	1.17	S	55	35
	23	0.03	0.02	0.02	0.02	0.05	NE	55	38
	24	--	--	--	--	--	NE	64	41
	25	--	--	--	--	--	N	63	36
	26	--	--	--	--	--	N	62	39
	27	--	--	--	--	--	NE	63	34
	28	--	--	--	--	--	W	63	31
Mar.	1	0.02	0.03	0.04	0.01	0.01	NE	54	40
	2	0.03	0.02	0.02	0.01	--	SE	61	44
	3	0.03	0.04	0.04	0.09	0.06	N	56	43
	4	0.01	0.01	0.01	0.01	--	NE	63	42
	5	0.01	0.01	0.01	0.01	0.01	S	59	44
	6	--	--	--	--	--	SW	50	32
	7	0.10	0.10	--	--	0.03	S	56	39
	8	0.10	0.10	0.16	0.16	0.08	SW	51	40
	9	0.05	0.10	0.03	0.03	0.02	N	52	35
	10	--	--	--	--	--	N	59	42
	11	--	--	--	--	--	N	60	41
	12	--	--	--	--	--	S	58	35
	13	0.33	0.32	0.33	0.30	0.20	SE	59	43
	14	0.07	0.07	0.07	0.06	0.19	E	55	42
	15	0.09	0.11	0.25	0.17	0.09	E	54	40
	16	0.17	0.15	0.15	0.26	0.12	SW	60	43
	17	0.40	0.45	0.55	0.42	0.27	S	59	45
	18	0.20	0.15	0.07	0.08	0.13	NE	58	48
	19	0.26	0.14	0.25	0.14	0.01	S	55	49
	20	0.34	0.30	0.35	0.22	0.09	SE	54	42

Table 31 cont.

Daily Weather Record
Bowers Slough Watershed and Corvallis
February 21, 1949 - June 1, 1949

		Station 1	2	3	4	CORVALLIS	CORVALLIS Wind	Temperature (degrees Fahr.) Max.	Min.
Date		Precipitation in inches							
Mar.	21	0.15	0.20	0.14	0.20	0.23	S	52	40
	22	0.13	0.12	0.14	0.15	0.09	SW	58	38
	23	0.01	0.01	0.01	0.02	0.04	W	57	40
	24	--	--	--	--	--	W	56	31
	25	0.07	0.18	0.16	0.15	0.03	W	55	40
	26	0.30	0.50	0.31	0.24	0.18	S	50	36
	27	0.23	0.24	0.41	0.39	0.23	N	51	39
	28	--	--	--	--	--	NE	53	30
	29	0.30	0.09	0.10	0.10	0.08	SW	53	38
	30	0.02	--	0.01	--	--	N	55	37
Apr.	1	--	--	--	--	--	W	53	
	2	--	--	--	--	--	SE	52	
	3	--	--	--	--	--	SE	60	
	4	--	--	--	--	--	W	73	
	5	--	--	--	--	--	NE	67	
	6	--	--	--	--	--	W	75	
	7	--	--	--	--	--	W	53	
	8	--	--	--	--	--	NE	62	
	9	--	--	--	--	--	W	65	
	10	--	--	--	--	--	N	67	
	11	--	--	--	--	--	W	55	
	12	--	--	--	--	--	W	62	
	13	--	--	--	--	--	N	61	
	14	--	--	--	--	--	N	77	
	15	--	--	--	--	--	W	59	
	16	--	--	--	--	--	E	54	
	17	--	--	--	--	--	W	53	
	18	--	--	--	--	--	SW	58	
	19	0.10	0.09	0.15	0.09	0.08	W	54	
	20	0.25	0.09	0.07	0.10	0.03	SW	58	
	21	0.04	0.09	0.05	0.15	0.10	W	55	
	22	0.04	0.05	0.06	0.06	--	N	67	
	23	--	--	--	--	0.03	SW	55	
	24	--	--	--	--	--	W	50	

Table 31 cont.

Daily Weather Record
Bowers Slough Watershed and Corvallis
February 21, 1949 - June 1, 1949

Date		Station 1	Station 2	Station 3	Station 4	CORVALLIS	CORVALLIS Wind	CORVALLIS Temperature (degrees Fahr.) Max.	Min.
		Precipitation in inches							
Apr.	25	--	--	--	--	--	N	62	
	26	--	--	--	--	--	W	66	
	27	0.02	0.08	0.05	0.05	--	W	62	
	28	0.03	0.01	0.02	0.02	0.02	W	55	
	29	--	--	--	--	0.02	W	53	
	30	0.48	0.42	0.47	0.45	0.27	SW	48	
May	1	1.30	1.14	1.60	0.20	1.03	S	55	
	2	1.10	1.45	0.46	0.28	0.45	SW	52	
	3	0.04	0.05	--	--	--	N	52	
	4	--	--	--	--	--	NE	63	
	5	--	--	--	--	--	N	72	
	6	--	--	--	--	--	N	80	
	7	--	--	--	--	--	W	74	
	8	--	--	--	--	0.01	W	58	
	9	--	--	--	--	--	W	72	
	10	--	--	--	--	--	NE	80	
	11	--	--	--	--	--	NE	85	
	12	--	--	--	--	--	W	83	
	13	--	--	--	--	--	SE	76	
	14	--	0.02	--	--	--	W	59	
	15	--	--	0.05	--	--	W	58	
	16	--	--	--	--	--	W	59	
	17	0.03	0.03	0.06	0.03	0.03	N	54	
	18	--	--	--	--	--	SE	73	
	19	0.30	0.25	0.25	0.17	0.03	S	56	
	20	0.08	0.08	0.08	0.19	0.20	W	59	
	21	0.03	0.05	0.05	0.18	0.05	SW	67	
	22	--	--	--	--	0.10	W	71	
	23	--	--	--	--	0.01	W	71	
	24	--	--	--	--	--	N	70	
	25	--	--	--	--	--	NE	79	
	26	--	--	--	--	--	W	68	
	27	--	--	--	--	--	W	67	
	28	--	--	--	--	--	N	58	
	29	--	--	--	--	--	W	62	
	30	--	--	--	--	--	SW	64	
	31	0.65	0.19	0.09	0.48	0.15	SW	61	

Table 32

Daily Weather Record - Corvallis, Oregon
February 1949

Day	Precipitation in inches	Temperature (degrees Fahr.) Max.	Min.	Wind Direction
1	0.01	45	29	East
2	0.05	39	30	South
3	0.38	40	31	South
4	0.45	37	30	Southwest
5	0.02	38	24	South
6	0.51	39	32	West
7	0.21	41	29	Southwest
8	0.06	43	32	Southwest
9	0.39	44	36	South
10	1.78	47	33	Southwest
11	0.21	43	25	West
12	Trace	42	27	West
13	Trace	40	27	South
14	0.23	38	30	Southeast
15	0.86	46	35	South
16	0.35	54	44	South
17	2.13	50	44	West
18	1.26	44	38	--
19	0.19	56	35	South
20	0.02	56	36	Southeast
21	0.25	59	45	South
22	1.17	55	35	South
23	0.05	55	38	Northeast
24	--	64	41	Northeast
25	--	63	36	North
26	--	62	39	North
27	--	63	34	Northeast
28	--	63	31	West
Total	10.58			

Table 33

Daily Weather Record - Corvallis, Oregon
March 1949

Day	Precipitation in inches	Temperature (degrees Fahr.) Max.	Min.	Wind Direction
1	0.01	54	40	Northeast
2	--	61	44	Southeast
3	0.06	56	43	North
4	Trace	63	42	Northeast
5	0.01	59	44	South
6	--	50	32	Southwest
7	0.03	56	39	South
8	0.08	51	40	Southwest
9	0.02	52	35	North
10	--	59	42	North
11	--	60	41	North
12	--	58	35	South
13	0.20	59	43	Southeast
14	0.19	55	42	East
15	0.09	54	40	East
16	0.12	60	43	Southwest
17	0.27	59	45	South
18	0.13	58	48	Northeast
19	0.01	55	49	South
20	0.09	54	42	Southeast
21	0.23	52	40	South
22	0.09	58	38	Southwest
23	0.04	57	40	West
24	Frost	56	31	West
25	0.03	55	40	West
26	0.18	50	36	South
27	0.23	51	39	North
28	Frost	53	30	Northeast
29	0.08	53	38	Southwest
30	--	55	37	North
31	Frost	59	32	Northeast
Total	2.19			

Table 34

Daily Weather Record - Corvallis, Oregon
April 1949

Day	Precipitation in inches	Temperature (degrees Fahr.) Maximum	Wind Direction
1-18	No rain		
19	0.08	54	West
20	0.03	58	Southwest
21	0.10	55	West
22	--	67	West
23	0.03	55	Southwest
24	Trace	50	West
25	--	62	North
26	--	66	West
27	Trace	62	West
28	0.02	55	West
29	0.02	53	West
30	0.27	48	Southwest
Total	0.55		

Table 35

Daily Weather Record - Corvallis, Oregon
May 1949

Day	Precipitation in inches	Temperature (degrees Fahr.) Maximum	Wind Direction
1	1.03	55	South
2	0.45	52	Southwest
3-7	No rain		
8	0.01	58	West
9-14	No rain		
15	Trace	58	West
16	--	59	West
17	0.03	54	North
18	0.01	73	Southeast
19	0.03	56	South
20	0.20	59	West
21	0.05	67	Southwest
22	0.10	71	West
23-27	No rain		
28	Trace	58	North
29	--	62	North
30	--	64	Southwest
31	0.15	61	Southwest
Total	2.06		

Table 36

Daily Weather Record - Corvallis, Oregon
June 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1	0.08	0.11	West
2-18	No rain	--	--
19	Trace	--	Southwest
20	0.47	0.66	North
21-25	No rain	--	--
26	Trace	--	Northeast
27	--	--	--
28	0.03	0.04	West
29	0.10	0.14	West
30	--	--	--
Total	0.68	0.95	

Table 37

Daily Weather Record - Corvallis, Oregon
July 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1-18	No rain	--	--
19	0.02	0.03	West
20-22	No rain	--	--
23	Trace	--	Southwest
24	0.01	0.01	West
25	Trace	--	West
Total	0.03	0.04	

Table 38

Daily Weather Record - Corvallis, Oregon
August 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1-5	No rain	--	--
6	0.03	0.04	East
7	0.24	0.34	West
8-31	No rain	--	--
Total	0.27	0.38	

Table 39

Daily Weather Record - Corvallis, Oregon
September 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1-7	No rain	--	--
8	0.07	0.10	South
9	0.65	0.91	West
10	0.03	0.04	West
11	Trace	--	Northwest
12-13	No rain	--	--
14	0.03	0.04	Southwest
15	0.04	0.06	West
16	0.20	0.28	West
17-18	No rain	--	--
19	0.02	0.03	West
20-27	No rain	--	--
28	0.49	0.69	West
29	0.03	0.04	West
30	--	--	--
Total	1.56	2.19	

Table 40

Daily Weather Record - Corvallis, Oregon
October 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1-3	No rain	--	--
4	0.14	0.20	West
5	0.40	0.56	West
6	0.35	0.49	West
7-8	No rain	--	--
9	0.18	0.25	West
10	0.21	0.29	South
11	0.07	0.10	West
12-14	No rain	--	--
15	0.05	0.07	South
16	--	--	--
17	0.02	0.03	South
18-24	No rain	--	--
25	0.02	0.03	Southwest
26	0.03	0.04	South
27	0.04	0.06	Southwest
28	0.21	0.29	West
29-31	No rain	--	--
Total	1.72	2.41	

Table 41

Daily Weather Record - Corvallis, Oregon
November 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Wind Direction
1-7	No rain	--	--
8	0.03	0.04	South
9	0.36	0.50	South
10	0.18	0.25	South
11	0.66	0.92	South
12	0.02	0.03	Southwest
13-20	No rain	--	--
21	0.04	0.06	South
22	0.05	0.07	South
23	1.70	2.38	Southwest
24	0.02	0.03	South
25	0.20	0.28	West
26	0.16	0.22	South
27	0.86	1.20	Northwest
28	0.26	0.36	South
29	0.24	0.34	North
30	0.31	0.43	South
Total	5.09	7.11	

Table 42

Daily Weather Record - Corvallis, Oregon
December 1949

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Temperature (degrees Fahr.) Max.	Min.	Wind
1	0.02	0.03	54	45	South
2	0.20	0.28	54	42	West
3-4	No rain	--	--*	--*	
5	0.48	0.67	50	34	South
6	0.01	0.01	45	34	Northeast
7-8	No rain	--	--*	--*	--
9	0.10	0.14	48	36	West
10-11	No rain	--	--*	--*	--
12	0.05	0.07	43	34	Southeast
13	Trace	--	47	37	Northeast
14	--	--	48	39	
15	0.25	0.35	45	31	Northeast
16	0.01	0.01	45	34	South
17	0.67	0.94	47	38	West
18	0.55	0.77	45	34	Southwest
19	0.02	0.03	41	27	South
20	--	--	--*	--*	--
21	0.01	0.01	47	38	South
22	Trace	--	48	43	South
23	0.33	0.46	45	42	South
24	0.53	0.74	50	39	West
25	0.22	0.31	49	30	Southeast
26	0.15	0.21	48	37	South
27	0.01	0.01	50	41	South
28	--	--	--*	--*	--
29	0.13	0.18	53	38	Southwest
30	0.21	0.29	45	31	Southwest
31	0.34	0.48	46	37	Southwest
Total	4.29	5.99			

* Temperature not recorded for this day.

Table 43

Daily Weather Record - Corvallis, Oregon
January 1950

Day	Precipitation in inches Rain	Precipitation in inches Snow	Prorated rainfall Bowers Slough	Temperature (degrees Fahr.) Max.	Temperature (degrees Fahr.) Min.	Wind
1	0.19	2.00	0.27	41	29	West
2	--	Trace	--	34	19	North
3	--	--	--	30	12	South
4	0.11	1.50	0.15	36	28	Northwest
5	Trace	--	--	38	29	Southwest
6	1.46	--	2.04	44	35	Southwest
7	1.01	1.50	1.41	38	32	South
8	0.25	Trace	0.35	37	32	West
9	0.53	1.25	0.74	38	30	Southwest
10	1.53	0.25	2.14	39	32	South
11	0.37	3.00	0.52	35	29	South
12	0.28	3.00	0.39	38	28	West
13	0.62	3.00	0.87	41	31	West
14	0.65	3.00	0.91	34	21	Northeast
15	0.28	7.00	0.39	34	19	South
16	0.55	0.50	0.77	34	27	East
17	0.05	2.50	0.07	29	19	Northeast
18	0.26	2.00	0.36	24	11	North
19	0.40	--	0.56	37	24	South
20	0.30	--	0.42	46	34	Southwest
21	0.56	--	0.78	52	41	Southeast
22	0.10	--	0.14	52	43	South
23	0.42	Trace	0.59	50	32	Southwest
24	0.21	4.00	0.29	34	26	West
25	--	Trace	--	34	23	South
26	0.71	0.50	0.99	40	32	South
27	0.81	6.00	1.13	39	26	North
28	0.25	1.00	0.35	38	26	Northeast
29	0.02	2.75	0.03	30	19	East
30	0.25	7.00	0.35	25	14	North
31	--	Trace	--	19	-1	East
Total	12.17	51.75	17.01			

Table 44

Daily Weather Record - Corvallis, Oregon
February 1950

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Temperature (degrees Fahr.) Max.	Min.	Wind
1	--	--	27	12	North
2	--	--	23	3	East
3	0.21	0.29	20	1	Southwest
4	0.27	0.39	45	12	West
5	0.08	0.11	46	35	North
6	0.82	1.15	44	32	South
7	0.60	0.84	45	34	Southwest
8	0.43	0.60	47	39	Southwest
9	--	--	56	38	West
10	0.14	0.20	51	37	Southwest
11	0.07	0.10	48	37	Southwest
12	0.19	0.27	49	39	South
13	0.26	0.36	51	44	Southwest
14	0.07	0.10	55	45	Southwest
15	0.11	0.15	53	46	South
16	0.10	0.14	53	44	South
17	--	--	59	37	Southwest
18	--	--	58	34	Southwest
19	--	--	53	35	Southwest
20	0.02	0.03	50	36	West
21	Trace	--	59	39	West
22	0.05	0.07	59	38	Southwest
23	0.61	0.85	55	44	West
24	0.67	0.94	55	49	South
25	0.38	0.53	64	51	West
26	0.15	0.21	59	40	Southwest
27	--	--	50	29	North
28	--	--	53	31	North
Total	5.23	7.33			

Table 45

Daily Weather Record - Corvallis, Oregon
March 1950

Day	Precipitation in inches	Prorated rainfall Bowers Slough	Temperature (degrees Fahr.) Max.	Min.	Wind
1	--	--	53	34	South
2	0.20	0.28	52	44	South
3	0.09	0.13	63	56	West
4	--	--	60	51	Southwest
5	0.10	0.14	59	42	West
6	0.04	0.06	50	36	South
7	--	--	49	29	East
8	0.15	0.21	49	37	West
9	0.14	0.20	53	38	West
10	0.19	0.27	49	33	West
11	--	--	49	29	West
12	--	--	49	27	South
13	0.21	0.29	51	38	South
14	0.04	0.06	53	39	West
15	--	--	52	38	Southwest
16	0.43	0.60	53	43	South
17	0.88	1.23	53	43	Southwest
18	0.22	0.31	49	39	Southeast
19	0.04	0.06	55	41	South
20	0.02	0.03	50	35	South
21	0.46	0.64	50	33	South
22	0.14	0.20	51	38	Southwest
23	0.14	0.20	51	39	Southwest
24	0.20	0.28	51	35	West
25	0.03	0.04	48	35	West
26	0.14	0.20	48	38	West
27	0.24	0.34	50	37	Southwest
28	0.01	0.01	52	34	West
29	--	--	64	35	Southeast
30	0.05	0.07	61	42	East
31	--	--	66	46	Southwest
Total	4.16	5.82			