TEXAS WATER DEVELOPMENT BOARD

REPORT 33

SYMPOSIUM ON CONSIDERATION OF SOME ASPECTS OF STORMS AND FLOODS IN WATER PLANNING

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Texas Section, American Society of Civil Engineers

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FOREWORD

Some of the numerous technical questions necessary to be resolved in water planning work and in detailed investigations of specific projects relate to the hydrologic and hydraulic analyses of the characteristics and effects of large storms and floods. The officers of the Hydraulics Division, Texas Section of the American Society of Civil Engineers (ASCE) have performed a distinct service to water resources planners by arranging for a symposium of technical papers providing very useful information on the various aspects of storms and floods and their relationship to the design of dams. These papers were presented at the Fall Meeting of the Hydraulics Division of the Texas Section, ASCE, in Fort Worth, Texas, October 7-9, 1965. They have been compiled and published in this volume by the Texas Water Development Board to make these important papers available for use throughout the State.

The Board wishes to acknowledge the assistance provided by the Texas Section, ASCE, and by the participants who have contributed their efforts in the preparation of the technical papers. As a variety of thoughts are included herein, the views expressed in the papers are those of the authors and do not necessarily reflect the views of the Texas Water Development Board.

TEXAS WATER DEVELOPMENT BOARD

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CHARACTERISTIC METEOROLOGY OF SOME LARGE

FLOOD-PRODUCING STORMS IN TEXAS--

EASTERLY WAVES[a]

by

Robert Orton[b]

Abstract

The dynamic meteorology and synoptic weather characteristics of easterly waves are discussed. The rainstorm of September 9-10, 1952, which produced storm totals up to 26.00 inches is described in detail.

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[a] As approved for publication by Environmental Science Services Administration, U.S. Department of Commerce.

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Introduction

The largest flood-producing storms to affect Texas are generated by weather disturbances of tropical origin. I am referring to those disturbances which are imbedded in the tropical easterlies or trade winds and are carried westward across the Gulf of Mexico into Texas. It is not surprising that this is so; one would hardly expect weather disturbances approaching Texas from the west or north, carried by the westerly current of middle latitudes, to pick up the enormous quantities of moisture found in easterly wind currents whose trajectories lie across several thousand miles of warm tropical waters.

In particular, I would like to discuss the meteorological circumstances which resulted in torrential rains of up to 26 inches within a two-day period in the Hill Country west of Austin, September 9-10, 1952. The meteorology of this unusually heavy rainstorm is characteristic of those large flood-producing storms generated by easterly waves.

Definition

An easterly wave is a migratory sinusoidal oscillation in the broad easterly wind patterns of the lower latitudes. It moves from east to west, usually at a slower rate than the current in which it is imbedded [1]. The easterly wave is essentially a weak trough of low pressure. Waves occur in the easterlies over many parts of the tropics. Those of interest in our region of the world are found in the deep easterly trade wind currents forming the southern portions of the Azores-Bermuda high pressure area (Figure 1).

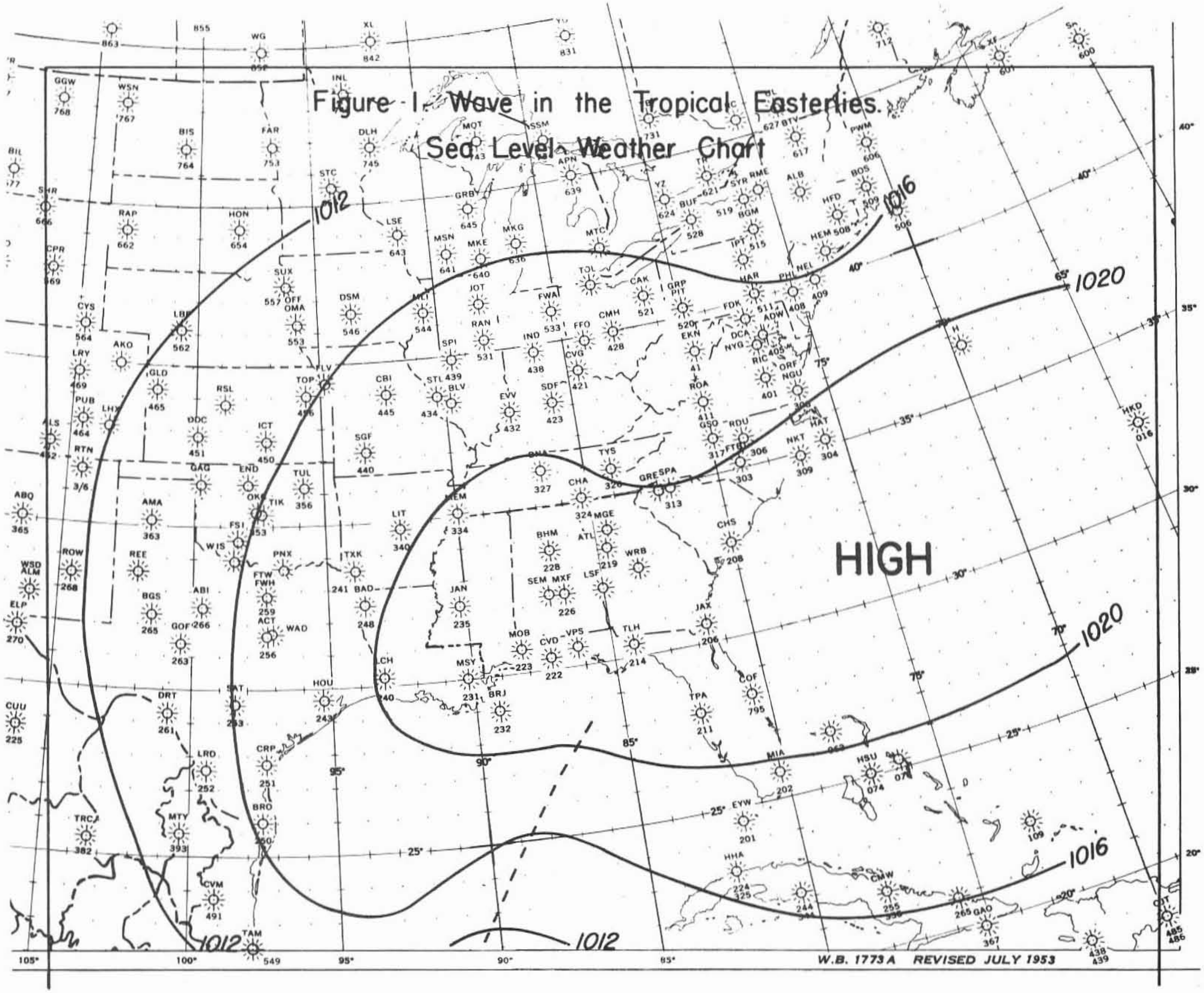

Figure I. Wave in the Tropical Easterlies. Sea Level Weather Chart

Dynamics and Weather

After the meteorologist concerned with southern Texas and Gulf Coast weather has observed sharp, well-defined troughs in the westerlies during the late fall, winter, and spring seasons, and then must direct his attention to weather systems arriving from the opposite direction during the summer months, he finds the slight poleward bulges of the isobars or streamlines that represent easterly waves on his weather map rather unimpressive. As a matter-of-fact, in the vast expanse of the tropical Atlantic Ocean where few weather reports are available, these perturbations in the easterly wind current are often difficult to detect. There is no significant temperature discontinuity in the easterly wave, no surface front such as usually accompanies waves in the westerlies. Ordinarily the troughs of low pressure tilt slightly toward the east with height and most often have their maximum intensity somewhere between 5,000 and 15,000 feet above sea level; thus, they are often more easily detected aloft than near the surface. Above 300 millibars (about 30,000 feet), disturbances with a wind field entirely different from that in the low levels may prevail [2]. Before upper air charts became available, Gordon Dunn, now Director, National Hurricane Center, Miami, Florida, first detected easterly waves by observing a series of centers of falling and rising pressure moving from east to west across the islands of the Caribbean. The movement of these centers of pressure change, known as isallobaric centers, still remains one of the most effective means of keeping track of easterly waves [3]. More recently, photographs of cloud formations by weather satellites have proven to be a valuable aid in the early detection of vigorous waves in areas where few surface reports are available.

As the wave moves westward more slowly than the wind current, massive clouds and showers or continuous rain occur to the east and in the center while to the west, ahead of the wave, the weather is exceptionally fine with little or no cloudiness observed. Figure 2 shows an idealized model of this type of disturbance with the curvature of the streamlines somewhat exaggerated compared to what is usually observed in nature. Subsidence and horizontal divergence precede the wave. Divergence means simply that the low-level wind is taking more air out of a specified area than it is bringing in. The air sinks or subsides and fair weather results. To the east of the trough line there is horizontal convergence, meaning that more air is moving into a specified area in the lower levels than is going out. The air is forced upward, the depth of the moist layer increases, heavy cumulus and cumulonimbus clouds form, and middle and high level cloud layers develop as the moist air extends to higher and higher levels. As the wave trough approaches, the normal east wind backs to northeast and the pressure falls. Behind the trough the pressure rises. Over the ocean, showers and thundershower activity and convergence reach a maximum 200 to 300 miles behind the surface trough [3].

The weather pattern I have just described to you is quite different from that accompanying the waves in the westerly current that move across Texas from west to east during the cooler seasons of the year. In the westerly trough, bad weather arrives ahead of the trough line, while precipitation ends and clearing occurs after the trough line has passed. This is the weather sequence with which most of you are familiar. I have already mentioned the enormous supply of moisture available to an easterly wave disturbance after a long trajectory across warm tropical waters, moisture that is available to produce record rainfall under certain meteorological conditions, as you will see. I would like to mention briefly one other significant difference in the easterly and the westerly waves. In easterly waves the effects of the variation in the relative vorticity and the vorticity of the earth's surface combine to give large variations in the stability. Vorticity is a measure of local

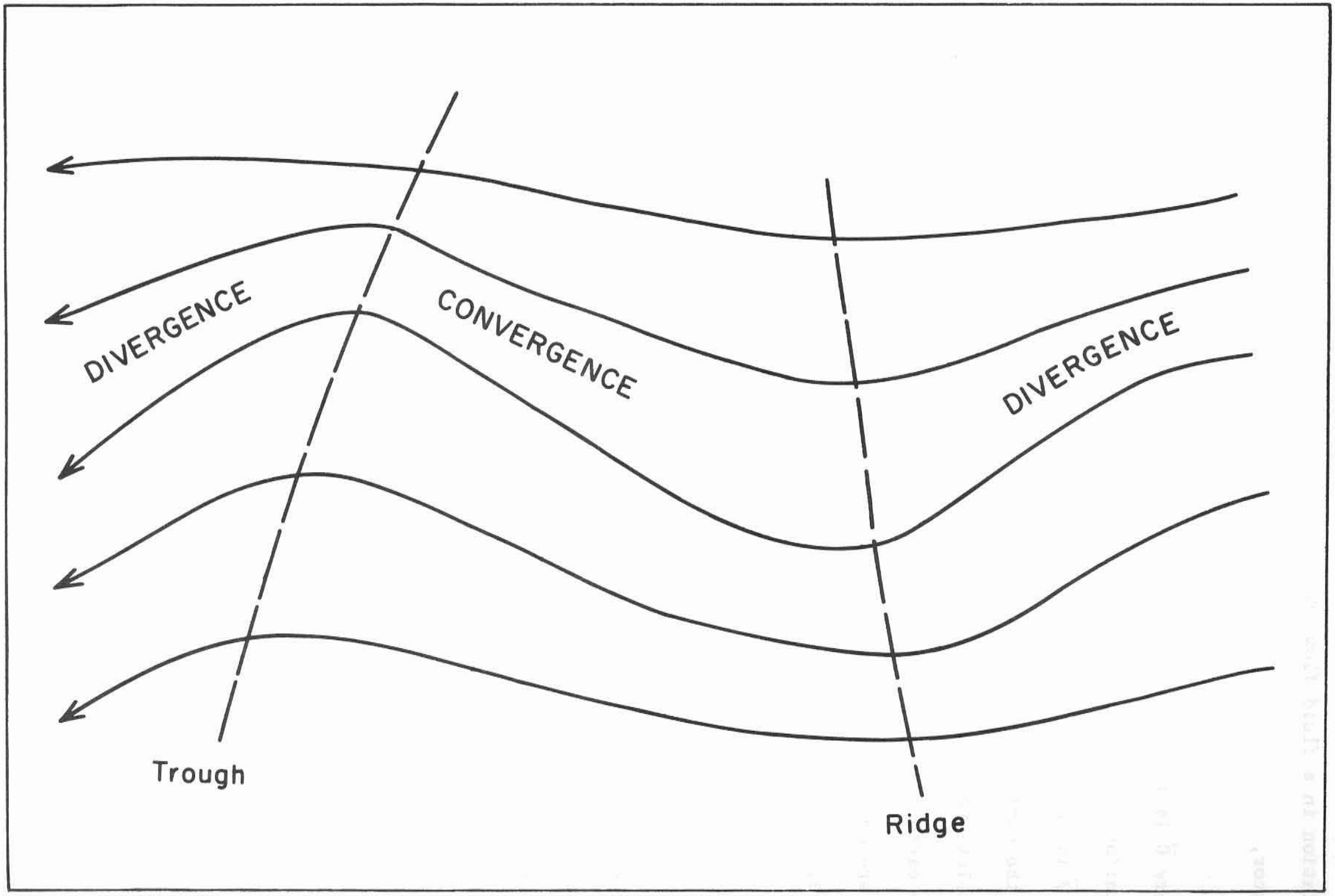

Figure 2. Streamlines of Airflow Around Wave in Tropical Easterlies

rotation in a fluid flow, defined mathematically as the curl of the velocity vector,

$$\vec{Q} = \Delta \times \vec{V},$$

where $\vec{Q}$ is the vorticity, $\vec{V}$ the velocity, and Δ the del-operator. In solid rotation, $\vec{Q}$ is equal to 2Ω, or twice the vector angular velocity of rotation. If $\vec{V}$ is the velocity of the air relative to the earth and $\vec{V}_E$ is the velocity of the earth's surface, then the absolute velocity of the air is $\vec{V} + \vec{V}_E$. The absolute vorticity is equal to the sum of the vorticity measured relative to the earth's surface and the vorticity of the earth's surface. In meteorology we are concerned only with the vertical component of the vorticity. In this case,

$$Q_z = q_z + 2\Omega \sin\phi = q_z + f$$

where q_z is the vertical component of the relative vorticity, Ω is the angular velocity of the earth, and ϕ is the latitude. The term $2\Omega \sin\phi$ is the vertical component of the earth's vorticity, and is the same as the coriolis parameter, f. In the easterly wave, as the air to the east of the trough line moves to a cyclonic regime, its vorticity relative to the earth's surface increases. Since the air is also moving toward higher latitudes, the coriolis parameter, or vorticity due to the earth's rotation, also increases. The absolute vorticity is thereby increased. The relation between changes of the absolute vorticity and the changes in the static stability of the air mass are expressed by

$$Q\ (\gamma_d - \gamma) = Q_0\ (\gamma_d - \gamma_0);$$

thus, an increase in the absolute vorticity must be accompanied by an increase in the lapse rate. In pronounced cases, the stability factor expressed as the dry adiabatic lapse rate minus the actual lapse rate $(\gamma_d - \gamma)$ may be reduced to less than 10 percent of its original value [4]. To the west of the trough line the decrease in the relative vorticity of the air as it moves away from the trough line, and the decrease in the coriolis parameter as the air moves toward lower latitudes, combine to make the air mass remarkably stable in the wedge portion of the easterly wave. In the waves of the westerlies, with which we are all more familiar, the variations in the relative vorticity are opposed by the variations in the coriolis parameter. In the westerly trough, as the cyclonic relative vorticity increases the air is moving toward low latitudes; consequently, the coriolis parameter decreases. The change toward a more unstable air mass is thus less pronounced than in the case of the easterly wave.

My purpose in reviewing very briefly some of the dynamics of the easterly wave is to acquaint you with the fact that for their amplitude, the easterly wave possesses a considerably greater potential for producing excessive rains than do wave disturbances in the westerlies. I have limited my discussion to a stable type of wave, which means that the trough may travel 2,000 or 3,000 miles around the southern periphery of the Azores-Bermuda high pressure area with little or no change in the shape of the wave until it crosses land areas. A little later, Mr. Carr is going to describe to you what happens when these waves become unstable and a vortex develops.

Synoptic Features

A wave in the easterlies is present over some part of the Caribbean almost every day from June through September, with a somewhat lesser frequency in May, October, and November. According to Dunn [3] a station in the eastern Caribbean may expect a wave passage on an average of about twice a week from June through September. Weather from these disturbances may affect the southern Texas region, expecially the coastal section, any time from June through October, but the maximum occurrence is in August and September. Most easterly waves average about 5° longitude per day crossing the Gulf of Mexico. In Texas, the excessive rainstorms that may accompany these waves are most likely to be confined to the area south of 32° north latitude and east of 100° west longitude. When an easterly wave passes inland, the model weather pattern previously described may be greatly distorted by orographic and diurnal influences. Small amplitude waves that cross the Texas coast during the afternoon are not often easily identified by their weather characteristics off shore, but generate numerous thunderstorms immediately inland. Usually these thundershowers dissipate after sundown. Weak waves nearing the coastline after dark are retarded and appear to remain off shore, becoming most active after sunrise. The weather activity accompanying waves that move inland during the morning hours tends to decrease or dissipate but regenerate during the afternoon hours. In other words, the weather associated with the easterly wave is strongly influenced by diurnal changes in both the low level wind field and the static stability of the air mass [5].

The majority of easterly waves weaken or lose their identity soon after moving inland so that only the Texas coastal plain is affected. These waves most often produce extensive cloudiness, thunderstorms, and a few locally heavy rain showers. Only the more vigorous waves, or those that later intensify as a result of extratropical influences, reach the Austin-San Antonio area and the Hill Country beyond. The weather associated with these well developed waves that reach the Balcones Escarpment is profoundly influenced by this orographic barrier. The forced ascent of the escarpment by the moist tropical air has contributed to the release of some of the heaviest rains in Texas weather history, including the great Thrall, Texas storm of September 9-10, 1921, when 36.40 inches fell within an 18-hour period [6].

As previously mentioned, easterly waves occur most frequently in August and September. These waves may intensify as the result of extratropical influences that are more likely to be present over southern Texas in September than in August. Riehl [7] found that whenever an easterly wave (moving westward) approaches a trough in the westerlies (moving eastward at a higher altitude) both trough and wave intensify. This development is most likely to occur in September. Polar air masses push farther southward in September than in August. Higher pressure associated with this cooler air mass may intensify an easterly wave over southern Texas by forcing a more cyclonic curvature of the flow into the wave trough, thus increasing its vorticity in the manner previously discussed.

By now, I hope I have made several points reasonably clear:

(1) The weather pattern associated with an easterly wave is quite different from that of a westerly wave.

(2) The rainstorm potential is considerably greater in the case of the easterly wave.

(3) Certain external influences, either orographic or meteorological, are usually necessary to transform the precipitation pattern associated with a stable easterly wave into a large flood-producer.

(4) The combination of factors required for such a high area-depth-duration rainstorm are not often present.

(5) The month of September is the most favorable period for these flood-producing storms to occur.

The Rainstorm of September 9-10, 1952

All of the necessary ingredients for a rainstorm of major proportions were present in South Central Texas on September 9-10, 1952. An article in the Monthly Weather Review some years ago brought out the following characteristics of this storm [8]. The week before the storm, southern Texas was invaded by continental polar air for the first time in the season. Modification of this air mass, which also covered part of northern Mexico, proceeded slowly during the next few days. As the center of the high pressure system accompanying the polar outbreak moved into the eastern United States, a wedge of high pressure extended southwestward into Texas. A reinforcement of the high by a fresh polar outbreak raised the pressure over the southeastern states on the 9th. The sea level 24-hour pressure change chart for midnight September 8, indicated an easterly wave had moved westward into the Gulf of Mexico, and the low pressure trough was approaching the Texas coast (pressure change chart not shown in text). By 3:30 p.m. of the 9th, the trough of the wave had moved well inland with the trough line near San Antonio (see Figure 3). The 700-mb constant pressure chart for 9:00 a.m. of the 9th (a pressure surface at approximately 10,000 feet above sea level) indicated that the trough was probably tilted slightly westward with altitude (chart not shown in text). Over water, as you may recall, the wave trough usually tilts toward the east as the height above the sea surface increases.

Heavy rains began falling in the Guadalupe and Colorado River, and Cibolo Creek watersheds about 6:00 p.m. on the 8th and continued to about 6:00 p.m. on the 10th. Although the rainfall on the 9th was substantial, the heaviest amounts fell on the 10th.

The San Antonio upper air sounding at 9:00 p.m. of September 9 (just prior to the heavy rain) indicated the air mass was convectively unstable from the surface to about 15,000 feet. A small amount of upperward movement would induce complete saturation of this air column. Presumably the Balcones Escarpment did provide the necessary lift to produce saturation and consequent instability. A study of the temperature field indicated slight warm temperature advection from the coastal section westward to the Hill Country at both the 850-mb level (approximately 5,000 feet M.S.L.) and the 700-mb level (approximately 10,000 feet M.S.L.). At 3:30 p.m. on the 9th, cooling due to rain, resulted in surface temperatures 12 degrees cooler at Junction than at Austin or San Antonio.

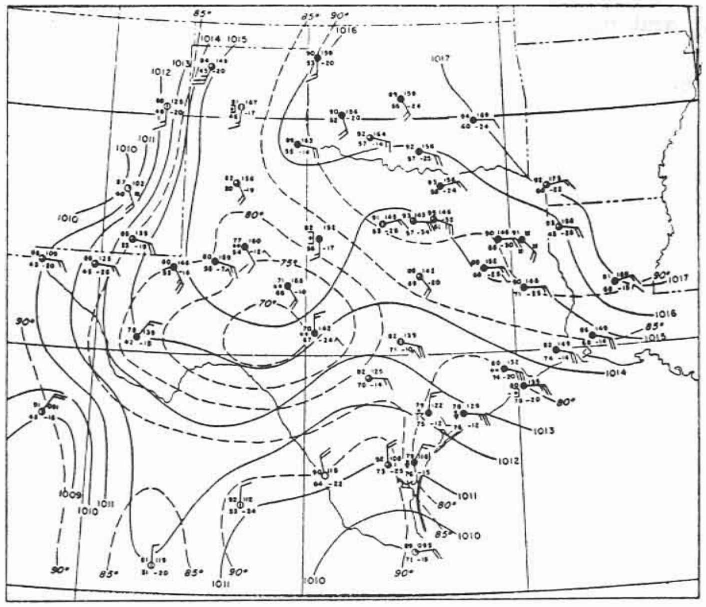

Figure 3. Sea level chart for 2130 GMT, September 9, 1952. Temperatures are in °F.

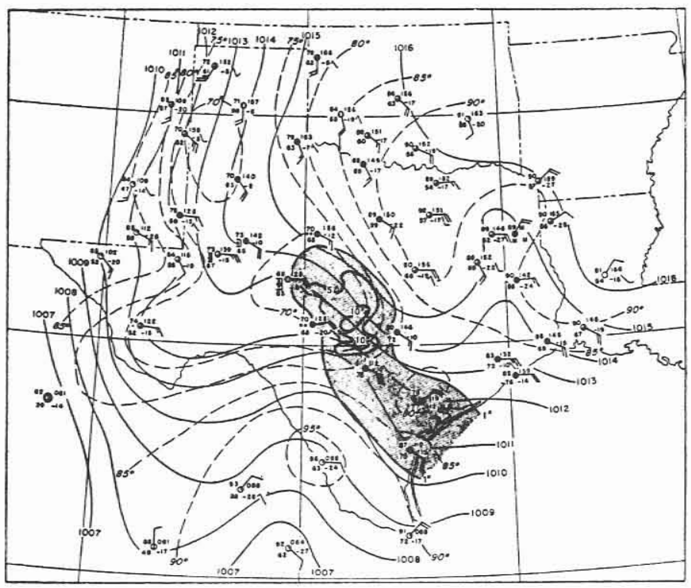

Figure 4. Sea level chart for 2130 GMT, September 10, 1952. Temperatures are in °F. A generalization of the isohyetal pattern in the vicinity of San Antonio for the 24-hour period ending 1230 GMT, September 11, 1952 is shown by stippling.

Between noon and midnight of the 9th, the high pressure over the southern states pushed southwestward toward Texas. This occurred just prior to the arrival of the easterly wave over South Central Texas. The result was a steepening of the pressure gradient in this area and an increase in the amplitude of the wave. Convergence on the east side of the trough line was increased through contributions from both an increase in the cyclonic relative vorticity and a more southerly component of the flow. These changes in the configuration of the sea level isobaric pattern are portrayed by the weather chart for 3:30 p.m., September 10 (Figure 4). Temperatures over the Hill Country did not rise despite the strenghtening of the warm advection over the heavy rain area on the 10th, indicating a concentrated area of strong vertical motion. These events, since they preceeded the period of heaviest rainfall, appear to be significant.

This particular combination of meteorological events resulted in rains of 10 to 12 inches over a large area. Storm totals of 20 to 26 inches were concentrated in a small center over Blanco and Kendall Counties. The official gauge at Hye, on the Pedernales River, in the west part of Blanco County, recorded 23.35 inches in 48 hours, of which 20.70 inches fell in one 24-hour period ending at 7:00 a.m. on the 11th. Unofficial measurements at Hye gave storm totals of 26.00 inches. A storm total of 25.10 inches (unofficial) was reported from Comfort in Kendall County. The highest stages ever known occurred in the Pedernales, and flooding occurred in the San Saba and Llano.

The outstanding feature of this flood was the rapid rise of Lake Travis. A total of 713,130 acre-feet of water poured into the lake to raise the level behind Mansfield Dam 57 feet. If the flood had not been stopped by the dam, it is estimated that a stage of 47 feet would have been reached at Austin with a flow of 750,000 cubic feet per second - four feet higher than the record July 1869 flood which produced a stage of 43 feet with a maximum discharge of 550,000 cubic feet per second [9]. The most damaging flood at Austin occurred in June

1935 and produced a stage of 41.2 feet with a maximum discharge of 481,000 cubic feet per second, resulting in property losses estimated at $13 million [10]. The September 1952 flood would have produced the highest stage at Austin since at least 1833, with a calculated return period of 126 years. Thanks to Mansfield Dam, there was no flooding at Austin or below as a result of the September 1952 storm. Total property losses from this storm amounted to several million dollars; 454 homes were damaged, and 17 homes destroyed. Five persons were drowned and three were injured [11].

While heavy rains occurred along the middle Texas coast in conjunction with the storm, as much as 9.34 inches at Palacios and 10.17 inches at Port O'Connor; amounts were surprisingly light at Austin and San Antonio. Austin recorded only 2.40 inches and San Antonio 2.45 inches. Between these two cities, 8.82 inches fell at New Braunfels and 9.65 inches fell at San Marcos. The highest totals for the storm, 20 to 26 inches, fell on the edge of the Edwards Plateau immediately west of this area which suggests that the Balcones Escarpment played a significant role in the development of this unusually heavy rainstorm.

Conclusion

The meteorology of the September 1952 rainstorm is reasonably characteristic of those few large flood-producing storms generated by the dissipating stages of more-or-less stable easterly waves. Specifically, the combination of events present in a storm of this type appear to be:

(1) The presence over southern Texas of a convectively unstable air mass.

(2) A large body of rain-cooled air which establishes a marked temperature gradient similar to that found along a cool front.

(3) Pronounced warm geostrophic advection (established by the isobaric configuration). The warmer air ascends the rain-cooled air similar to a frontal surface.

(4) Strong vertical motion resulting from horizontal convergence.

(5) A decrease in air mass stability due to vorticity changes.

(6) A decrease in stability due to the lifting of the air current over higher terrain, in this case, the Balcones Escarpment.

The circumstances which lead to this particular combination of events need not always be the same. In the September 1952 storm, it was apparently the near simultaneous arrival over South Central Texas of a pressure surge from the northeast and the easterly wave trough that set the stage for torrential rains.

In another instance, the simultaneous arrival of a westerly trough (at higher altitude) and the easterly wave could possibly have about the same results.

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CHARACTERISTIC METEOROLOGY OF SOME LARGE FLOOD-PRODUCING STORMS IN TEXAS-- HURRICANES

by

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Abstract

Some elements of hurricane-genesis, hurricane growth, and a hurricane model are discussed. A Gulf of Mexico hurricane which produced heavy rainfall in Texas is described and illustrated.

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General

The best link between the title of this paper and the characteristic behavior of hurricanes affecting Texas is, simply, that hurricanes characteristically are notorious nonconformers to model. Many of our foremost scientists have spent years and years studying historical hurricanes but have been unsuccessful in their efforts to construct a wholly dependable model. After so much time and study someone attempts to again define a new model, however, he is almost sure to see the model violated by perhaps the very next hurricane to form. One such model will be shown here today. The hurricane chosen for discussion is itself a violation of model.

Hurricane Genesis and Growth

A condition found to be most favorable for hurricane genesis is a situation in tropical seas where there is a pre-existing stable tropical disturbance over warm water and an outside influence subsequently arrives on the scene to make the disturbance unstable and trigger it into cyclonic action.[1] The Intertropical Zone of Convergence (ITC)[2] is always present and is itself strong enough to qualify as a tropical disturbance capable of spawning hurricanes when triggered by an outside influence. Two other disturbances capable of hurricane genesis when intensified by outside influences are "easterly waves" and "troughs" in the westerlies.

1/ Riehl, H., 1954: Tropical Meteorology, McGraw-Hill Book Co., New York, 392 pp.

2/ The Intertropical Zone of Convergence is the boundary between the Northeast Trade Winds of the Northern Hemisphere and the Southeast Trade Winds of the Southern Hemisphere. The two trade winds converge along the ITC.

The outside influence triggering the disturbance into cyclonic action could be a wind shear or a velocity surge in the trade winds (due to reinforcement of the Atlantic Ocean High Pressure Cell). Or, the pre-existing disturbance could be triggered into cyclonic action if it merged with another disturbance. Such could be the case if a stable trough in the westerlies met and merged with a stable trough in the easterlies, or, if an easterly wave intersected a portion of the ITC.

Once triggered into cyclonic action and made unstable, the disturbance will not necessarily intensify to hurricane force if any one of many conditions are not met. For one thing, the incipient hurricane is sure to be short-lived if the ascending air within the cyclone is not forced upward at a rate sufficient to carry great quantities of water vapor (moist air) to very high altitudes. Also, while converging air currents are necessary at low levels, diverging currents are of prime importance at high levels: the rising air must be allowed to escape. A pumping action is thus set up and a constant supply of moisture is made available. A portion of the moisture is converted into heat--the life-blood of any hurricane. As the ascending air cools, invisible water vapor is condensed into cloud droplets and the latent heat of condensation is thus converted into sensible heat. The kinetic energy of radial motion is also converted to tangential kinetic energy, and heat may be added to the system by descending air currents, but the main energy input is the latent heat produced from water vapor.[3]

When the input of latent or sensible heat is reduced by any means, a hurricane will soon weaken. When heat or energy is added, or when more air escapes from the hurricane at high levels than is fed into the hurricane at low levels, the hurricane will intensify. A comprehensive discussion of all

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3/ Dunn, G. E. and B. I. Miller, 1964: Atlantic Hurricanes, Louisiana State University Press, Baton Rouge, Ch. 7.

hurricane processes is beyond the scope of this short paper. However, some of the hurricane-genesis and growth "ingredients" already discussed are illustrated on Figure 1.

A Hurricane Model

The Hurricane Model in Figure 2 is a model adopted by "Project Stormfury"[4] personnel, a group of scientists engaged in hurricane-seeding experiments. An actual hurricane (Hurricane Esther, 1961) is shown in Figure 3 as it looked approximately one-half hour before a seeding experiment was conducted.[5] Note the non-typical features of Esther when compared with the Model: spiral rainbands are better developed south of the eye of Esther, and the coverage of middle and high clouds is extensive south of Esther.

Hurricane Cindy, September 16-20, 1963

Cindy, 1963, was destined to be dubbed non-typical even before she was born--for she was the first Gulf of Mexico hurricane in two years. Not since the devastating "Carla," in 1961, had the Texas Coast been crossed by one of these violent maidens of nature.

Cindy first attained hurricane force a bare 200 miles out in the Gulf from Corpus Christi. She had a relatively short stretch of warm water over which to move and to obtain her strength and energy--namely, the moisture containing the latent heat which would ultimately be converted to sensible heat during the condensation process. Barring the possibility of remaining nearly stationary for a long time, Cindy was on a suicide course from the moment she was born; for her birthplace was too far north, too close to land, and too near the

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[4] "Project Stormfury" is a joint U.S. Weather Bureau-U.S. Navy program of scientific experiments designed to discover and test methods of modifying hurricanes. An initial three-year interdepartmental agreement launching the program began on July 30, 1962.

[5] National Hurricane Research Project Report No. 60, 1961: A Cloud Seeding Experiment in Hurricane Esther, 1961, U.S. Weather Bureau, Washington, D.C., 30 pp.

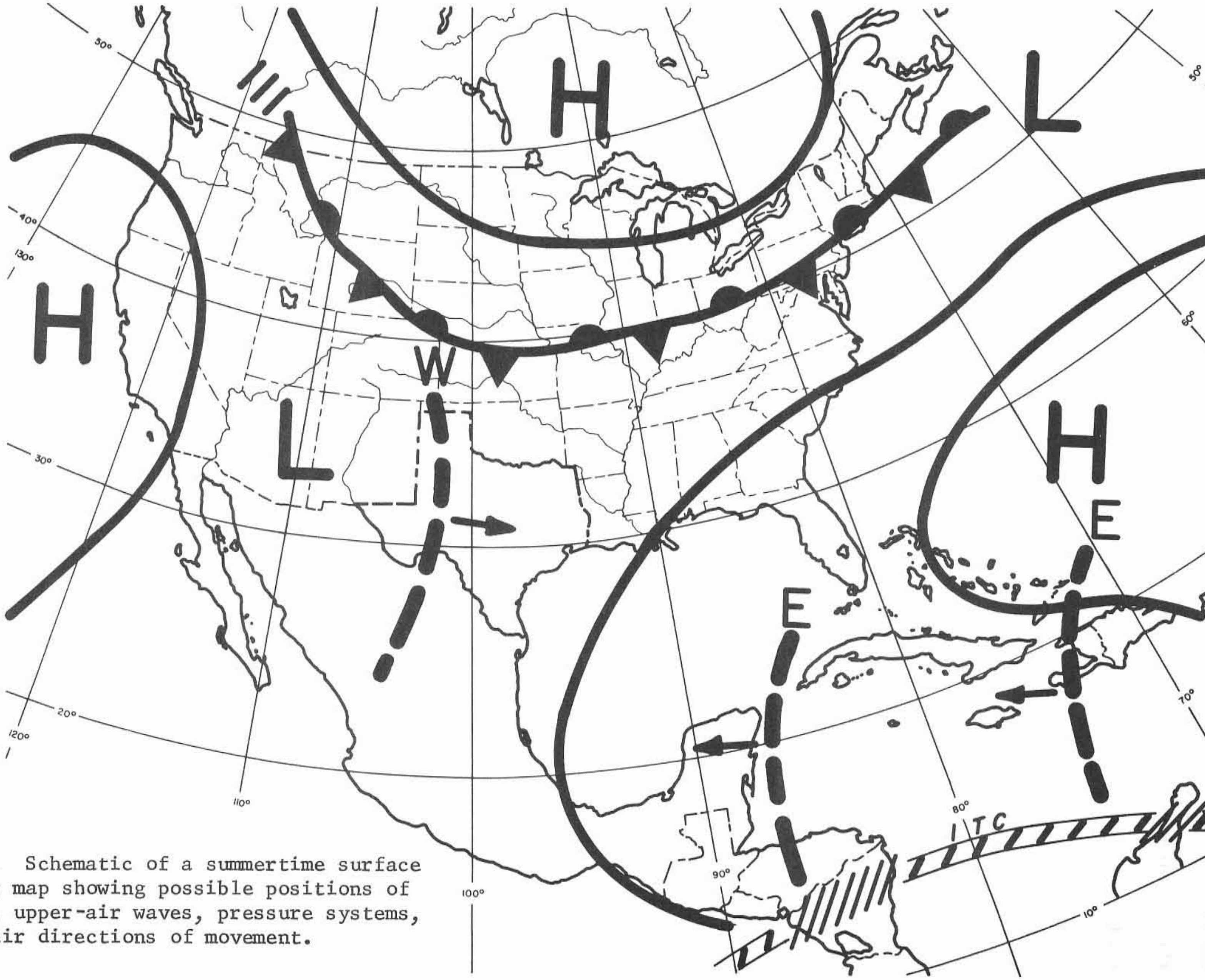


Figure 1. Schematic of a summertime surface weather map showing possible positions of fronts, upper-air waves, pressure systems, and their directions of movement.

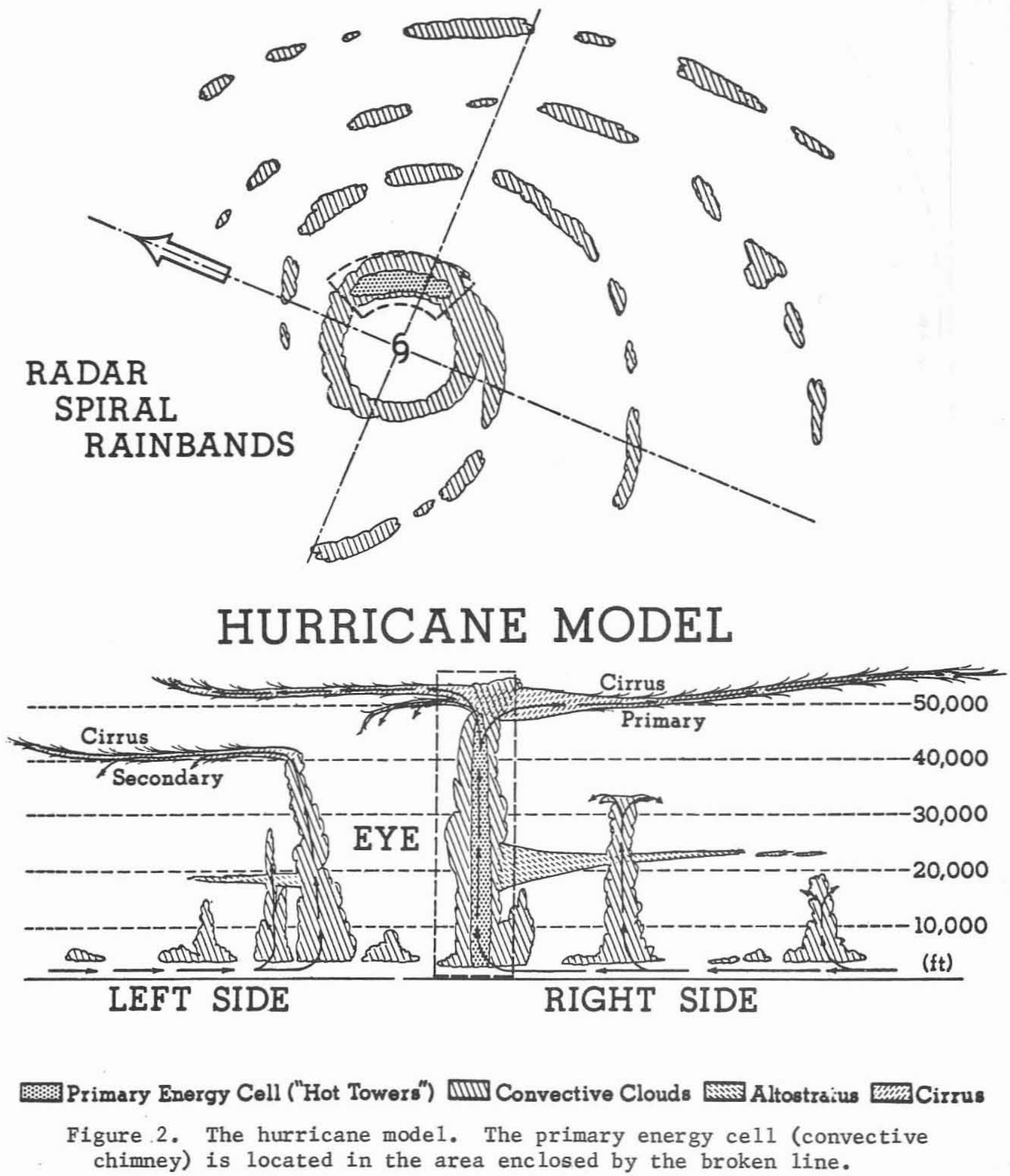
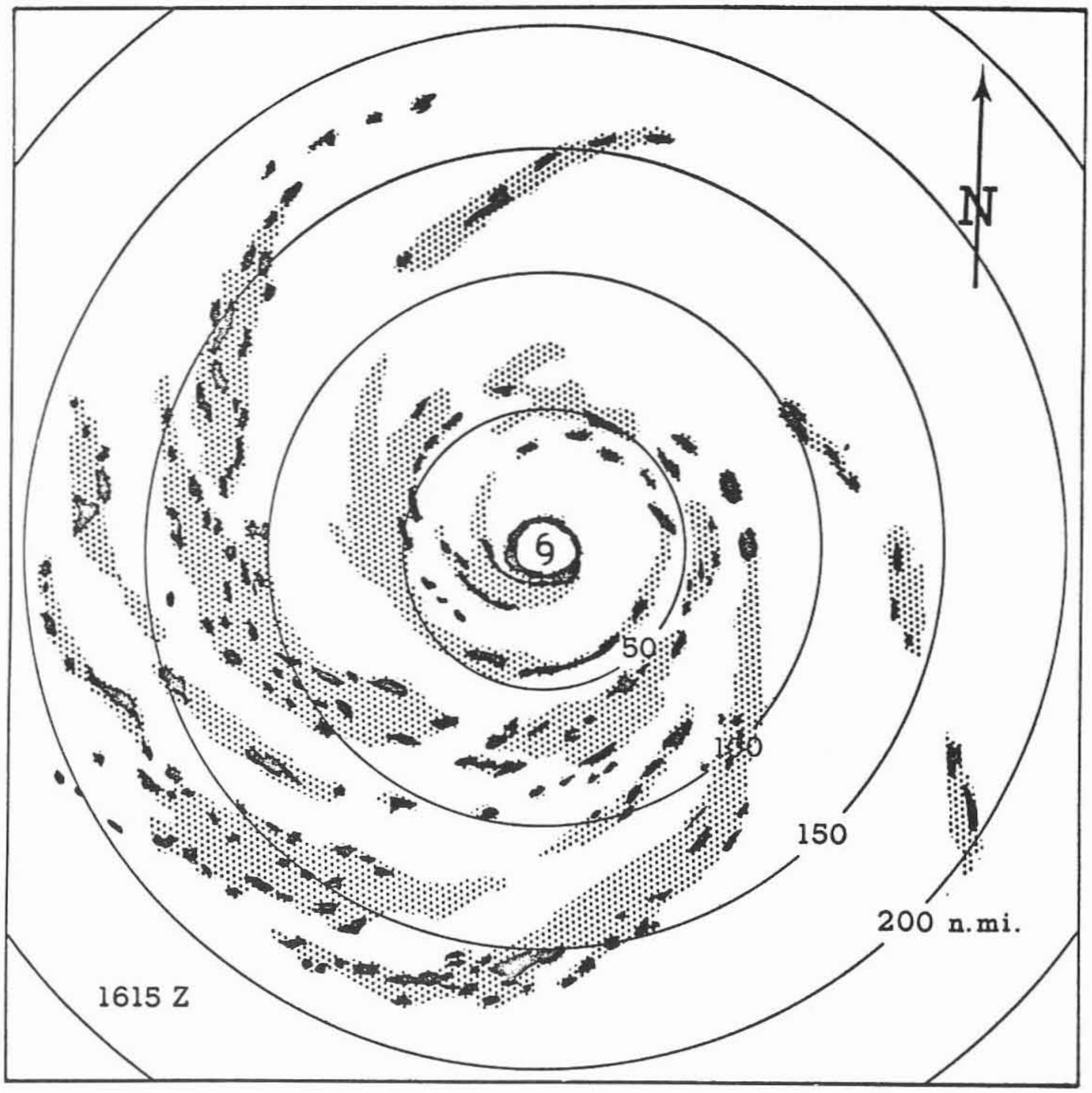


Figure 2. The hurricane model. The primary energy cell (convective chimney) is located in the area enclosed by the broken line.

(From NHRP Report No. 60, Hurricane Esther, 1961)



Radar composite of Hurricane Esther, 1615 GMT, Sept. 16, 1961.
(WV-30 aircraft, APS-20E, 10-cm. radar)

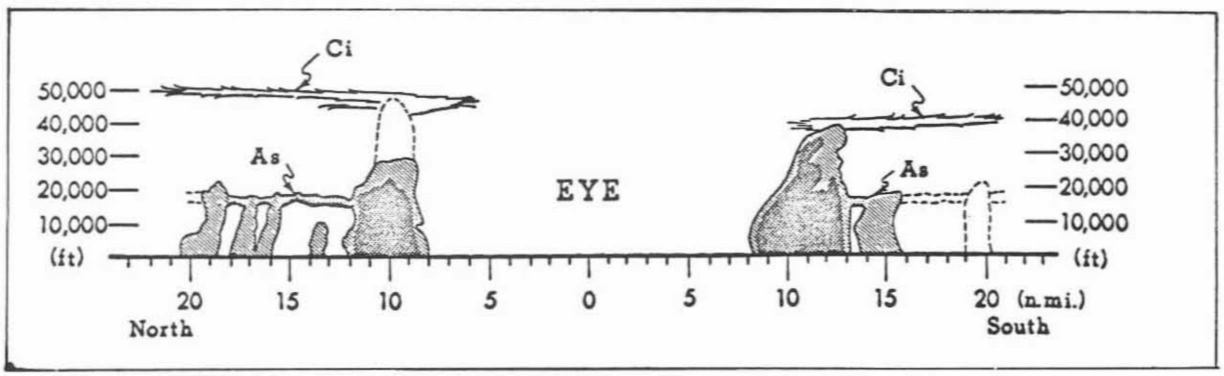


Figure 3. Cross section radar composite of Hurricane Esther, 1945 GMT, Sept. 16, 1961. (DC-6 aircraft, RDR-1, 3-cm. radar)

(From NHRP Report No. 60, Hurricane Esther, 1961) edge of the westerly winds of the temperate zone. In September these temperate zone westerly winds had already begun to migrate southward. Her path became as erratic as the path of a chip of wood when tossed into the river near a counter-current. As the chip nears the counter-current, it drifts first in one direction and then the other until it finally crosses the boundary of the two currents and is firmly in the grip of one or the other.

As shown in Figure 4, Cindy's path was at first northward; then she weakened as she neared the coast line and began to draw dry continental air into her circulation. But as she crossed the coast line, it was as if she were a living thing and knew instinctively that she would surely die if she didn't get back over the water. Cindy remained almost stationary for about 18 hours shortly after moving inland. In a vain attempt to cling to life, she seemed to try to turn back, but only succeeded in turning southwestward, quickening her forward speed, and drawing more and more dry air into her system as she paralleled the Texas Coast, but, alas, remaining inland and out of reach of her life-blood--the warm waters of the Gulf.

Cindy, 1963, left her mark, however. As reported in the U.S. Weather Bureau National Summary, September 1963, Vol. 14, No. 9, Cindy caused 80 mph sustained winds over the Gulf and 80 mph gusts as she crossed the coast line early on the morning of September 17, 1963, near High Island, about midway between Galveston and Port Arthur. Her slow movement resulted in an extended period of heavy rainfall over southeast Texas and southwest Louisiana. Rainfall totals were 15 to 20 inches in portions of Jefferson, Newton, and Orange Counties, Texas. Deweyville, in southern Newton County, had a 3-day total of 23.50 inches, 20.60 inches of which fell in a 24-hour period. Flood damage from high tides was comparatively light, but flooding due to the heavy rains caused water to enter about 4,000 homes. Property damage was estimated at \$11.7 million and crop damage was about \$500,000 in Texas alone. Two small

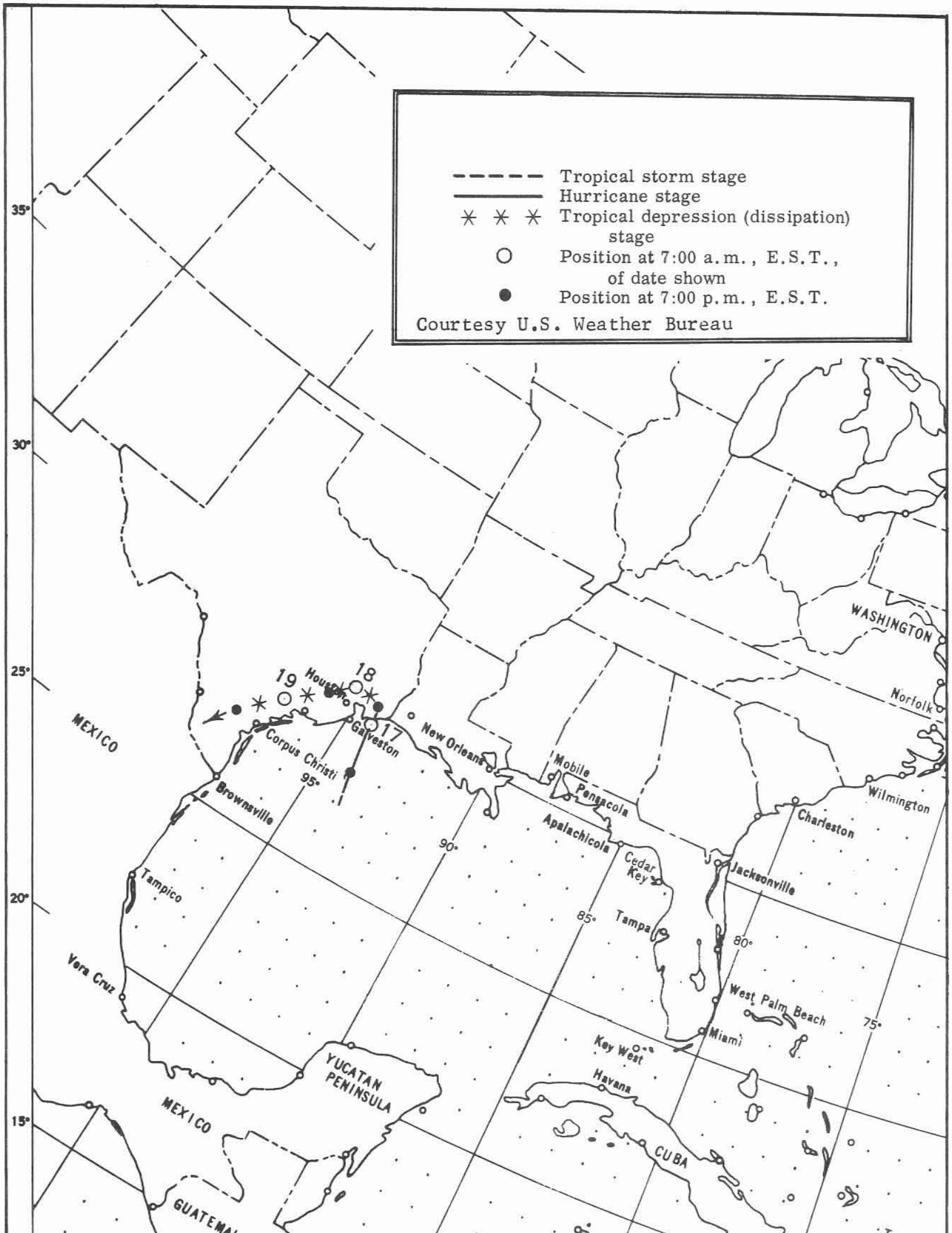


Figure 4. Hurricane Cindy, September 16-20, 1963

twin sisters drowned at Port Acres on September 22 in the persistent flood water still covering the area.

Hurricane Alice, 1954 Season

On June 25, 1954, Hurricane "Alice" entered Mexico about 85 miles south of Brownsville. This hurricane was classified as being of minor intensity. Alice, 1954, subsequently traveled up the Rio Grande Valley to the Lower Pecos and Devils River watersheds where her rains caused flooding which killed 17 people. Later in today's program Mr. Vance Myers will discuss the floods produced by this hurricane when he speaks on the subject of transposition of large storms over various size watersheds.

Conclusions

Hurricanes may form over warm water in low latitudes when a pre-existing tropical disturbance is acted on by an outside influence. After formation, hurricanes may not intensify unless special conditions are met. Hurricanes sometimes form, move inland, and become extratropical too rapidly to be studied except in retrospect. A small, short-lived hurricane can be the cause of very heavy rainfall and extensive flooding as it moves inland.

CHARACTERISTIC METEOROLOGY OF SOME LARGE FLOOD-PRODUCING STORMS IN TEXAS--THUNDERSTORMS

by

Carl W. Morgan[a]

Abstract

The mechanism whereby thunderstorms are produced is discussed. Two examples of thunderstorm activity which have given heavy rainfall in Texas are described.

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Introduction

Thunderstorms are weather phenomena of relatively small areal extent and exist in one location for only a short period. Lightning and thunder, usually gusty surface winds, heavy rain, and occasionally hail are characteristic. According to H. B. Brooks (1946) and others, the most frequently observed diameter of thunderstorms in the United States is about 3 km, and the average diameter is about 8 km. Thunderstorm cells may occur in groups or families to form squall-line thunderstorms which may extend over many miles.

Thunderstorms are characterized by great vertical extent. The base of these thunderstorm clouds is most frequently as low as 1.5 km above the ground, and their top is often well above 7 km.

Thunderstorm Mechanism

The thunderstorm represents a violent and spectacular form of atmospheric convection. Byers (1949) describes it as a cumulus cloud gone wild. Only a small number of cumuli continue their growth to attain thunderstorm proportions. The cumulus occurs as a result of heating from below in any air-mass with relatively steep lapse rate. Since the direct heating by way of the ground is the main cause of strong convection, thunderstorms have their maximum frequency in the afternoon. In addition to thermal instability the lifting of air can be accomplished by topography, fronts, and isobaric convergence.

By whatever mechanism the lifting is produced it provides thermodynamic cooling upon expansion as the parcel is moved to a lower pressure. This rate of cooling is known as the dry adiabatic lapse rate (-5.4°F/1000 ft) (Figure 1). If the air contains moisture in the form of vapor then at a certain height

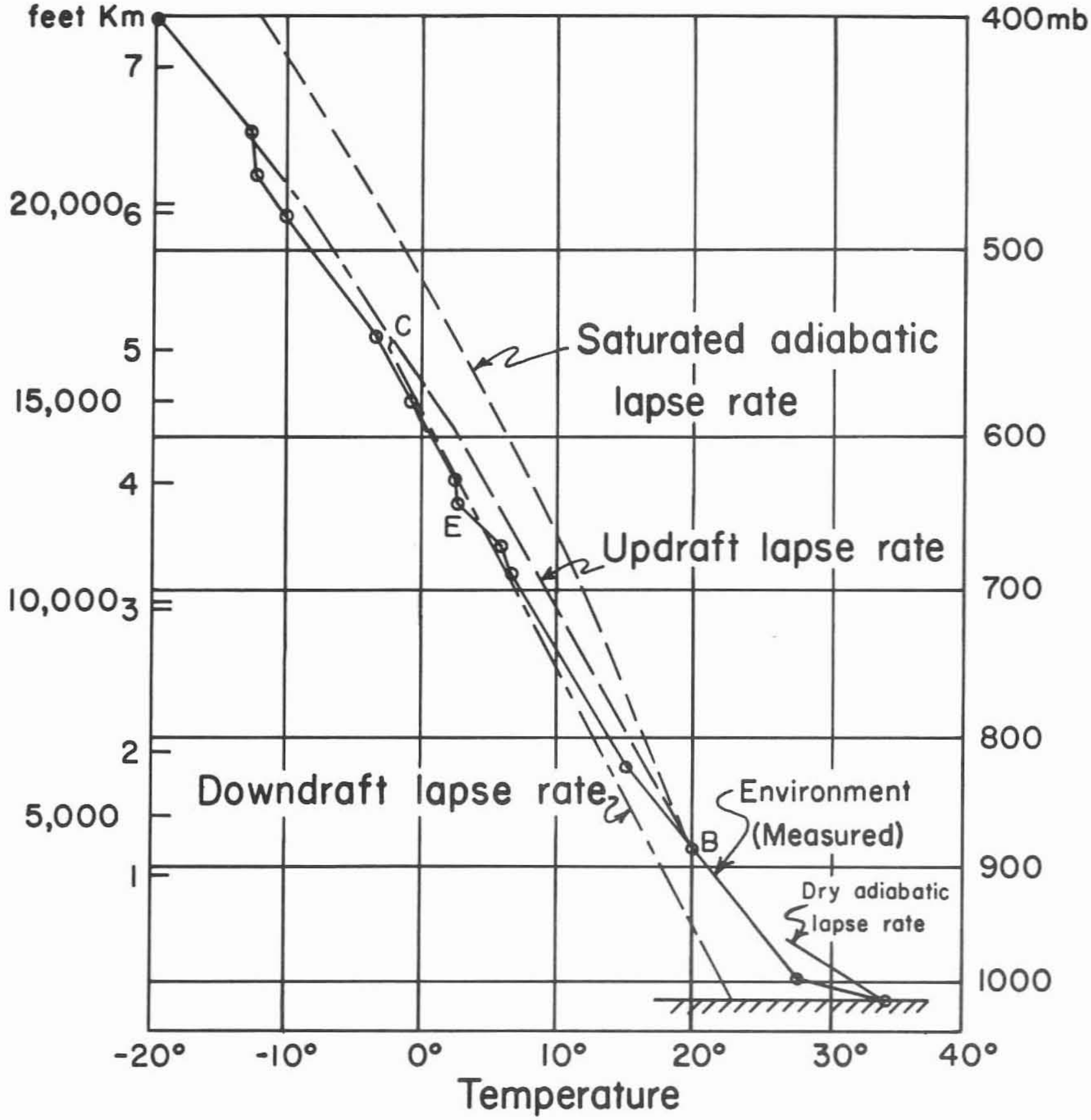

Figure 1. Thermodynamics of Updraft and Downdraft, (Byers and Braham, 1949)

condensation takes place producing cloud droplets and a cloud begins to form. Further lifting will produce additional condensation releasing the latent heat of vaporization. This heat reduces the cooling rate of the ascending air producing the saturated-adiabatic lapse rate of approximately -3.0°F/1000 ft. This lapse rate of the parcel may be less than that of its environment and it will arrive at a level warmer and lighter than its surrounding. The buoyant force will cause it to continue to rise, a condition known as instability.

As the top of the cloud rises, air from below takes its place and the cloud tends to grow. The upflow of air through the cloud may be very violent, condensation proceeds rapidly, the cloud droplets increase in size, may form soft hail, be covered with more water and finally build up to sufficient size to fall through and out of the clouds.

Byers and Braham (1949) in their excellent book The Thunderstorm state that the life cycle of the thunderstorm cell is divided into three stages depending upon the direction and magnitude of the predominating vertical flow. They are: (1) Cumulus stage (2) Mature stage and (3) Dissipating stage.

The cumulus stage is characterized by updrafts that extend throughout most of the cloud. (Figure 2) It should be noted that velocity, temperature, and hydrometeor distribution have been represented in a symmetrical manner for simplicity although such symmetry may not be realized. Maximum velocities occur at the higher altitudes late in the period and speeds of 50 ft/sec are not unusual. Mass continuity is maintained by the flow of converging surface winds and by horizontal inflow through the sides of the cloud. The quantity of water and size of the water particles in the cloud are small at first but continually increase with time. When the size of individual drop or ice particles increases to such an extent that they can no longer be supported by the existing updraft they begin to fall relative to the earth. The drag on the ascending air by the precipitation aids in producing a downdraft adjacent to the continuing portion

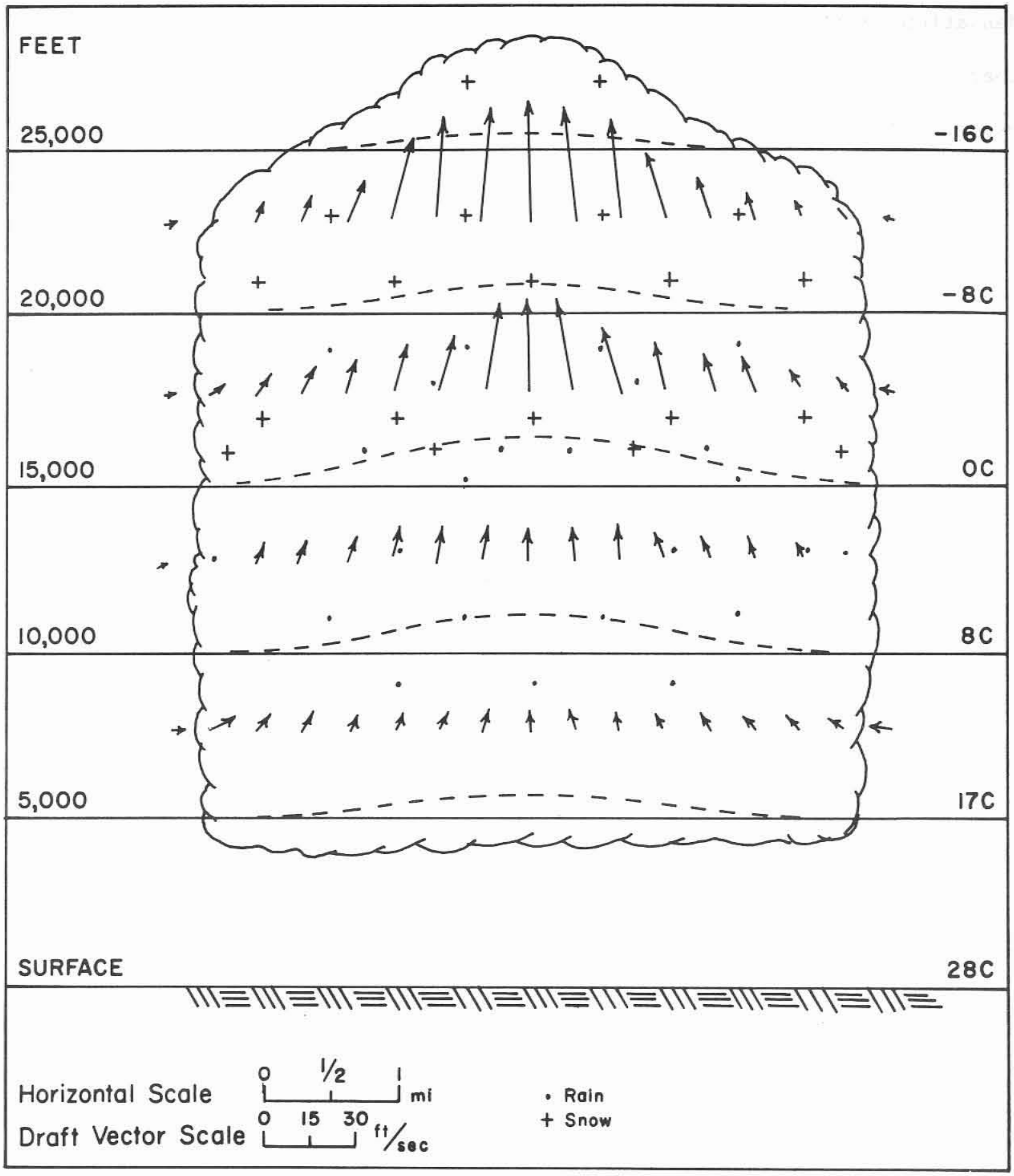

Figure 2. Cumulus Updraft, (Byers and Braham, 1948)

of the updraft. The downdraft has its greatest horizontal extent in the lower levels of the cell. The occurrence of rain at the earth's surface is the identifying feature of transition to the mature stage. Figure 3 shows a vertical cross-section through an average thunderstorm cell at a time approximating the middle of its mature stage.

The downdraft through the spread of its momentum and creation of new areas of descending air by the precipitation falling from the remaining updraft, spreads rapidly over the entire area of the cell at successively higher and higher altitudes. During the dissipating stage (Figure 4) this process continues until there is only a downdraft of air with little or no vertical motion and with light surface rain.

Thunderstorms in Texas

Let us consider two examples of thunderstorm activity which have given large rainfalls in Texas.

Storm of May 31, 1935, near D'Hanis, Texas:

The D'Hanis storm was one of the most intense small-area short duration storms of record. Its production of 22 inches in 2 hours and 45 minutes is a world's record for that time (Jennings, 1950). This storm was of the thunderstorm "cloudburst" type resulting from a northward-flow of moist tropical air from the Gulf of Mexico which underwent convergence as the isobaric pattern changed from anticyclonic over the Gulf of Mexico to straight over Texas.

An important contributing cause of the heavy rain appeared to be the lifting of the tropical air by the orography of the region. A few miles above D'Hanis, Texas, the Balcones fault zone exists and therefore the rise in the land elevation in the general storm area is rather abrupt.

On the early morning of May 31, the upper air charts showed a tongue of moist air, the axis of which was located just to the west of San Antonio, protruding from the Gulf over the storm area where the greatest amounts of moisture

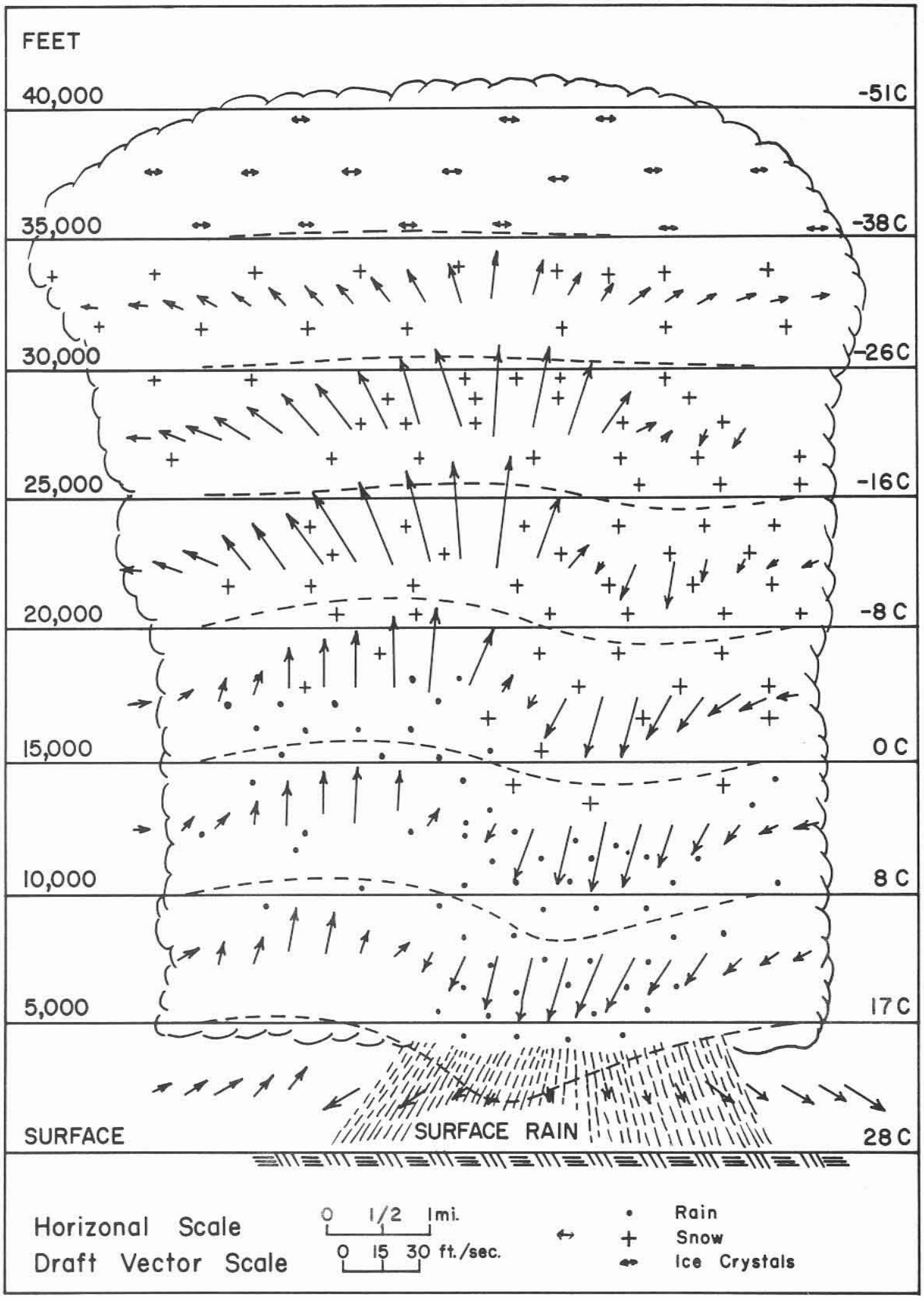

Figure 3. Mature Stage Updraft and Downdraft, (Byers and Braham, 1949)

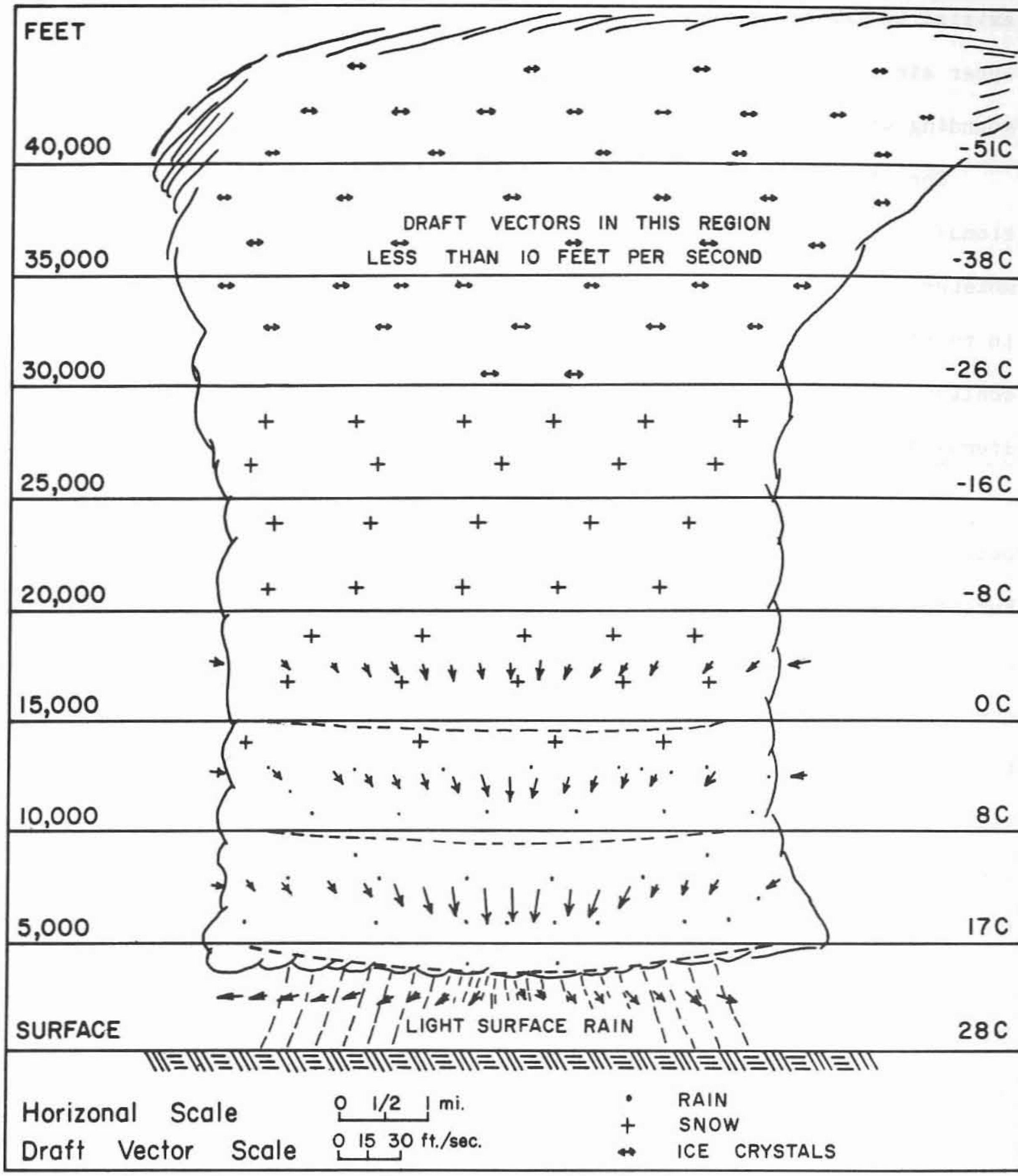

Figure 4. Thunderstorm Cell in its Dissipating Stage of Development, (Byers and Braham, 1949)

existed in the atmosphere. The air was convectively unstable as shown by the upper air soundings at Oklahoma City, over which the Gulf was flowing. No sounding was available at any station in Texas.

The lift the air received during its travel from the Gulf plus the additional lift over the Balcones escarpment was critical from the lapse rate and moisture distribution that existed on the early morning of May 31, and served to release the latent energy and excessive rainfall over the Seco Creek and contiguous area.

Storms of May, 1957, North Central Texas:

The weather of May, 1957, was indeed turbulent. More tornadoes were observed in the United States during May, 1957, than any other month of record. Furthermore, record number of tornadoes were reported for any one week, May 20-26, and for any one day (Dunn, 1957). The recurrent heavy rains produced frequent and severe flooding in parts of Texas, Oklahoma and neighboring areas to the east and northeast. Of particular interest are the moderate to heavy convective-type rainfalls of May 12-13, 1957. Surface dew points ahead of a cold front suggested an ample supply of moisture. The air was convectively unstable and laden with low level moisture. At the surface the air parcels originating along the Texas gulf coast began areal convergence along a line just south of Junction and Waco, which agreed reasonably well with the southern boundary of the precipitation area. The maximum reported rainfall, 5.05 inches, occurred near Waco. The area northeast of Fort Worth experienced tornadoes, hail and up to 3-1/2 inches of rain (Cole and Lowry, 1957).

The lifting processes causing the rain were interrelated, but the main contribution apparently resulted from low level convergence induced by wind shears and changes in curvature.

Conclusions

The thunderstorm will produce high rates of rainfall. The meteorological conditions required for its formation are often fulfilled over Texas. Very moist tropical maritime air masses carried by the southeasterly winds moving inland up the land slopes and up the slopes of cold fronts and continental air masses or by convection may become unstable and proceed through the full cycle of the thunderstorm.

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CRITERIA AND LIMITATIONS FOR THE TRANSPOSITION OF LARGE STORMS OVER VARIOUS SIZE WATERSHEDS[a]

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Introduction

Our most important clues to the storms of the future are the storms of the past. Storms that threaten Texas have been described in previous papers. This paper continues the story by discussing some problems--and solutions--in using storms of the past to draw specific conclusions about future storm potential over specific basins. The paper draws on the experience of the Hydrometeorological Branch of the Weather Bureau. However, most of the principles and problems described are of general application.

The Problem

Eleven years ago almost unbelievable flows of water came racing down the draws and arroyos that drain into the Devils River and the lower Pecos River, and on into the Rio Grande immediately above Del Rio. This was the combined effect of Hurricane Alice, of other meteorological factors, and of topographic factors. Mr. Carr has already discussed this storm.

The flow at the mouth of the Pecos of almost one million cfs was eight times any previous flow during a long record. The maximum annual flows at Comstock are listed in Table 1 and plotted in Figure 1. Some of the maximum annual discharges are affected by diversions upstream of the gage, but this accounts for only a small part of the difference between the maximum flood and the prior floods.

This is but an extreme illustration of the commonly recognized fact that storm experience over a single basin alone is not a dependable indicator of what might occur over that basin in the future. The Pecos record through 1953 gives no hint of what was to come in 1954.

Every few years somewhere in the United States there is a flood which far transcends the previous experience of the local people. Last year (1964) central and western Montana was the victim. In early June, as an unusually heavy snowpack was going into full melt, an intense low passed over the area; rain poured for 30 hours. Downstream from mountain valleys the prairie became a sea.

In 1950, a hurricane passing over northern Florida stalled, looped, and hung around for 3 days. Forty-five inches of rain was collected at one place (near Yankeetown); all previous station records in the United States from 18 hours up to 3 days were exceeded.

What nature seems to be showing is this:

First, storm experience over most basins is wholly inadequate to reveal what could happen over that basin. Flood records are broken all the time, by wide margins.

Second, the known storms in the United States as a whole (over the last half century or so) form enough of a consistent pattern that we are confident that the principal storm types have showed up. We do not really expect any completely new revolutionary types of storms.

Third, even if storm types and characteristics are known, the biggest observed rainfalls within large regions are not the ultimate. Regional rainfall extremes will continue to be exceeded, though generally not by wide margins.

Tasks, then, in design of a spillway against a probable maximum flood include:

(1) Seek out the major storms of the region, whether they are over the basin or not,

(2) Move them to the basin, directly or indirectly, and

(3) Accept them as prototypes of the kinds of storms that could occur in the future, but adjust them upward on a rational basis, closer to what is thought to be nature's maximum potential.

For design against a "standard project" flood the last step would generally be omitted.

We will discuss the second step, which goes under the name of transposition, and the third step, which is called maximization.

Transposition

Transposition may be approached in two ways. One can look at a storm and decide where it can be transposed without changing its character, as a device in analyzing the rainfall climate. This is called setting transposition limits. Or one can start with a basin and decide from a selection of extreme storms, which can be moved to it. We will take up examples of each approach by way of illustrating the kinds of judgments that must be made.

First example. The largest western Gulf of Mexico hurricane rainfall of record, over 5,000 to 10,000 square miles in 24 hours, was in an August 1940 storm centered in southern Louisiana. The total storm isohyetal map is shown in Figure 2.

To where in Texas is this storm transposable?

The first step in setting transposition limits is to identify the storm type. No problem here: a hurricane.

The second step in setting transposition limits is to outline the area in which this storm type has been experienced. Again, no problem. It is well known that hurricanes frequent the entire Texas coast.

The third step is to look for particular characteristics of the individual storm that might affect its transposability. Review of the record shows that the Louisiana storm was not a full-strength hurricane but was weak. Wind damage in the storm was negligible.

Another important characteristic is found from the surface weather map for 0030 CST August 7, 1940, Figure 3. Hurricanes and tropical storms are notorious heavy rain producers when they collide with either mountains or fronts. Here we find a cool High immediately north of the little hurricane which is entering the coast near the Texas-Louisiana border. The interaction of air masses is too diffuse to warrant drawing the front from the eastern United States, on back into the north part of the hurricane circulation. But a collision of cool air diverging from the High and the hurricane circulation is nonetheless present and is suggested by arrows on the diagram.

The fourth step is the delineation of the transposition limits which the particular storm characteristics require. This is generally done subjectively and by deduction.

Figure 4 depicts the approximate transposition limits of storm of Aug. 6-9, 1940. It has been the practice of the Hydrometeorological Branch to restrict the rainfall center of strong hurricanes to within about 50 miles of the coast, as this is where they usually occur. But this weak storm is equivalent in intensity to a more severe hurricane that has weakened over land for some time. Therefore the rain center is considered transposable farther inland, up to about 150 miles from the coast.

The drift of the modified polar air into the northern part of the storm from the east, shown in Figure 3, is also taken into account. We can envision such a circulation along the Texas coast to about Corpus Christi. South of there, such an easterly drift would have to come from over the Gulf of Mexico. The season is August. The warming over the water would diminish the effectiveness of the differences between the air from the High and that from the hurricane circulation. We therefore exclude this particular storm from transposition south of Corpus Christi. Heavy hurricane rain can fall there all right, but other storms supply better evidence than this one.

Second example. Now for another example. A famous intense storm was centered at Warner, in eastern Oklahoma, early in May 1943. The rainfall mass curve is found in Figure 5. In the annals of storms in Texas and adjacent states this storm is overshadowed by summer storms. However, if probable maximum precipitation were to be estimated separately for each month of the year, as is sometimes required, this storm would provide the largest late spring transposed values in North Texas, for some sizes of area and durations.

Now to run through the four steps again. First, the weather map (Figure 6-surface weather map 1830 CST May 9, 1943) shows the rain was centered northeast of the crest of a wave on a front, a very characteristic location for a heavy rainfall center.

Second, this general storm type of open wave on a front is a very common one at middle latitudes. It may certainly be expected anywhere within Texas or adjacent states.

Third, the particular characteristics of the storm affect transposability: The weather map shows that the front was very strong--that is, the horizontal temperature gradient in the air nearby is large. The dashed lines are isotherms--lines of equal temperature. The packing of the lines parallel to the front shows a strong temperature gradient, for example 84°F ahead of the front near Fort Worth dropping to 48 degrees in the Panhandle.

A second characteristic is the strong air flow from the Gulf of Mexico to the rain area. This is indicated by the closely spaced north-south isobars--the solid lines. They run approximately along the wind direction. This type of flow is very characteristic of heavy rainstorms anywhere in the central United States. A third characteristic is the Mexican Low, into which the front trails. Upper-air charts (not shown) reveal an upper closed Low near here.

To apply our fourth rule, what do these conditions mean as to the transposability of the storm? First, the temperature contrast means that the transposed storm must be placed far enough away from the Rocky Mountains to permit a good thrust of cold air southward unobstructed by the mountains. The inflow from the Gulf restricts us to locations exposed to southerly flows from the Gulf. These two conditions place a western limit to transposability (Figure 7).

The placement of the southern limit to transposition is more subtle and more difficult. It is felt that the associated Mexican Low cannot be forced too far south into the mountains of central Mexico and that the circulation shown in the weather map is not characteristic of the immediate coast. Therefore the transposition limit is placed as shown in Figure 7. This does not mean that there are not extremely heavy coastal rains in association with fronts but rather that the evidence for these lies more clearly in other storms.

Reference to previous paper. Mr. Orton has presented most of the information that would be necessary to set transposition limits for the September 1952 storm--the storm type, the particular characteristics, and where this type is found. Because of the apparent role of the Balcones Escarpment, transposition would be limited to the escarpment, or an area with similar opportunity for triggering of a storm by the terrain. The type--easterly wave--would limit the transposition northward, and the necessity for inflow from the Gulf would limit the transposition westward.

Last example. In our last example we start not with a storm but with a basin. See Figure 8, storm transposition to Colorado River Basin above Fox Crossing. The seven storms indicated on the figure, and a few other weaker ones, were transposed to the Colorado River Basin above Fox Crossing for the purpose of estimating the probable maximum precipitation over the entire Basin, and also over some subsidiary basins within it. These transpositions were proposed by a District Office of the Corps of Engineers and checked by the Weather Bureau. It was determined that each of these storms would be controlling, or near-controlling, in the final array of probable maximum depth-duration-area values; then each transposition was examined in the light of the characteristics of the storms and their weather maps in the manner that has been illustrated, and each was found valid.

We will make at this point only one comment on these transpositions. The Cheyenne, Oklahoma storm resulted from frontal waves somewhat like the Warner storm referred to earlier. Cheyenne is at an elevation of about 2000 feet. The ground provides some lift to tropical air reaching this location. It is thought that this lifting by the ground and releasing of instability may have been significant factors in the storm; therefore transposition is limited to locations at about 1,000 to 3,000 feet elevation and situated to provide similar lifts. The Colorado Basin above Fox Crossing fulfills these requirements.

It should be emphasized that the solution of spillway design floods by transposition of storms is not always as easy as this diagram would imply. Texas, along with all its other superlatives, has experienced a number of outstanding rainstorms. One of the storms of the State, centered at Thrall, Texas in 1921, provides the greatest volumes of record in the United States, for certain sizes of area and durations. Sometimes it is necessary to accept more dubious transpositions than these in order to obtain an adequate sample of large storms.

Sets of Storm Values

In a typical spillway design problem, critical depths of runoff, and therefore of precipitation, are required for a succession of duration increments, such as 6 hours. Such precipitation increments are available by subtraction if we have the maximum total for 6 hours, for 12 hours, 18 hours, for one day, etc. In order to find these maximum values by a transposition technique, we need to consider a spectrum of storms. A storm that provides the extreme value for 6 hours will not necessarily provide it for 24 hours. If, as in the Fox Crossing problem, extreme precipitation for a variety of basin sizes as well as a variety of durations is required, then this further expands the set of controlling storms. This is illustrated by a table which names the controlling transposed moisture-adjusted storms for the Colorado Basin above Fox Crossing, for areas from 1,000 to 50,000 square miles and durations of 6 hours to 3 days. See Figure 9. Final values are always smoothed over both duration and size of area, thus providing another enveloping step to compensate for lack of uniform storm experience at the various durations and area sizes.

Maximization

Introduction to this section. Transposition usually takes care of our second premise in deriving a spillway design storm, namely that the history of storms in the United States contains the principal storm types and characteristics. It does so of course by bringing these storm types and characteristics to the basin. The third premise is that, while we do not expect revolutionary new storm types, we do expect some new storm magnitudes to exceed previous records. If the probable maximum precipitation over a basin is the project requirement, then we must find a rational basis for maximizing the transposed storms.

Rainfall and moisture. Rainfall intensity is related to many factors, but it is clear that one very important factor is the concentration of water vapor in the atmosphere, especially in the lower levels. Some examples will illustrate this. In West Texas the surface air dew points are low and precipitation is usually absent or light. On those occasions when heavy rain comes to West Texas, it will be found that the winds have turned to Southeast and that the high dew points common along the coast and in East Texas will have penetrated far inland.

Over the south-central United States nasty weather with precipitation can prevail over vast areas in winter with the dew point on the Gulf Coast in the low 50's (°F). But for a real flood-producing storm in the south-central United States in winter the coastal dew points will rise above 70°F, not far below summer values.

These facts and many others lead to the following theory: If we can survey a considerable number of outstanding rainstorms, it is likely that one or more will come fairly close to displaying a probable maximum mechanism. By mechanism we mean the velocity with which moist air enters the storm area, the vigor with which it is lifted to great heights, thus forcing the release of rain, and the atmospheric conditions that produce these horizontal and vertical velocities. If then we process through this storm mechanism of near-maximum efficiency, the maximum moisture that can be expected for the season and region instead of the actual moisture in the observed storm, we obtain, by computation, precipitation depths closely approaching the probable maximum. Note that an inherent part of this theory is that a large number of outstanding storms must be surveyed, in order to have a good probability of obtaining one of near-maximum efficiency.

This theory is applied in a simple manner. Observed storm rainfall depths over various sizes of area for various durations are multiplied by a ratio. The ratios are contained in tables and depend on the moisture observed in the warm air current flowing into the storm, and the maximum expected moisture at the same location. Dew point is the usual index of such moisture.

The observed storm dew point comes from surface weather maps--dew points at appropriate stations are averaged. The maximum dew point is scaled from maps which envelope values of station dew points from a long record, all reduced to the same elevation. Figure 10 is an example of maximum enveloping dew points during the month of May.

The theory of the moisture adjustment is more complex than its application. Figure 11 depicts schematically the basis for the moisture-adjustment ratio. In major rainstorms moist air rises from within a few thousand feet of the ground through clouds of great vertical development, depicted here by two cumulo-nimbus clouds. The rising air cools at a rate, called the moist adiabatic lapse rate, which is precisely known but varies with temperature. It turns out that the higher the specific humidity of air, the more water vapor it must give up by condensation in rising a given height through a cloud.

Another factor is the vertical development of the cloud. It is assumed from observations of cloud top heights and deduction based on tropopause heights and other factors that the higher the dew point the greater the cloud height that can result. An empirical cloud height vs. dew point relation has been adopted. It is further assumed that the depth of the inflow layer is proportional to the cloud height.

The total moisture adjustment is around 5 percent per degree Fahrenheit difference in dew point. Of this, about 3% is from the increase in yield of water per unit lift at the higher temperature, and 2% from the assumed greater volume of lifted air and greater lift. Figure 12 shows the lifts and resulting losses in specific humidity of the air that would be associated with a 60°F and a 75°F dew point.

A convenient simplification is possible. It turns out that the depths of rainfall calculated for various surface dew points by this lifting model is rather closely proportional to the precipitable water (vertically integrated mass of water vapor) between the surface and some great heights associated with saturated air at the respective dew points.

Thus the arithmetic for the standard moisture adjustment is simply the multiplication of observed rainfall depths for various durations and sizes of areas by the ratio of two precipitable waters, that associated with the observed representative storm dew point and with the maximum dew point.

Other maximizations. In the United States east of the Rockies it is common practice to base estimates of probable maximum precipitation on storm values maximized for moisture only, transposed and enveloped. If the transposable storms are too few either from limitations of the data or of the topography, then compensation must be found by applying other maximizations. In regions where lifting of air by mountains has a major influence on precipitation storms, storms can be maximized for wind as well as moisture if the rain-bearing wind direction is predictable and relatively constant. An example of a region where this is done is the west slope of the Sierra Nevada in California, where virtually all of the precipitation falls with westerly winds from the Pacific.

In certain regions of the world where data are very sparse--or where the only storm data available for a project are those from within the borders of one or two small countries--other kinds of maximizations must be devised to arrive at a probable maximum precipitation estimate. These are expedients and are not a substitute for good observations, collected over a period of years, from a wide area.

Relocation Adjustments

We said we would come back to the question of adjusting transposed storm values for relocation. This was held until now because of similarities to the moisture maximization adjustment just described.

As an example of a relocation adjustment...If you live a few miles from a city and need a minimum temperature forecast so as to know whether to cover your tomatoes, being a good customer of the Weather Bureau, you listen to the forecast for the city. You then allow for the fact that suburban temperatures on clear cold nights are generally lower than city minimum temperatures. The difference can range to 7 degrees or more. This allowance is a relocation adjustment. Or the allowance may be stated in the forecast by the Weather Bureau. It is still a relocation adjustment. That you accept the general weather situation as applying both to city and suburbs means that you are within the transposition limits of the city minimum temperatures, and apply a local modification.

In a similar fashion adjustments are applied to storms when they are relocated.

The first principal adjustment is for proximity to the moisture source. This is worked out in the same manner as the moisture maximization adjustment and is based on values on the maximum dew point charts at the respective locations.

Another type of adjustment is for barrier--that is for mountain intervening between the oceanic moisture source and the place studied. This is a common situation because many of the basins best adapted for the construction of dams are rimmed by mountains.

A transposition adjustment that we would like to apply, but generally do not, is for the available energy. For example rainfall along fronts is related to the intensity of the front; that is the strength of the horizontal temperature gradient. The maximum frontal intensity, defined in this way, that may be expected varies from place to place just as maximum moisture varies from place to place. If a reliable quantitative method could be developed for relating intensity of precipitation to frontal intensity or to some other horizontal temperature gradient factor, this would increase the number of storms that could serve as evidence of maximum storm potential over a particular basin. As yet experiments with this type of adjustments have not been very successful.

Solution of this problem would also facilitate seasonal transportation. For example, an estimate of probable maximum precipitation in June might be required for combining the maximum rainfall with maximum rate of snowmelt. Known storms in May could be more readily used as evidence of storm potential in June if they could be adjusted downward, in a more refined way than now available, for the seasonal trend toward decreasing temperature gradients.

Generalized Charts

Our final topic is generalized charts. This is the most up-to-date method of deriving basin estimates by transposing storms, yet it is not new. Bailey and Schneider published some generalized charts for use in spillway design estimates in Civil Engineering in 1939. Generalized charts of course involve a great deal of labor.

A generalized chart is a map on which are shown isopleths of probable maximum precipitation, or of some other category such as "standard project" rainfall, for a specified size of area and duration and, frequently, for a specified month. Such charts may appear in sets for various durations, areas, and months, which have been tested for smooth transitions from one chart to another.

An outstanding advantage of generalized charts is that it becomes unnecessary to develop transpositions to every basin. We may stick to the most definite and clear-cut transpositions and produce a network of values. These are then enveloped by isopleths which use the premise that the pattern is geographically smooth, or else that it is closely related to the topography.

Figure 12 is a generalized chart of hurricane 10,000-sq. mi. 24-hr. rainfall. This is a working diagram, not a final result--for a "standard project" type of estimate. It is therefore constructed by enveloping storms without maximization. This chart is what one gets by assuming that hurricanes are transposable anywhere as long as their distance from a generalized coast is maintained. The isopleths therefore parallel the coast.

A value in western North Carolina is slightly undercut because of strong local orographic influence, which the chart is not intended to depict. The Thrall, Texas storm--having some connection to a previous tropical storm but not a pure example of a hurricane--had 11 inches where this chart shows nine.

Finally, generalized values of probable maximum precipitation over 200 square miles in 24 hours are reproduced from a publication of the Weather Bureau, in Figure 13. Weather Bureau generalized charts of probable maximum precipitation now cover all states and territories under the U.S. flag up to basin sizes of 400 square miles. We are presently engaged in refining these results in certain western states and extending them to larger area sizes. The eastern United States is covered for basin areas up to 1,000 square miles.

Plans are not yet crystallized, but we hope to extend the eastern U.S. generalized charts of PMP up to basin sizes of 10,000 square miles or more.

Table 1

ANNUAL PEAK DISCHARGE PECOS RIVER NEAR COMSTOCK, TEXAS

Year	Date	Peak Discharge for Year 1000's cfs	Data Source
1900	Apr.6	107.0	a/
1901	-	9.20*	b/
1902	May 18	33.5	a/
1903	June 29	2.14*	b/
1904	June 27	72.0	a/
1905	Apr. 23	47.0	a/
1906	Aug. 11	90.0	a/
1907	Nov. 6	2.88*	b/
1908	July 7	68.0	a/
1909	Aug. 1	1.78*	b/
1910	Sept. 6	102.0	a/
1911	Apr. 4	27.0	a/
1912	Apr. 7	1.11*	b/
1913	May 4	63.0	a/
1914	Oct. 23	67.0	a/
1915	Apr. 22	52.0	a/
1916	Sept. 1	97.0	a/
1917	May 12	1.59	b/
1918	Aug. 15	7.14	b/
1919	Sept. 16	87.0	a/
1920	Oct. 4	5.22*	b/
1921	June 13	18.5	a/
1922	June 18	77.0	a/
1923	Sept. 17	1.50	b/

Table 1

ANNUAL PEAK DISCHARGE PECOS RIVER NEAR COMSTOCK, TEXAS--Continued

Year	Date	Peak Discharge for Year 1000's cfs	Data Source
1924	Sept. 22	12.8	a/
1925	May 28	61.0	a/
1926	July 23	4.38	b/
1927	June 13	14.6	a/
1928	May 13	19.8	a/
1929	Oct. 14	6.32	b/
1930	Oct. 14	20.1	b/
1931	May 18	2.62*	b/
1932	Sept. 1	116.0	a/
1933	Oct. 14	4.50	b/
1934	June 14	8.22	b/
1935	Sept. 4	84.4	a/
1936	Sept. 27	31.1	a/
1937	May 10	2.80	b/
1938	July 24	31.5	a/
1939	May 5	5.80	b/
1940	June 25	5.61	b/
1941	Sept. 18	18.7	b/
1942	Missing		
1943	July 15	11.2	b/
1944	Sept. 6	8.96	b/
1945	Oct. 7	27.7	b/
1946	Oct. 6	65.0	b/

Table 1

ANNUAL PEAK DISCHARGE PECOS RIVER NEAR COMSTOCK, TEXAS--Continued

Year	Date	Peak Discharge for Year 1000's cfs	Data Source
1947	May 11	6.10	b/
1948	July 4	51.3	b/
1949	July 26	98.5	b/
1950	July 13	44.9	b/
1951	May 24	8.18	b/
1952	May 27	3.57	b/
1953	Aug. 24	14.8	b/
1954	June 28	948.0	b/

* Maximum daily average discharge for year.
a/ International Boundary and Water Commission, U.S. and Mexico - Water Bulletins, "Flow of the Rio Grande and Related Data."
b/ U.S. Geological Survey, Water Supply Papers, "Surface Water Supply of the U.S. Part VIII. Western Gulf of Mexico Basins."

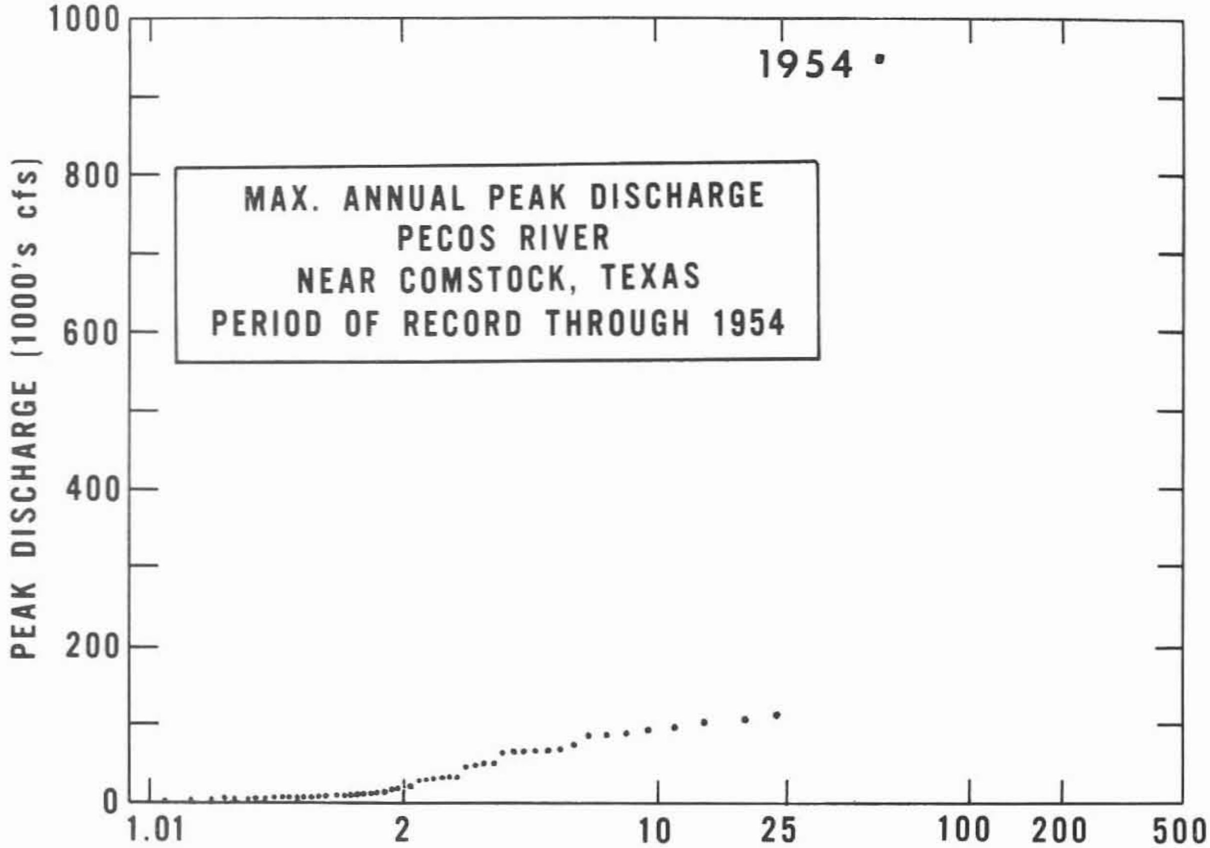

Figure 1. Return Period (Years)

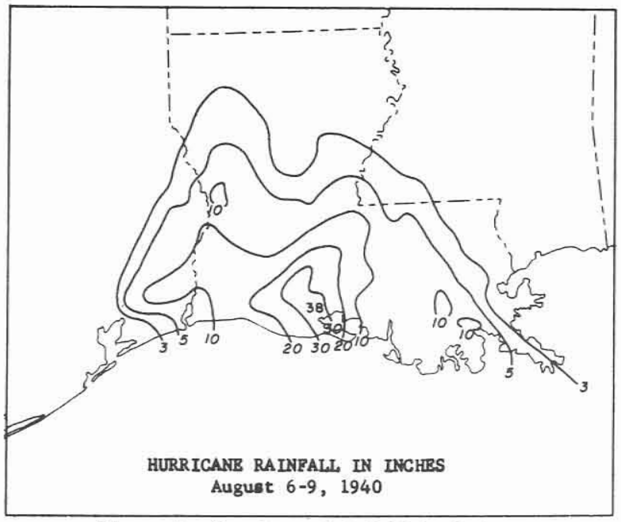

Figure 2. Hurricane Rainfall in Inches
August 6-9, 1940

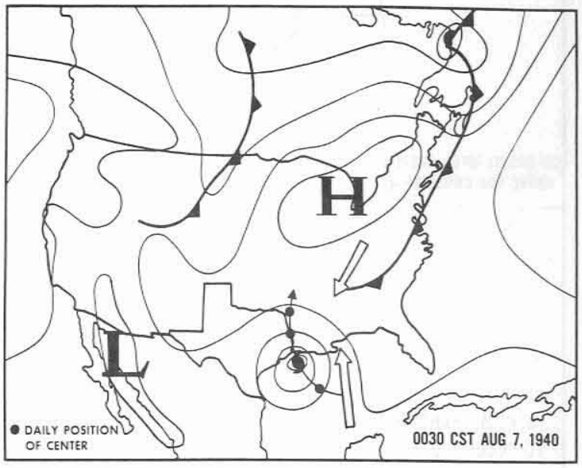

Figure 3. 0030 CST August 7, 1940

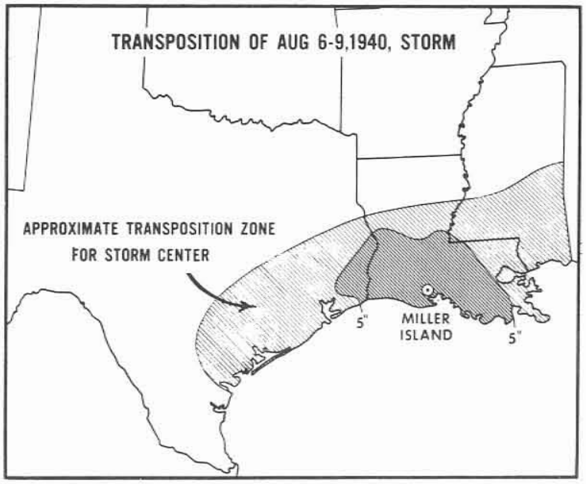

Figure 4. Transposition of August 6-9, 1940 Storm

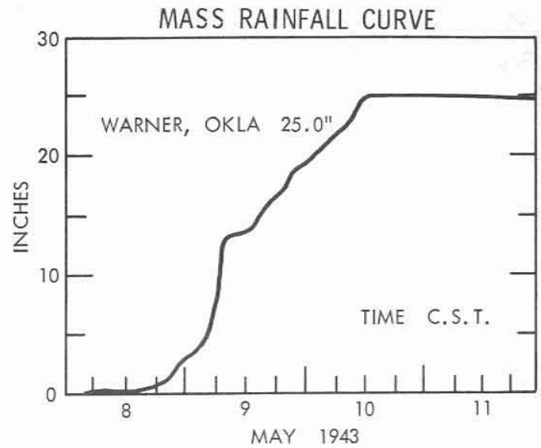

Figure 5. Mass Rainfall Curve

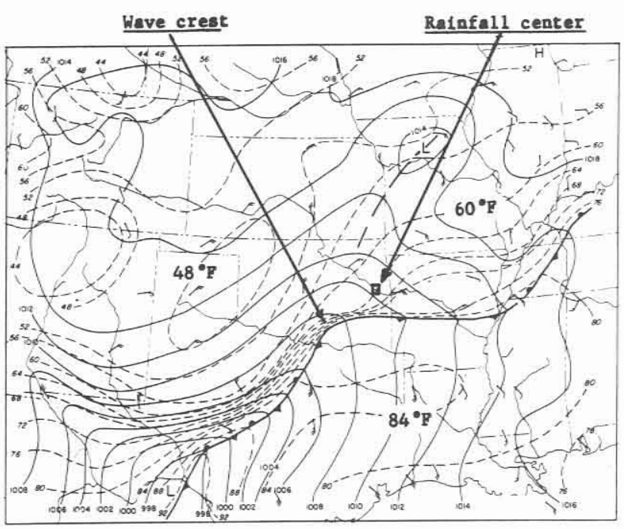

Figure 6. Weather Map for 6 PM CST May 9, 1943

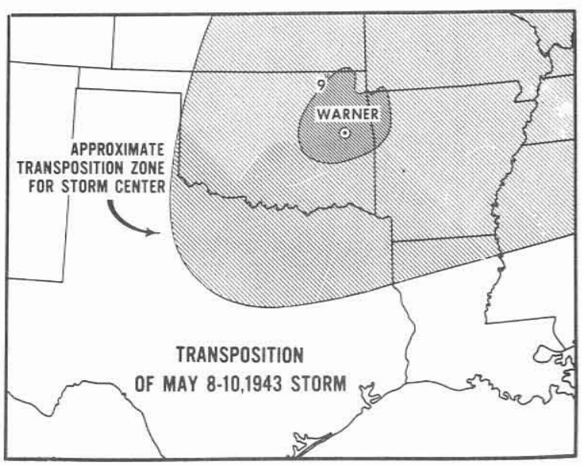

Figure 7. Transposition of May 8-10, 1943 Storm

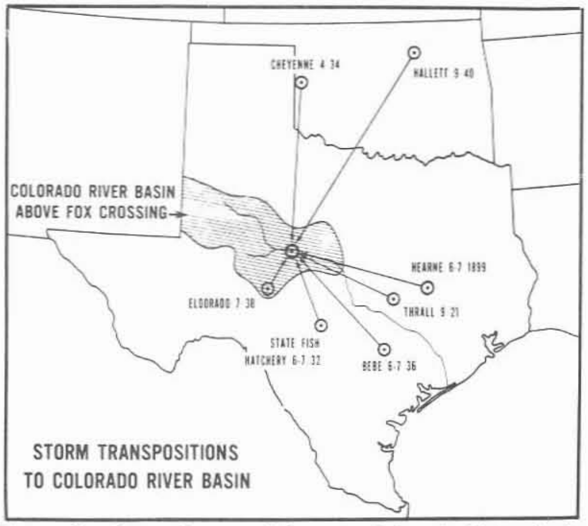

Figure 8. Storm Transpositions to Colorado River Basin

CONTROLLING TRANSPOSED STORMS, COLORADO BASIN

Thsds. Sq. Mi.	Hours 6	12	24	48	72
1	C	T	T	H	H
5	T	T	T	H	H
10	T	T	H	H	H
20	B	B	H	H	H
50	B	B	H	H	H

C - CHEYENNE, OKLA April 3-4, 1934
T - THRALL, TEXAS September 8-10, 1921
B - BEBE, TEXAS June 27-July 4, 1936
H - HEARNE, TEXAS June 2-July 6, 1899

Figure 9. Controlling Transposed Storms, Colorado Basin

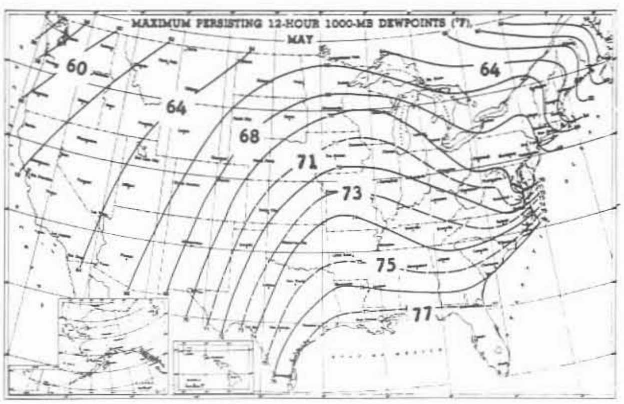

Figure 10. Maximum Persisting 12-Hour 1000-MB Dewpoints (°F)

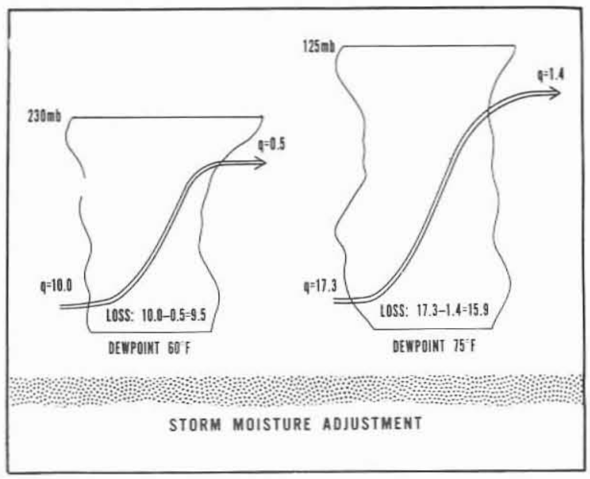

Figure 11. Storm Moisture Adjustment

GENERALIZED CHART OF 24-HR 10,000 MI^2 HURRICANE RAIN

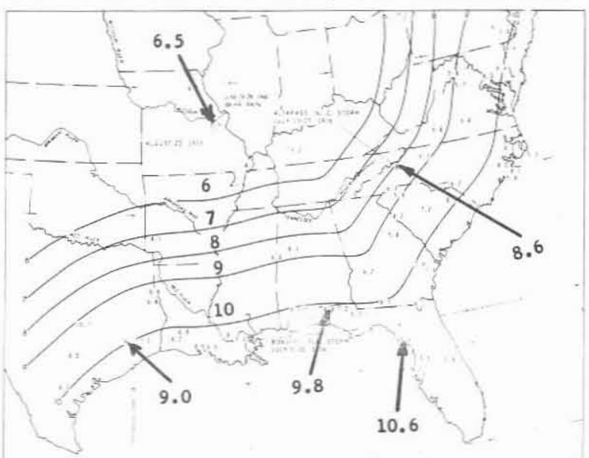

Figure 12. Generalized Chart of 24-Hour 10,000 MI^2 Hurricane Rain

PROBABLE MAXIMUM PRECIPITATION (INCHES)

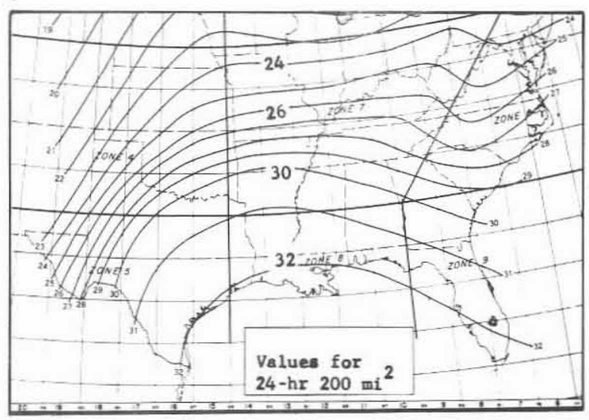

Figure 13. Probable Maximum Precipitation (Inches)

SPILLWAY DESIGN FLOOD CRITERIA FOR SMALL DAMS[a]

By H. N. McGill[b]

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[a] As approved for presentation and publication by the Soil Conservation Service, U.S. Department of Agriculture.

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Introduction

Before the subject is discussed, small dams should be defined. One dictionary definition of a dam is a female parent, used especially of quadrupeds. Small is defined as petite. Thus, one definition of a small dam could be a petite female bovine. This definition does not fit our subject today! But even when we define dams as barriers to prevent the flow of water, the exact meaning of a small dam and the spillway design flood criteria to use become almost as widely variant as the first definition.

Small dams can be divided into two categories. One group would include farm ponds and on-farm erosion control and grade stabilization structures. The product of the capacity in acre-feet and the dam height in feet is less than 3,000 for these structures. Spillway design flood criteria for these structures are simpler than for larger more expensive structures.

Most spillway design flood criteria require the development of inflow hydrographs that express the rate of discharge as a function of time. In these procedures, the determination of critical storm duration becomes difficult. A change in detention storage or spillway capacity changes the critical storm duration. But when units are expressed in terms of a mass diagram, routings can continue until maximum storage in the structure is reached.

A "mass flood routing" procedure that is used in design of these smaller on-farm structures is presented in the Soil Conservation Service, Texas Engineering Handbook, Section 17, Erosion Control Structures. It uses the mass diagram to accumulate inflow and involves two flood routings. One develops a mass diagram of inflow to the site by routing runoff from the point of origin to the site. The other procedure routes the accumulated inflow through the site.

A second categorization of small dams is those structures for which the product of storage and height of dam is over 3,000. The present policy of the Soil Conservation Service limits detention storage capacity to 5,000 acre-feet and total structure capacity to 25,000 acre-feet. Generally, floodwater retarding structures built by the Soil Conservation Service in Texas fall within this range. This paper will be devoted to a discussion of these structures.

Construction of floodwater retarding structures began in Texas in 1949. On June 30, 1965, there were 1,039 either constructed or under contract in the State. Up to this time the Soil Conservation Service has observed about 5,000 structure-years of operation in Texas.

During the 16 years of flood prevention operations in the State, changes in design procedures have resulted from experience and research findings. These have not significantly changed the basic criteria nor the results obtained. In this paper, current procedures used by the Soil Conservation Service will be presented.

Structure Classification

Spillway design flood criteria vary according to structure classification. A number of factors are considered in determining classification. Consideration is given to the damage that might occur to existing and future developments downstream resulting from a sudden breach of the earth embankment and to the structures themselves. The effect of a failure on public confidence is an important factor. State and local regulations and the responsibility of the involved public agencies are recognized. The stability of the spillway materials, the physical characteristics of the site and the valley downstream, and the relationship of the site to industrial and residential areas all have a bearing on the amount of potential damage in the event of failure.

The following structure classes are used to associate criteria with the damage that might result from a sudden major breach of the earth dam embankment.

Class (a) - Structures located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or county roads.

Class (b) - Structures located in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service of relatively important public utilities.

Class (c) - Structures located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways or railroads.

Emergency Spillway and Freeboard Hydrograph Development

Two hydrographs are developed for use in emergency spillway design. One of these, the emergency spillway hydrograph, is used to establish the minimum design dimensions of the emergency spillway. The emergency spillway is designed to pass this hydrograph when routed through the structure at a non-erosive velocity in the emergency spillway. The second, the freeboard hydrograph, is used to establish the minimum elevation of the top of the dam. The procedure for developing these design hydrographs is the same.

Design Storm

Figures 1 to 6 show the 6-hour point rainfall amounts that are used in hydrograph development in Texas. These include both emergency spillway design and freeboard hydrograph rainfall for classes (a), (b), and (c) structures.

Engineering Memorandum SCS-27 (Rev.), Earth Dams, establishes minimum Service requirements. The freeboard hydrographs for class (a) and (b) structures are somewhat less than requirements generally used in this State.

Areal rainfall is obtained by adjusting point rainfall with the use of Figure 8. Where average annual rainfall is 25 inches or more, the humid and subhumid climate curve is used. Where average annual rainfall is 15 inches or less, the arid and semi-arid climate curve adjustment is applied. For areas of the State where annual rainfall is from 15 to 25 inches, the adjustment is interpolated between the two climatic curves.

A storm duration of 6 hours is used except when the time of concentration of the structure drainage area (T_c) is greater than 6 hours, in which case a storm duration at least equal to T_c is used. The relative increase in rainfall amount for storm durations over 6 hours is shown in Figure 7(c).

It is recognized that the determination of T_c for a watershed is often difficult and varies with the individual making the determination. Most methods involve channel or flood plain lengths and certain watershed elevations. A study of stream gage records in the State indicates that T_c can be related to drainage area, land resource area and shape of the watershed. This relationship is shown in Figure 12.

Distribution of the 6 hour design storm is shown in Figure 7(b)[1]. The same pattern is used for longer duration storms.

Design Storm Runoff

The SCS method of estimating direct runoff from storm rainfall is based on methods developed by SCS hydrologists in the last three decades. The hydrologic principles of the method are not new, but they are put to new uses. Because most SCS work is with ungaged watersheds (not gaged for runoff) the method was made to be usable with rainfall and watershed data that are ordinarily available or easily obtained for such watersheds.

Soil properties influence the process of generation of runoff from rainfall. When runoff from individual storms is the major concern, as in the design of small dams, the properties can be represented by a hydrologic parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influences of both the surface and the horizons of a soil are thereby included. The hydrologic soil groups, as defined by SCS methods are:

A. (Low runoff potential) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with a moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High Runoff Potential) Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with permanent high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

In estimating runoff, the effects of the surface conditions of the watershed are evaluated by means of land use and treatment classes. These are listed in Table 2[1] which also shows the runoff curve numbers for hydrologic soil-cover complexes in which the classes are used.

The weighted curve number is computed for the structure drainage area by the summation of products of curve number and the percentage of the drainage area represented by each curve number. Table 1 illustrates this computation. The volume of storm runoff is obtained by use of Figure 9[1] (Sheets 1 and 2) and the areal amount of rainfall.

Hydrograph Computation

Design hydrographs are made using the time, discharge and accumulated runoff ratios for dimensionless hydrographs given in Table 4.[1] The hydrograph family number is determined from Figure 11.[1]

An example of the development of the freeboard hydrograph for a floodwater retarding structure in Dickens County, Texas, is shown in the following step procedure. The structure drainage area is 21.85 square miles, runoff curve number 75 and class (a) structure. Tabulated data are shown in Figure 10. The following steps are used to complete the hydrograph:

1. Determine the time of concentration T_c.

   Length of watershed: 10 miles
   Average width of watershed: 21.85/10 or 2.2 miles
   Length/width ratio: 10.0/2.2 or 4.5
   Length/width ratio factor (Figure 12): 2.2
   Land Resource Area (Figure 13): RR
   T_c (Where L/W = 1.0) (Figure 12) is 1.6 hours
   T_c = (2.2) (1.6) = 3.5 hours

2. Determine the 6-hour freeboard storm rainfall amount (P) in inches. For this location and structure class, use Figure 2 and find point rainfall is 10.9 inches.

The drainage area of the watershed exceeds 10 square miles; thus, an area adjustment should be applied to the point rainfall. Figure 14 shows that the average annual rainfall for the watershed is 21 inches. The adjustment factor will need to be interpolated between the arid and semi-arid climate and the humid and subhumid climate curves in Figure 7, Standard Drawing No. ES 1003a, or in Figure 8.

The humid and subhumid climate factor for 21.85 square miles is .94. The arid and semi-arid climate factor for 21.85 square miles is .87. The applicable adjustment factor is 0.6 (.94-.87) + .87 or .91. Area rainfall is (10.9) (.91) or 10.0.

3. Make the duration adjustment of rainfall amount.

   (Reference Figure 7, Standard Drawing No. ES-1003c)

   Because the time of concentration is not over 6 hours, no adjustment is made. Hence, use storm duration of 6 hours. An example showing necessary adjustment of duration to use when T_c exceeds 6 hours is presented in Chapter 21, SCS National Engineering Handbook, Section 4, Hydrology.

4. Determine the runoff amount Q.

   Enter Figure 9, Sheet 1 of 2 or Sheet 2 of 2 with P = 10.0 and CN = 75 and find Q = 6.87 in.

5. Determine the hydrograph family.

   Enter Figure 11 (ES-1011) with CN = 75 and P = 10.0 inches, read hydrograph family 2.

6. Compute the initial value of T_p.

   Time to peak, $T_p = 0.7\ T_c = .7 \times 3.5 = 2.45$ hours.

7. Determine the duration of excess rainfall.

   Enter Figure 15 (ES-1012) with P = 10.0 inches on CN = 75, read $T_o = 5.13$.

8. Compute ratio T_o/T_p.

   Ratio $T_o/T_p = 5.13/2.45 = 2.09$

9. Select a revised T_o/T_p ratio from Table 3.

This table shows the hydrograph families and T_o/T_p ratios for which dimensionless hydrographs are listed in Table 4. Enter Table 3 with the computed ratio of Step 8 and select the tabulated ratio nearest it. For this example, the selected ratio is 2.0.

10. Compute revised T_p.

    $$\text{Revised } T_p = \frac{T_o}{\text{Used To/Tp}} = 5.13/2.0 = 2.56 \text{ hours}$$

11. Compute q_p.

    $$q_p = \frac{484\ A}{\text{Rev } T_p} = \frac{(484)(21.85)}{2.56} = 4130 \text{ c.f.s.}$$

12. Compute Qq_p.

    $Qq_p = (6.87)(4130) = 28300$ c.f.s.

13. Compute the times at which the hydrograph rates will be computed. Multiply the revised T_p value of 2.56 computed in Step 10 by the t/T_p values in Table 4, Sheet 4 of 15, (Hydrograph Family 2 under heading $T_o/T_p = 2$) to obtain time t in hours for column 2 of Figure 15. Time t for line 2, Figure 10 = t/T_p (Table 4) x Revised T_p = (.28)(2.56) 0.72 hours.

14. Compute the hydrograph rate.

    Multiply Qq_p of 28,300 by the values of q_c/q_p shown in Table 4, Sheet 4 of 15 (Hydrograph Family 2 under heading $T_o/T_p = 2$). The computed rates are shown in column 3 of Figure 10. The q in c.f.s. for line 2, Figure 10 = 28,300 x .004 = 113 c.f.s.

15. Check the total runoff of the computed hydrograph.

    Use equation $Q = \frac{(\Delta t)(\Sigma q)}{645\ A}$. To obtain t, divide the total time of 17.22 hours by the number of lines excluding the first line in column 2, Figure 10. Compute 17.22/24 = 0.72 hour. Σq is the sum of all qs in column 3 of Figure 10 and equals 136,986 c.f.s. by the equation $Q = \frac{(0.72)(136,986)}{(645)(21.85)} = 6.99$ inches. This approximates the actual Q of 6.87 inches and indicates that no gross errors occurred in the hydrograph calculations.

16. Plot or tabulate the hydrograph for flood routing.

[1] Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology.

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1-59

Table 1
UNITED STATES DEPARTMENT OF AGRICULTURE
Soil Conservation Service

Watershed Duck Creek

SOIL COVER COMPLEX

Site or Sample No. 5
Area, Acres 13984

Cover	Practice	Condition or Rotation	II - Curve Number				Present Area				Future Area			
			A	B	C	D	A	B	C	D	A	B	C	D
Rowcrops	St. Row	poor	72	81	88	91								
	St. Row	good	67	78	85	89								
	Contoured	poor	70	79	84	88								
	Contoured	good	65	75	82	86						1720		
*	C&T	poor	66	74	80	82								
	C&T	good	62	71	78	81								
Small grains *	St. Row	poor	65	76	84	88								
	St. Row	good	63	75	83	87						1802		
	C&T	poor	61	72	79	82								
	C&T	good	59	70	78	81								
Legumes or rotation * meadow	St. Row	poor	66	77	85	89								
	St. Row	good	58	72	81	85								
	C&T	poor	63	73	80	83								
	C&T	good	51	67	76	80								
Native range or pasture		poor	68	79	86	89								
		fair	49	69	79	84						4212	6250	
		good	39	61	74	80								
Woods		poor	45	66	77	83								
		fair	36	60	73	79								
		good	25	55	70	77								
Meadow (Perm.)		good	30	58	71	78								
Farmsteads			59	74	82	86								
Roads**	Dirt		72	82	87	89								
	Hard Surface		74	84	90	92								

* Contoured and Terraced (Includes less than 1% slope)
** Includes Rights-of-way

Present ________ Future 75
Date 4-20-65
A.B.C.

Table 2

Runoff curve numbers for hydrologic Soil-Cover Complexes
For Watershed Condition II, and $I_a = 0.2$ (S)

Land Use or Cover	Treatment or Practice	Hydrologic Condition	Hydrologic Soil Group A	B	C	D
Fallow	Straight row	--	77	86	91	94
Row crops	do	Poor	72	81	88	91
	do	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	do	Good	65	75	82	86
	do and terraced	Poor	66	74	80	82
	do and terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	do and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	do	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	do	Good	55	69	78	83
	do and terraced	Poor	63	73	80	83
	do and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	do	Fair	25	59	75	83
	do	Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woods (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		--	59	74	82	86
Roads (dirt) 2/		--	72	82	87	89
(hard surface) 2/		--	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

Table 3--Hydrograph families and T_o/T_p ratios for which dimensionless hydrograph ratios are given in Table 4.

Hydrograph Family	T_o/T_p 1	1.5	2	3	4	6	10	16	25	36	50	75
1	*	*	*	*	*	*	*	*	*	*	*	*
2	*	*	*	*	*	*	*	*	*	*	*	*
3	*	*	*	*	*	*	*	*	*	*	*	*
4	*	*	*	*	*	*	*	*	*	*	*	
5	*	*	*	*	*	*	*	*	*	*	*	

Asterisks signify that dimensionless hydrograph tabulations are given in Table 4.

Table 4--Time and discharge ratios for dimensionless hydrographs
Sheet 1 of 15

Hydrograph family 1

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.029	2	0.32	.012	2	0.29	.007	2	0.35	.005
3	0.56	.150	3	0.64	.118	3	0.58	.035	3	0.70	.027
4	0.84	.472	4	0.96	.377	4	0.87	.164	4	1.05	.101
5	1.12	.798	5	1.28	.711	5	1.16	.432	5	1.40	.302
6	1.40	.901	6	1.60	.815	6	1.45	.669	6	1.75	.563
7	1.68	.776	7	1.92	.719	7	1.74	.740	7	2.10	.650
8	1.96	.568	8	2.24	.526	8	2.03	.680	8	2.45	.576
9	2.24	.389	9	2.56	.352	9	2.32	.561	9	2.80	.460
10	2.52	.258	10	2.88	.225	10	2.61	.441	10	3.15	.374
11	2.80	.173	11	3.20	.143	11	2.90	.319	11	3.50	.290
12	3.08	.115	12	3.52	.090	12	3.19	.212	12	3.85	.201
13	3.36	.078	13	3.84	.057	13	3.48	.140	13	4.20	.127
14	3.64	.052	14	4.16	.037	14	3.77	.094	14	4.55	.078
15	3.92	.036	15	4.48	.024	15	4.06	.063	15	4.90	.047
16	4.20	.024	16	4.80	.015	16	4.35	.042	16	5.25	.028
17	4.48	.016	17	5.12	.008	17	4.64	.028	17	5.60	.016
18	4.76	.009	18	5.44	.004	18	4.93	.017	18	5.95	.009
19	5.04	.005	19	5.76	.002	19	5.22	.011	19	6.30	.005
20	5.32	.002	20	6.08	.001	20	5.51	.007	20	6.65	.003
21	5.60	.001	21	6.40	.000	21	5.80	.004	21	7.00	.002
22	5.88	.000				22	6.09	.002	22	7.35	.001
						23	6.38	.001	23	7.70	.000
						24	6.67	.000			

Table 4--Continued
Sheet 2 of 15

Hydrograph family 1

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.35	.003	2	0.44	.003	2	0.56	.002	2	0.66	.001
3	0.70	.015	3	0.88	.013	3	1.12	.013	3	1.32	.006
4	1.05	.049	4	1.32	.041	4	1.68	.027	4	1.98	.015
5	1.40	.122	5	1.76	.084	5	2.24	.047	5	2.64	.027
6	1.75	.298	6	2.20	.176	6	2.80	.071	6	3.30	.037
7	2.10	.528	7	2.64	.386	7	3.36	.115	7	3.96	.047
8	2.45	.585	8	3.08	.497	8	3.92	.278	8	4.62	.062
9	2.80	.518	9	3.52	.430	9	4.48	.394	9	5.28	.092
10	3.15	.413	10	3.96	.335	10	5.04	.322	10	5.94	.223
11	3.50	.334	11	4.40	.258	11	5.60	.235	11	6.60	.309
12	3.85	.273	12	4.84	.202	12	6.16	.174	12	7.26	.243
13	4.20	.231	13	5.28	.164	13	6.72	.136	13	7.92	.171
14	4.55	.185	14	5.72	.139	14	7.28	.110	14	8.58	.124
15	4.90	.128	15	6.16	.124	15	7.84	.092	15	9.24	.097
16	5.25	.080	16	6.60	.100	16	8.40	.079	16	9.90	.081
17	5.60	.047	17	7.04	.060	17	8.96	.073	17	10.56	.070
18	5.95	.028	18	7.48	.033	18	9.52	.068	18	11.22	.061
19	6.30	.017	19	7.92	.018	19	10.08	.065	19	11.88	.055
20	6.65	.010	20	8.36	.009	20	10.64	.053	20	12.54	.050
21	7.00	.006	21	8.80	.005	21	11.20	.027	21	13.20	.047
22	7.35	.004	22	9.24	.003	22	11.76	.012	22	13.86	.045
23	7.70	.003	23	9.68	.002	23	12.32	.006	23	14.52	.044
24	8.05	.002	24	10.12	.001	24	12.88	.003	24	15.18	.043
25	8.40	.001	25	10.56	.000	25	13.44	.002	25	15.84	.040
26	8.75	.000				26	14.00	.001	26	16.50	.034
						27	14.56	.000	27	17.16	.020
									28	17.82	.008
									29	18.48	.004
									30	19.14	.002
									31	19.80	.001
									32	20.46	.000

Table 4--Continued
Sheet 3 of 15

Hydrograph family 1

$T_o/T_p = 25$			$T_o/T_p = 36$			$T_o/T_p = 50$			$T_o/T_p = 75$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.0000	1	0.00	.0000
2	1.22	.002	2	1.70	.002	2	2.00	.0019	2	3.00	.0017
3	2.44	.009	3	3.40	.008	3	4.00	.0052	3	6.00	.0039
4	3.66	.018	4	5.10	.014	4	6.00	.0083	4	9.00	.0054
5	4.88	.027	5	6.80	.020	5	8.00	.0118	5	12.00	.0084
6	6.10	.036	6	8.50	.026	6	10.00	.0151	6	15.00	.0106
7	7.32	.046	7	10.20	.033	7	12.00	.0192	7	18.00	.0137
8	8.54	.116	8	11.90	.077	8	14.00	.0259	8	21.00	.0197
9	9.76	.232	9	13.60	.177	9	16.00	.0578	9	24.00	.0516
10	10.98	.146	10	15.30	.101	10	18.00	.1330	10	27.00	.0900
11	12.20	.088	11	17.00	.058	11	20.00	.0941	11	30.00	.0593
12	13.42	.062	12	18.70	.044	12	22.00	.0506	12	33.00	.0321
13	14.64	.051	13	20.40	.036	13	24.00	.0357	13	36.00	.0226
14	15.80	.045	14	22.10	.030	14	26.00	.0297	14	39.00	.0188
15	17.09	.039	15	23.80	.027	15	28.00	.0254	15	42.00	.0161
16	18.30	.035	16	25.50	.024	16	30.00	.0219	16	45.00	.0142
17	19.52	.031	17	27.20	.022	17	32.00	.0192	17	48.00	.0125
18	20.74	.027	18	28.90	.020	18	34.00	.0172	18	51.00	.0112
19	21.96	.025	19	30.60	.018	19	36.00	.0159	19	54.00	.0105
20	23.18	.025	20	32.30	.017	20	38.00	.0150	20	57.00	.0100
21	24.40	.025	21	34.00	.017	21	40.00	.0145	21	60.00	.0097
22	25.62	.020	22	35.70	.017	22	42.00	.0140	22	63.00	.0094
23	26.84	.005	23	37.40	.004	23	44.00	.0136	23	66.00	.0090
24	28.06	.002	24	39.10	.002	24	46.00	.0131	24	69.00	.0087
25	29.28	.000	25	40.80	.000	25	48.00	.0125	25	72.00	.0084
						26	50.00	.0123	26	75.00	.0081
						27	52.00	.0016	27	78.00	.0002
						28	54.00	.0000	28	81.00	.0000

Table 4--Continued
Sheet 4 of 15

Hydrograph family 2

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.026	2	0.22	.003	2	0.28	.004	2	0.32	.003
3	0.56	.170	3	0.44	.041	3	0.56	.040	3	0.64	.017
4	0.84	.480	4	0.66	.161	4	0.84	.170	4	0.96	.093
5	1.12	.802	5	0.88	.362	5	1.12	.428	5	1.28	.311
6	1.40	.885	6	1.10	.604	6	1.40	.645	6	1.60	.530
7	1.68	.770	7	1.32	.740	7	1.68	.715	7	1.92	.615
8	1.96	.550	8	1.54	.790	8	1.96	.677	8	2.24	.575
9	2.24	.380	9	1.76	.746	9	2.24	.574	9	2.56	.487
10	2.52	.257	10	1.98	.640	10	2.52	.472	10	2.88	.409
11	2.80	.166	11	2.20	.536	11	2.80	.369	11	3.20	.344
12	3.08	.113	12	2.42	.414	12	3.08	.247	12	3.52	.279
13	3.36	.078	13	2.64	.303	13	3.36	.168	13	3.84	.206
14	3.64	.052	14	2.86	.219	14	3.64	.113	14	4.16	.135
15	3.92	.034	15	3.08	.160	15	3.92	.075	15	4.48	.087
16	4.20	.023	16	3.30	.117	16	4.20	.050	16	4.80	.054
17	4.48	.015	17	3.52	.088	17	4.48	.034	17	5.12	.032
18	4.76	.009	18	3.74	.064	18	4.76	.021	18	5.44	.019
19	5.04	.004	19	3.96	.047	19	5.04	.014	19	5.76	.012
20	5.32	.002	20	4.18	.035	20	5.32	.008	20	6.08	.008
21	5.60	.001	21	4.40	.025	21	5.60	.004	21	6.40	.005
22	5.88	.000	22	4.62	.018	22	5.88	.003	22	6.72	.003
			23	4.84	.012	23	6.16	.002	23	7.04	.002
			24	5.06	.007	24	6.44	.001	24	7.36	.001
			25	5.28	.004	25	6.72	.000	25	7.68	.000
			26	5.50	.003						
			27	5.72	.002						
			28	5.94	.001						
			29	6.16	.000						

Table 4--Continued
Sheet 5 of 15

Hydrograph family 2

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.32	.002	2	0.34	.001	2	0.63	.002	2	0.90	.002
3	0.64	.009	3	0.68	.005	3	1.26	.009	3	1.80	.007
4	0.96	.036	4	1.02	.015	4	1.89	.027	4	2.70	.020
5	1.28	.129	5	1.36	.037	5	2.52	.063	5	3.60	.037
6	1.60	.332	6	1.70	.098	6	3.15	.236	6	4.50	.148
7	1.92	.501	7	2.04	.244	7	3.78	.364	7	5.40	.277
8	2.24	.550	8	2.38	.407	8	4.41	.307	8	6.30	.214
9	2.56	.500	9	2.72	.464	9	5.04	.226	9	7.20	.149
10	2.88	.422	10	3.06	.429	10	5.67	.172	10	8.10	.112
11	3.20	.358	11	3.40	.367	11	6.30	.136	11	9.00	.088
12	3.52	.302	12	3.74	.309	12	6.93	.113	12	9.90	.073
13	3.84	.274	13	4.08	.261	13	7.56	.097	13	10.80	.063
14	4.16	.230	14	4.42	.224	14	8.19	.085	14	11.70	.056
15	4.48	.195	15	4.76	.193	15	8.82	.078	15	12.60	.052
16	4.80	.147	16	5.10	.169	16	9.45	.074	16	13.50	.048
17	5.12	.099	17	5.44	.152	17	10.08	.069	17	14.40	.045
18	5.44	.061	18	5.78	.139	18	10.71	.053	18	15.30	.044
19	5.76	.037	19	6.12	.129	19	11.34	.025	19	16.20	.042
20	6.08	.023	20	6.46	.113	20	11.97	.009	20	17.10	.023
21	6.40	.013	21	6.80	.085	21	12.60	.004	21	18.00	.006
22	6.72	.008	22	7.14	.055	22	13.23	.002	22	18.90	.003
23	7.04	.005	23	7.48	.035	23	13.86	.001	23	19.80	.001
24	7.36	.004	24	7.82	.020	24	14.49	.000	24	20.70	.000
25	7.68	.003	25	8.16	.012						
26	8.00	.002	26	8.50	.008						
27	8.32	.001	27	8.84	.005						
28	8.64	.000	28	9.18	.004						
			29	9.52	.003						
			30	9.86	.002						
			31	10.20	.001						
			32	10.54	.000						

Table 4--Continued
Sheet 6 of 15

Hydrograph family 2

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.0000	1	0.00	.0000
2	1.30	.002	2	1.79	.002	2	2.50	.0018	2	3.00	.0012
3	2.60	.006	3	3.58	.006	3	5.00	.0047	3	6.00	.0027
4	3.90	.014	4	5.37	.012	4	7.50	.0087	4	9.00	.0044
5	5.20	.024	5	7.16	.019	5	10.00	.0145	5	12.00	.0067
6	6.50	.088	6	8.95	.057	6	12.50	.0615	6	15.00	.0108
7	7.80	.210	7	10.74	.157	7	15.00	.1184	7	18.00	.0309
8	9.10	.146	8	12.53	.104	8	17.50	.0621	8	21.00	.0790
9	10.40	.097	9	14.32	.068	9	20.00	.0433	9	24.00	.0624
10	11.70	.072	10	16.11	.047	10	22.50	.0342	10	27.00	.0357
11	13.00	.057	11	17.90	.040	11	25.00	.0274	11	30.00	.0283
12	14.30	.049	12	19.69	.034	12	27.50	.0234	12	33.00	.0234
13	15.60	.044	13	21.48	.030	13	30.00	.0209	13	36.00	.0196
14	16.90	.039	14	23.27	.026	14	32.50	.0187	14	39.00	.0167
15	18.20	.035	15	25.06	.025	15	35.00	.0167	15	42.00	.0150
16	19.50	.033	16	26.85	.023	16	37.50	.0159	16	45.00	.0137
17	20.80	.031	17	28.64	.021	17	40.00	.0153	17	48.00	.0126
18	22.10	.029	18	30.43	.020	18	42.50	.0147	18	51.00	.0115
19	23.40	.028	19	32.22	.019	19	45.00	.0142	19	54.00	.0108
20	24.70	.027	20	34.01	.018	20	47.50	.0136	20	57.00	.0104
21	26.00	.014	21	35.80	.017	21	50.00	.0131	21	60.00	.0101
22	27.30	.004	22	37.59	.007	22	52.50	.0008	22	63.00	.0098
23	28.60	.001	23	39.38	.001	23	55.00	.0000	23	66.00	.0095
24	29.90	.000	24	41.17	.000				24	69.00	.0092
									25	72.00	.0089
									26	75.00	.0086
									27	78.00	.0003
									28	81.00	.0000

Table 4--Continued
Sheet 7 of 15

Hydrograph family 3

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.26	.048	2	0.29	.028	2	0.30	.012	2	0.34	.004
3	0.52	.219	3	0.58	.190	3	0.60	.123	3	0.68	.088
4	0.78	.521	4	0.87	.450	4	0.90	.343	4	1.02	.289
5	1.04	.762	5	1.16	.656	5	1.20	.570	5	1.36	.489
6	1.30	.844	6	1.45	.734	6	1.50	.657	6	1.70	.543
7	1.56	.778	7	1.74	.685	7	1.80	.630	7	2.04	.507
8	1.82	.621	8	2.03	.585	8	2.10	.562	8	2.38	.445
9	2.08	.441	9	2.32	.445	9	2.40	.484	9	2.72	.385
10	2.34	.305	10	2.61	.350	10	2.70	.379	10	3.06	.340
11	2.60	.214	11	2.90	.199	11	3.00	.267	11	3.40	.294
12	2.86	.149	12	3.19	.132	12	3.30	.177	12	3.74	.223
13	3.12	.103	13	3.48	.089	13	3.60	.116	13	4.08	.149
14	3.38	.070	14	3.77	.057	14	3.90	.076	14	4.42	.096
15	3.64	.048	15	4.06	.038	15	4.20	.050	15	4.76	.056
16	3.90	.034	16	4.35	.025	16	4.50	.033	16	5.10	.033
17	4.16	.024	17	4.64	.015	17	4.80	.020	17	5.44	.019
18	4.42	.016	18	4.93	.008	18	5.10	.011	18	5.78	.013
19	4.68	.010	19	5.22	.005	19	5.40	.006	19	6.12	.008
20	4.94	.006	20	5.51	.003	20	5.70	.004	20	6.46	.004
21	5.20	.003	21	5.80	.002	21	6.00	.002	21	6.80	.003
22	5.46	.001	22	6.09	.001	22	6.30	.001	22	7.14	.002
23	5.72	.000	23	6.38	.000	23	6.60	.000	23	7.48	.001
									24	7.82	.000

Table 4--Continued
Sheet 8 of 15

Hydrograph family 3

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.36	.003	2	0.42	.002	2	0.54	.001	2	0.90	.002
3	0.72	.044	3	0.84	.021	3	1.08	.008	3	1.80	.016
4	1.08	.203	4	1.26	.138	4	1.62	.069	4	2.70	.122
5	1.44	.400	5	1.68	.320	5	2.16	.231	5	3.60	.230
6	1.80	.478	6	2.10	.390	6	2.70	.303	6	4.50	.185
7	2.16	.450	7	2.52	.363	7	3.24	.269	7	5.40	.139
8	2.52	.397	8	2.94	.314	8	3.78	.223	8	6.30	.113
9	2.88	.342	9	3.36	.270	9	4.32	.188	9	7.20	.094
10	3.24	.296	10	3.78	.232	10	4.86	.159	10	8.10	.081
11	3.60	.257	11	4.20	.199	11	5.40	.139	11	9.00	.072
12	3.96	.234	12	4.62	.174	12	5.94	.122	12	9.90	.064
13	4.32	.210	13	5.04	.155	13	6.48	.108	13	10.80	.057
14	4.68	.169	14	5.46	.144	14	7.02	.097	14	11.70	.053
15	5.04	.111	15	5.88	.137	15	7.56	.089	15	12.60	.050
16	5.40	.067	16	6.30	.127	16	8.10	.081	16	13.50	.049
17	5.76	.037	17	6.72	.101	17	8.64	.078	17	14.40	.048
18	6.12	.022	18	7.14	.063	18	9.18	.077	18	15.30	.047
19	6.48	.014	19	7.56	.033	19	9.72	.077	19	16.20	.046
20	6.84	.008	20	7.98	.018	20	10.26	.075	20	17.10	.024
21	7.20	.006	21	8.40	.010	21	10.80	.055	21	18.00	.006
22	7.56	.004	22	8.82	.005	22	11.34	.030	22	18.90	.004
23	7.92	.002	23	9.24	.003	23	11.88	.012	23	19.80	.002
24	8.28	.001	24	9.66	.002	24	12.42	.006	24	20.70	.000
25	8.64	.000	25	10.08	.001	25	12.96	.004			
			26	10.50		26	13.50	.002			
			27	10.92	.000	27	14.04	.001			
						28	14.58	.000			

Table 4--Continued
Sheet 9 of 15

Hydrograph family 3

$T_o/T_p = 25$			$T_o/T_p = 36$			$T_o/T_p = 50$			$T_o/T_p = 75$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.0000	1	0.00	.0000
2	1.23	.002	2	1.62	.002	2	2.25	.0008	2	3.25	.0009
3	2.46	.009	3	3.24	.006	3	4.50	.0070	3	6.50	.0057
4	3.69	.073	4	4.86	.047	4	6.75	.0474	4	9.75	.0289
5	4.92	.173	5	6.48	.130	5	9.00	.0972	5	13.00	.0667
6	6.15	.132	6	8.10	.097	6	11.25	.0642	6	16.25	.0445
7	7.38	.096	7	9.72	.069	7	13.50	.0460	7	19.50	.0317
8	8.61	.076	8	11.34	.052	8	15.75	.0375	8	22.75	.0257
9	9.84	.064	9	12.96	.045	9	18.00	.0322	9	26.00	.0219
10	11.07	.055	10	14.58	.041	10	20.25	.0285	10	29.25	.0195
11	12.30	.050	11	16.20	.037	11	22.50	.0258	11	32.50	.0176
12	13.53	.046	12	17.82	.034	12	24.75	.0239	12	35.75	.0160
13	14.76	.042	13	19.44	.031	13	27.00	.0219	13	39.00	.0147
14	15.99	.038	14	21.06	.028	14	29.25	.0201	14	42.25	.0136
15	17.22	.035	15	22.68	.025	15	31.50	.0185	15	45.50	.0127
16	18.45	.033	16	24.30	.024	16	33.75	.0173	16	48.75	.0118
17	19.68	.032	17	25.92	.024	17	36.00	.0165	17	52.00	.0113
18	20.91	.031	18	27.54	.024	18	38.25	.0162	18	55.25	.0109
19	22.14	.031	19	29.16	.024	19	40.50	.0159	19	58.50	.0107
20	23.37	.031	20	30.78	.023	20	42.75	.0156	20	61.75	.0105
21	24.60	.031	21	32.40	.023	21	45.00	.0153	21	65.00	.0103
22	25.83	.025	22	34.02	.023	22	47.25	.0150	22	68.25	.0101
23	27.06	.004	23	35.64	.023	23	49.50	.0147	23	71.50	.0099
24	28.29	.001	24	37.26	.007	24	51.75	.0028	24	74.75	.0097
25	29.52	.000	25	38.88	.003	25	54.00	.0000	25	78.00	.0003
			26	40.50	.000				26	81.25	.0000

Hydrograph family 4

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.051	2	0.28	.038	2	0.32	.031	2	0.28	.018
3	0.56	.220	3	0.56	.166	3	0.64	.173	3	0.56	.086
4	0.84	.490	4	0.84	.360	4	0.96	.360	4	0.84	.200
5	1.12	.738	5	1.12	.551	5	1.28	.494	5	1.12	.311
6	1.40	.830	6	1.40	.651	6	1.60	.555	6	1.40	.386
7	1.68	.751	7	1.68	.686	7	1.92	.567	7	1.68	.415
8	1.96	.573	8	1.96	.650	8	2.24	.555	8	1.96	.422
9	2.24	.392	9	2.24	.543	9	2.56	.490	9	2.24	.417
10	2.52	.259	10	2.52	.392	10	2.88	.370	10	2.52	.402
11	2.80	.174	11	2.80	.267	11	3.20	.242	11	2.80	.394
12	3.08	.118	12	3.08	.180	12	3.52	.150	12	3.08	.387
13	3.36	.079	13	3.36	.120	13	3.84	.098	13	3.36	.363
14	3.64	.053	14	3.64	.081	14	4.16	.063	14	3.64	.316
15	3.92	.036	15	3.92	.055	15	4.48	.038	15	3.92	.236
16	4.20	.025	16	4.20	.036	16	4.80	.024	16	4.20	.164
17	4.48	.017	17	4.48	.024	17	5.12	.013	17	4.48	.108
18	4.76	.011	18	4.76	.015	18	5.44	.008	18	4.76	.073
19	5.04	.006	19	5.04	.009	19	5.76	.004	19	5.04	.047
20	5.32	.003	20	5.32	.005	20	6.08	.002	20	5.32	.030
21	5.60	.001	21	5.60	.003	21	6.40	.001	21	5.60	.020
22	5.88	.000	22	5.88	.001	22	6.72	.000	22	5.88	.013
			23	6.16	.000				23	6.16	.008
									24	6.44	.005
									25	6.72	.003
									26	7.00	.002
									27	7.28	.001
									28	7.56	
									29	7.84	.000

Table 4--Continued
Sheet 11 of 15

Hydrograph family 4

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.40	.023	2	0.40	.014	2	0.50	.015	2	0.62	.015
3	0.80	.143	3	0.80	.088	3	1.00	.079	3	1.24	.064
4	1.20	.272	4	1.20	.191	4	1.50	.151	4	1.86	.112
5	1.60	.326	5	1.60	.244	5	2.00	.177	5	2.48	.128
6	2.00	.340	6	2.00	.250	6	2.50	.170	6	3.10	.119
7	2.40	.337	7	2.40	.246	7	3.00	.159	7	3.72	.105
8	2.80	.323	8	2.80	.240	8	3.50	.152	8	4.34	.097
9	3.20	.306	9	3.20	.233	9	4.00	.146	9	4.96	.094
10	3.60	.293	10	3.60	.223	10	4.50	.141	10	5.58	.091
11	4.00	.286	11	4.00	.212	11	5.00	.136	11	6.20	.089
12	4.40	.266	12	4.40	.202	12	5.50	.131	12	6.82	.087
13	4.80	.197	13	4.80	.194	13	6.00	.126	13	7.44	.085
14	5.20	.122	14	5.20	.189	14	6.50	.121	14	8.06	.082
15	5.60	.067	15	5.60	.187	15	7.00	.116	15	8.68	.079
16	6.00	.036	16	6.00	.185	16	7.50	.112	16	9.30	.076
17	6.40	.021	17	6.40	.175	17	8.00	.112	17	9.92	.074
18	6.80	.013	18	6.80	.131	18	8.50	.111	18	10.54	.072
19	7.20	.008	19	7.20	.080	19	9.00	.111	19	11.16	.071
20	7.60	.005	20	7.60	.046	20	9.50	.110	20	11.78	.070
21	8.00	.002	21	8.00	.027	21	10.00	.110	21	12.40	.069
22	8.40	.001	22	8.40	.016	22	10.50	.100	22	13.02	.069
23	8.80	.000	23	8.80	.009	23	11.00	.065	23	13.64	.069
			24	9.20	.005	24	11.50	.033	24	14.26	.069
			25	9.60	.003	25	12.00	.025	25	14.88	.069
			26	10.00	.002	26	12.50	.007	26	15.50	.069
			27	10.40	.001	27	13.00	.004	27	16.12	.068
			28	10.80	.000	28	13.50	.002	28	16.74	.053
						29	14.00	.001	29	17.36	.023
						30	14.50	.000	30	17.98	.009
									31	18.60	.004
									32	19.22	.002
									33	19.84	.001
									34	20.46	.000

Hydrograph family 4

$T_o/T_p = 25$			$T_o/T_p = 36$			$T_o/T_p = 50$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.0000	1	0.00	.0000
2	1.02	.025	2	1.50	.0306	2	2.00	.0277
3	2.04	.070	3	3.00	.0575	3	4.00	.0464
4	3.06	.092	4	4.50	.0672	4	6.00	.0435
5	4.08	.082	5	6.00	.0492	5	8.00	.0378
6	5.10	.068	6	7.50	.0433	6	10.00	.0335
7	6.12	.062	7	9.00	.0418	7	12.00	.0307
8	7.14	.059	8	10.50	.0408	8	14.00	.0291
9	8.16	.056	9	12.00	.0400	9	16.00	.0282
10	9.18	.055	10	13.50	.0391	10	18.00	.0274
11	10.20	.054	11	15.00	.0382	11	20.00	.0266
12	11.22	.053	12	16.50	.0371	12	22.00	.0258
13	12.24	.052	13	18.00	.0358	13	24.00	.0250
14	13.26	.050	14	19.50	.0341	14	26.00	.0242
15	14.28	.049	15	21.00	.0319	15	28.00	.0234
16	15.30	.047	16	22.50	.0308	16	30.00	.0230
17	16.32	.046	17	24.00	.0306	17	32.00	.0229
18	17.34	.045	18	25.50	.0306	18	34.00	.0227
19	18.36	.044	19	27.00	.0306	19	36.00	.0226
20	19.38	.044	20	28.50	.0306	20	38.00	.0225
21	20.40	.044	21	30.00	.0306	21	40.00	.0224
22	21.42	.044	22	31.50	.0306	22	42.00	.0222
23	22.44	.044	23	33.00	.0306	23	44.00	.0221
24	23.46	.044	24	34.50	.0306	24	46.00	.0219
25	24.48	.044	25	35.00	.0306	25	48.00	.0219
26	25.50	.039	26	37.50	.0085	26	50.00	.0217
27	26.52	.012	27	39.00	.0009	27	52.00	.0029
28	27.54	.004	28	40.50	.0000	28	54.00	.0000
29	28.56	.001						
30	29.58	.000						

Hydrograph family 5

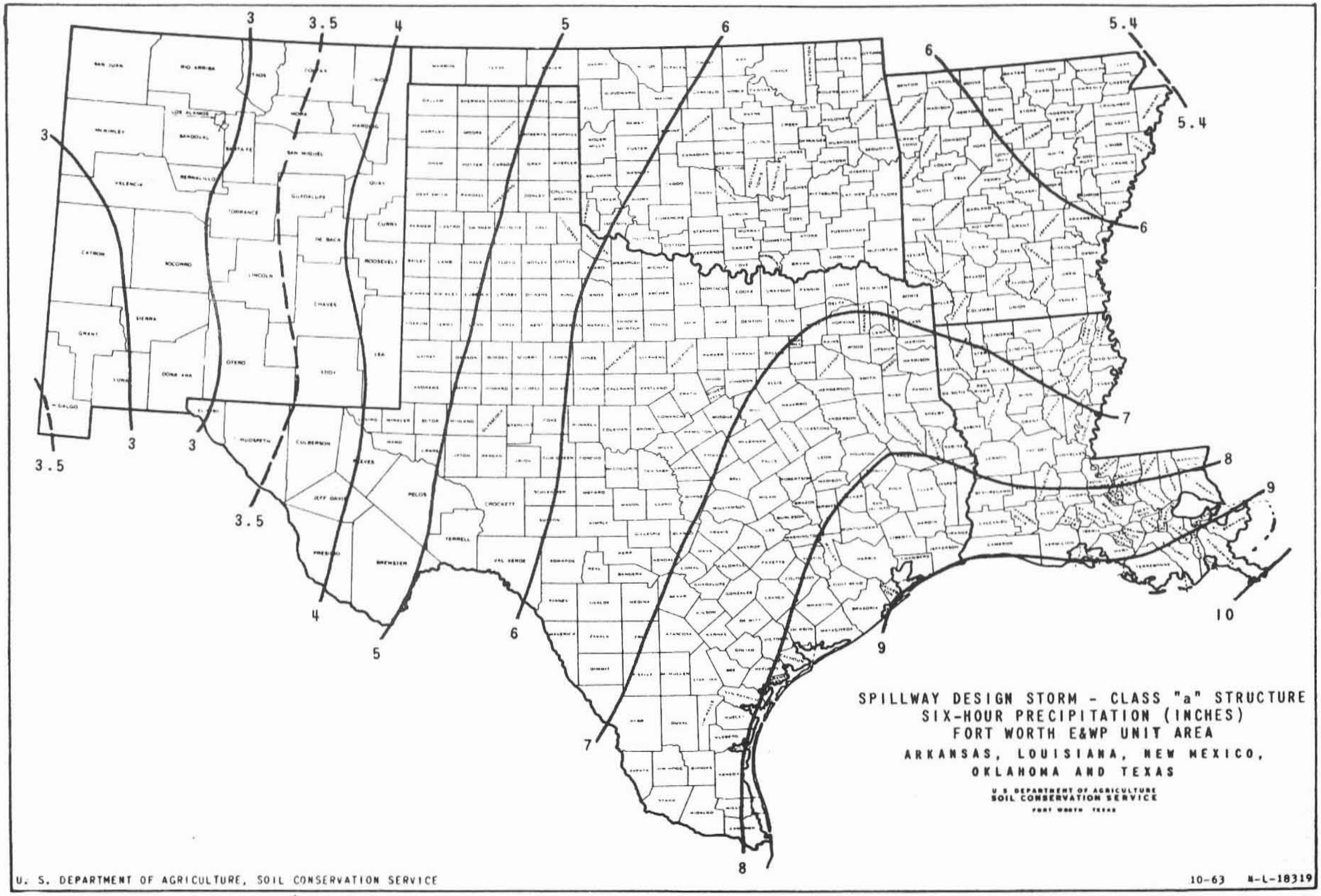
$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	.000	.000
2	0.26	.021	2	0.25	.013	2	0.25	.010	2	.034	.010
3	0.52	.106	3	0.50	.065	3	0.50	.048	3	0.68	.068
4	0.78	.289	4	0.75	.173	4	0.75	.127	4	1.02	.150
5	1.04	.530	5	1.00	.306	5	1.00	.227	5	1.36	.229
6	1.30	.740	6	1.25	.434	6	1.25	.318	6	1.70	.283
7	1.56	.848	7	1.50	.562	7	1.50	.389	7	2.04	.315
8	1.82	.767	8	1.75	.680	8	1.75	.448	8	2.38	.339
9	2.08	.590	9	2.00	.737	9	2.00	.523	9	2.72	.378
10	2.34	.406	10	2.25	.673	10	2.25	.609	10	3.06	.459
11	2.60	.279	11	2.50	.530	11	2.50	.642	11	3.40	.509
12	2.86	.193	12	2.75	.381	12	2.75	.576	12	3.74	.446
13	3.12	.134	13	3.00	.262	13	3.00	.450	13	4.08	.310
14	3.38	.092	14	3.25	.185	14	3.25	.322	14	4.42	.190
15	3.64	.065	15	3.50	.129	15	3.50	.222	15	4.76	.117
16	3.90	.044	16	3.75	.090	16	3.75	.156	16	5.10	.069
17	4.16	.030	17	4.00	.063	17	4.00	.109	17	5.44	.040
18	4.42	.021	18	4.25	.045	18	4.25	.075	18	5.78	.025
19	4.68	.015	19	4.50	.031	19	4.50	.053	19	6.12	.016
20	4.94	.009	20	4.75	.022	20	4.75	.037	20	6.46	.009
21	5.20	.005	21	5.00	.014	21	5.00	.025	21	6.80	.005
22	5.46	.002	22	5.25	.009	22	5.25	.017	22	7.14	.003
23	5.72	.000	23	5.50	.005	23	5.50	.011	23	7.48	.001
			24	5.75	.003	24	5.75	.007	24	7.82	.000
			25	6.00	.001	25	6.00	.004			
			26	6.25	.000	26	6.25	.002			
						27	6.50	.001			
						28	6.75	.000			

Hydrograph family 5

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.36	.010	2	0.52	.015	2	0.67	.013	2	0.80	.008
3	0.72	.053	3	1.04	.070	3	1.34	.061	3	1.60	.046
4	1.08	.124	4	1.56	.130	4	2.01	.091	4	2.40	.060
5	1.44	.181	5	2.08	.159	5	2.68	.102	5	3.20	.065
6	1.80	.220	6	2.60	.172	6	3.35	.107	6	4.00	.067
7	2.16	.243	7	3.12	.178	7	4.02	.110	7	4.80	.067
8	2.52	.256	8	3.64	.182	8	4.69	.111	8	5.60	.068
9	2.88	.263	9	4.16	.183	9	5.36	.111	9	6.40	.068
10	3.24	.273	10	4.68	.184	10	6.03	.112	10	7.20	.068
11	3.60	.308	11	5.20	.218	11	6.70	.112	11	8.00	.068
12	3.96	.380	12	5.72	.285	12	7.37	.112	12	8.80	.068
13	4.32	.427	13	6.24	.324	13	8.04	.116	13	9.60	.068
14	4.68	.377	14	6.76	.267	14	8.71	.160	14	10.40	.068
15	5.04	.260	15	7.28	.133	15	9.38	.198	15	11.20	.068
16	5.40	.155	16	7.80	.064	16	10.05	.212	16	12.00	.068
17	5.76	.094	17	8.32	.029	17	10.72	.168	17	12.80	.086
18	6.12	.055	18	8.84	.016	18	11.39	.074	18	13.60	.121
19	6.48	.032	19	9.36	.007	19	12.06	.027	19	14.40	.133
20	6.84	.019	20	9.88	.003	20	12.73	.010	20	15.20	.136
21	7.20	.012	21	10.40	.001	21	13.40	.005	21	16.00	.137
22	7.56	.007	22	10.92	.000	22	14.07	.002	22	16.80	.098
23	7.92	.004				23	14.74	.000	23	17.60	.033
24	8.28	.002							24	18.40	.012
25	8.64	.000							25	19.20	.004
									26	20.00	.001
									27	20.80	.000

Hydrograph family 5

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p = 50		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.0000	1	0.00	.0000
2	1.25	.015	2	1.50	.0195	2	2.00	.0167
3	2.50	.039	3	3.00	.0275	3	4.00	.0204
4	3.75	.043	4	4.50	.0294	4	6.00	.0214
5	5.00	.044	5	6.00	.0300	5	8.00	.0216
6	6.25	.044	6	7.50	.0301	6	10.00	.0216
7	7.50	.044	7	9.00	.0301	7	12.00	.0216
8	8.75	.044	8	10.50	.0301	8	14.00	.0216
9	10.00	.044	9	12.00	.0301	9	16.00	.0216
10	11.25	.044	10	13.50	.0301	10	18.00	.0216
11	12.50	.044	11	15.00	.0301	11	20.00	.0216
12	13.75	.044	12	16.50	.0301	12	22.00	.0216
13	15.00	.044	13	18.00	.0301	13	24.00	.0216
14	16.25	.044	14	19.50	.0301	14	26.00	.0216
15	17.50	.044	15	21.00	.0301	15	28.00	.0216
16	18.75	.045	16	22.50	.0301	16	30.00	.0216
17	20.00	.067	17	24.00	.0311	17	32.00	.0217
18	21.25	.083	18	25.50	.0364	18	34.00	.0243
19	22.50	.087	19	27.00	.0425	19	36.00	.0287
20	23.75	.087	20	28.50	.0480	20	38.00	.0329
21	25.00	.088	21	30.00	.0525	21	40.00	.0363
22	26.25	.035	22	31.50	.0561	22	42.00	.0391
23	27.50	.006	23	33.00	.0584	23	44.00	.0411
24	28.75	.002	24	34.50	.0598	24	46.00	.0423
25	30.00	.000	25	36.00	.0603	25	48.00	.0430
			26	37.50	.0167	26	50.00	.0433
			27	39.00	.0018	27	52.00	.0058
			28	40.50	.0000	28	54.00	.0002
						29	56.00	.0000


SPILLWAY DESIGN STORM - CLASS "a" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO,
OKLAHOMA AND TEXAS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH TEXAS
3
3.5
4
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5.4
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U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
10-63 4-L-18319

Figure 1

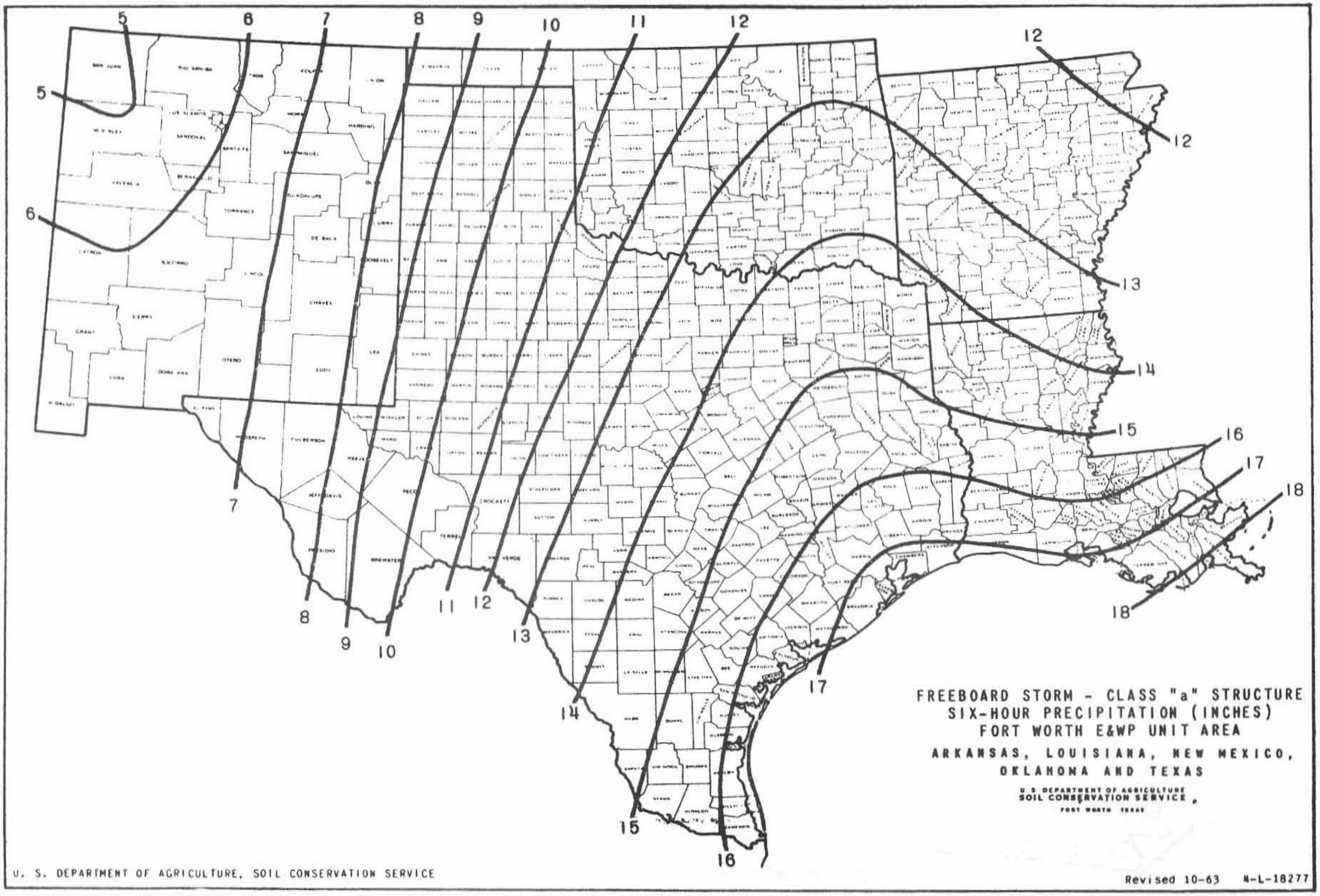
FREEBOARD STORM - CLASS "a" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO,
OKLAHOMA AND TEXAS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH TEXAS
U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
Revised 10-63 M-L-18277
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Figure 2

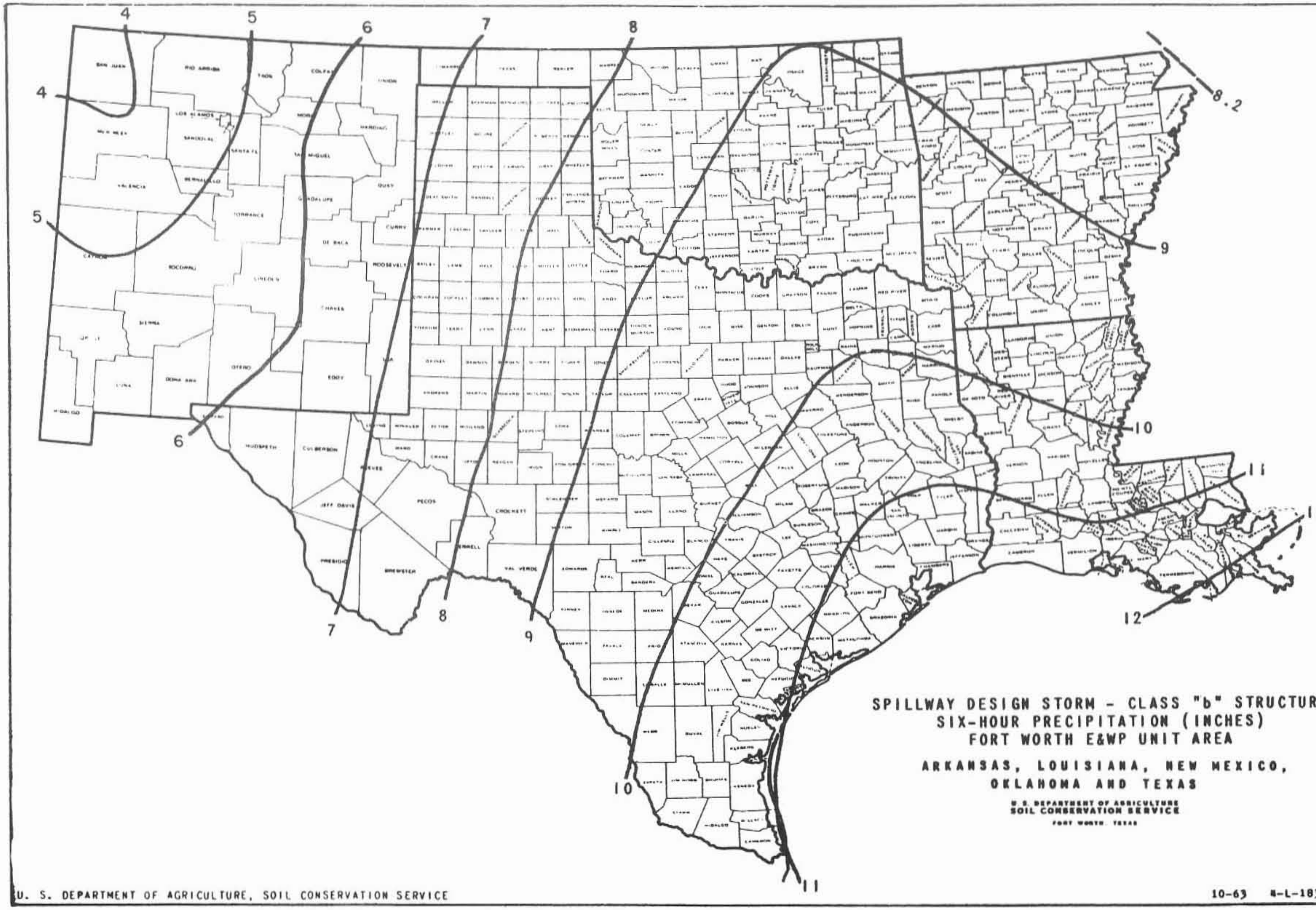

SPILLWAY DESIGN STORM - CLASS "b" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO,
OKLAHOMA AND TEXAS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH, TEXAS
U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
10-63 4-L-18318

Figure 3

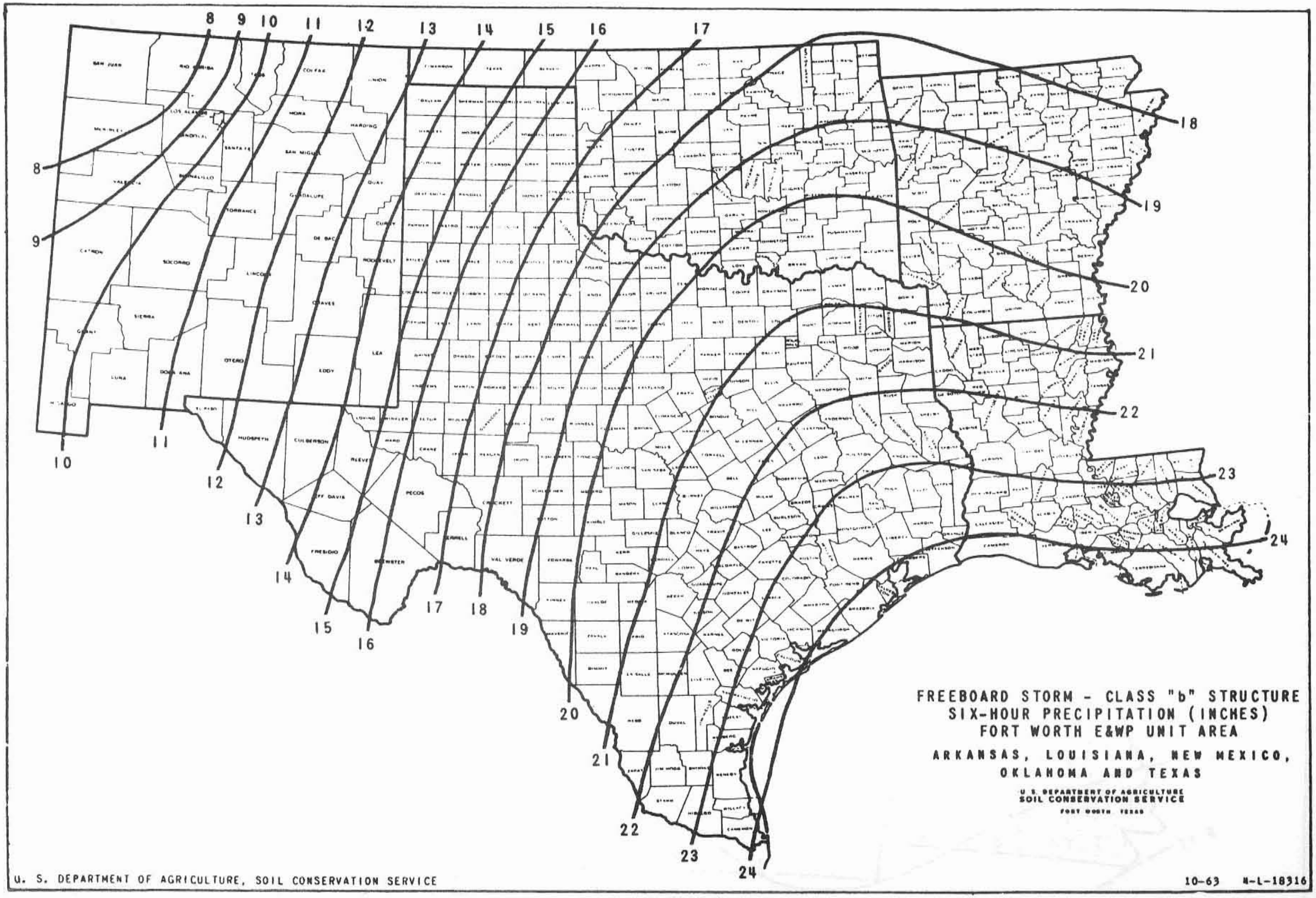
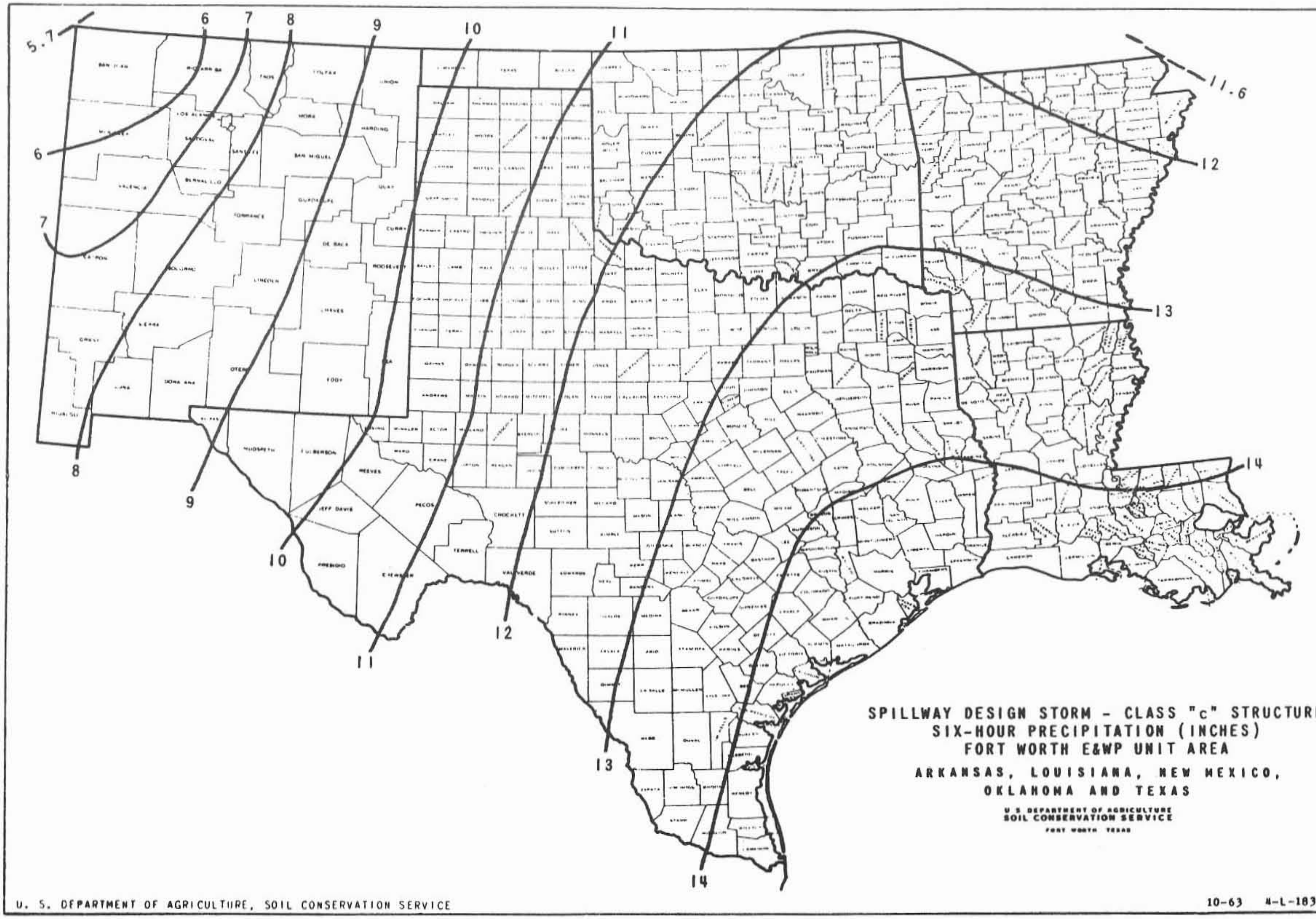

FREEBOARD STORM - CLASS "b" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO,
OKLAHOMA AND TEXAS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH TEXAS
U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
10-63 4-L-18316

Figure 4


SPILLWAY DESIGN STORM - CLASS "c" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO, OKLAHOMA AND TEXAS
U S DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
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U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
10-63 4-L-18315

Figure 5

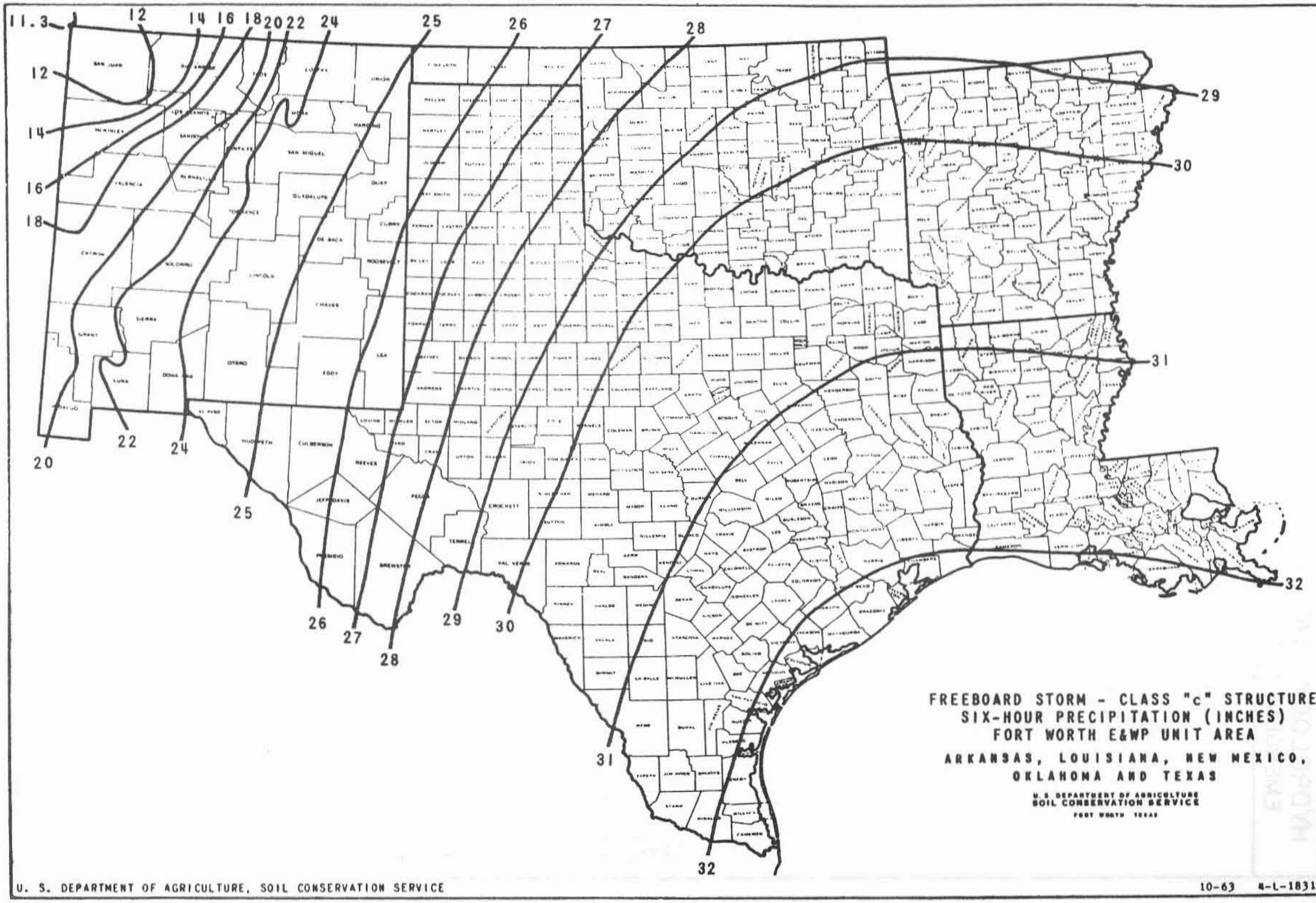
FREEBOARD STORM - CLASS "c" STRUCTURE
SIX-HOUR PRECIPITATION (INCHES)
FORT WORTH E&WP UNIT AREA
ARKANSAS, LOUISIANA, NEW MEXICO,
OKLAHOMA AND TEXAS
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH TEXAS
U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
10-63 4-L-18317
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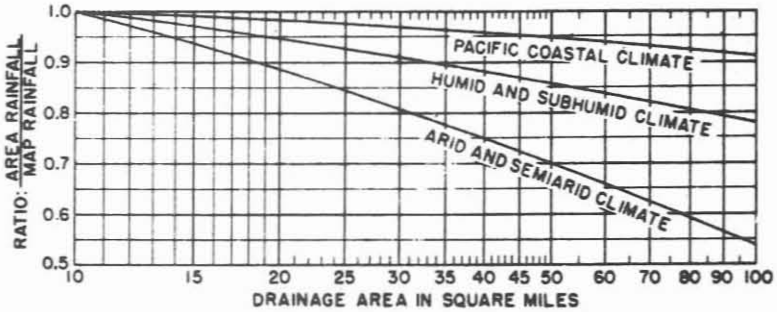
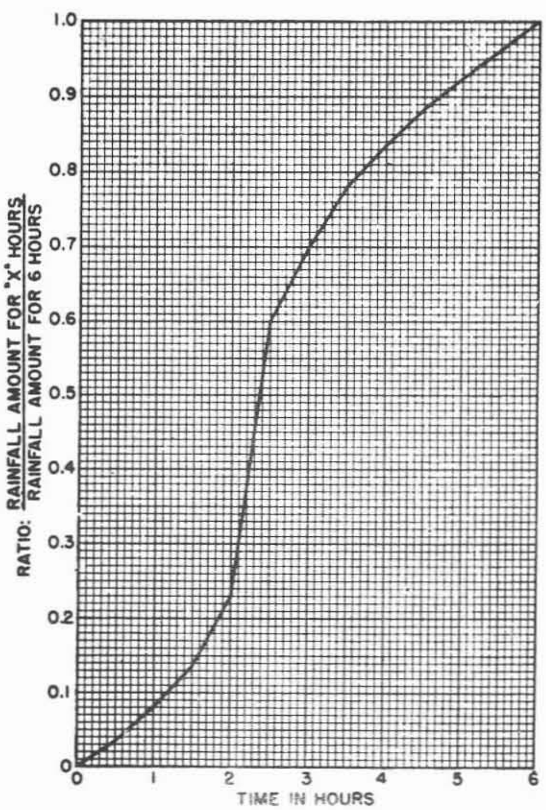
Figure 6

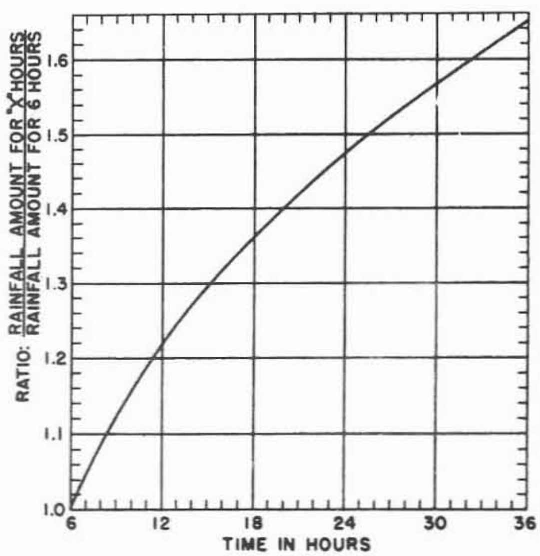
HYDROLOGY: CRITERIA FOR DESIGN STORMS USED IN DEVELOPING EMERGENCY SPILLWAY DESIGN AND FREEBOARD HYDROGRAPHS



(a) RAINFALL RATIOS FOR DRAINAGE AREAS OF 10 TO 100 SQUARE MILES



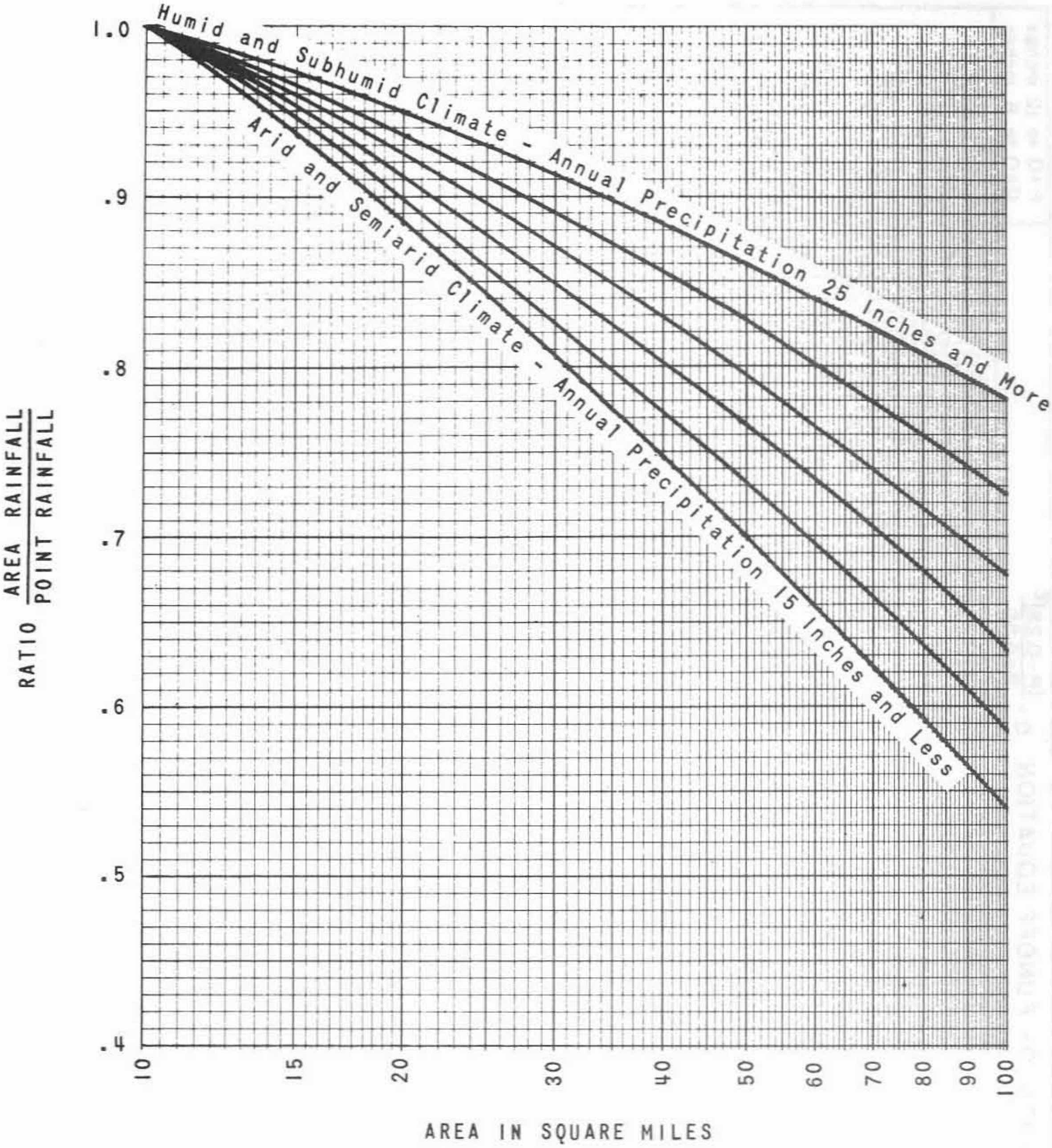
(b) SIX HOUR DESIGN STORM DISTRIBUTION



(c) RELATIVE INCREASE IN RAINFALL AMOUNT FOR STORM DURATIONS OVER SIX HOURS

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES 1003 SHEET 1 OF 1 DATE 7-2-56

Figure 7



Note: When the drainage area is more than 100 square miles, the adjustment will be determined by the Engineering Division.

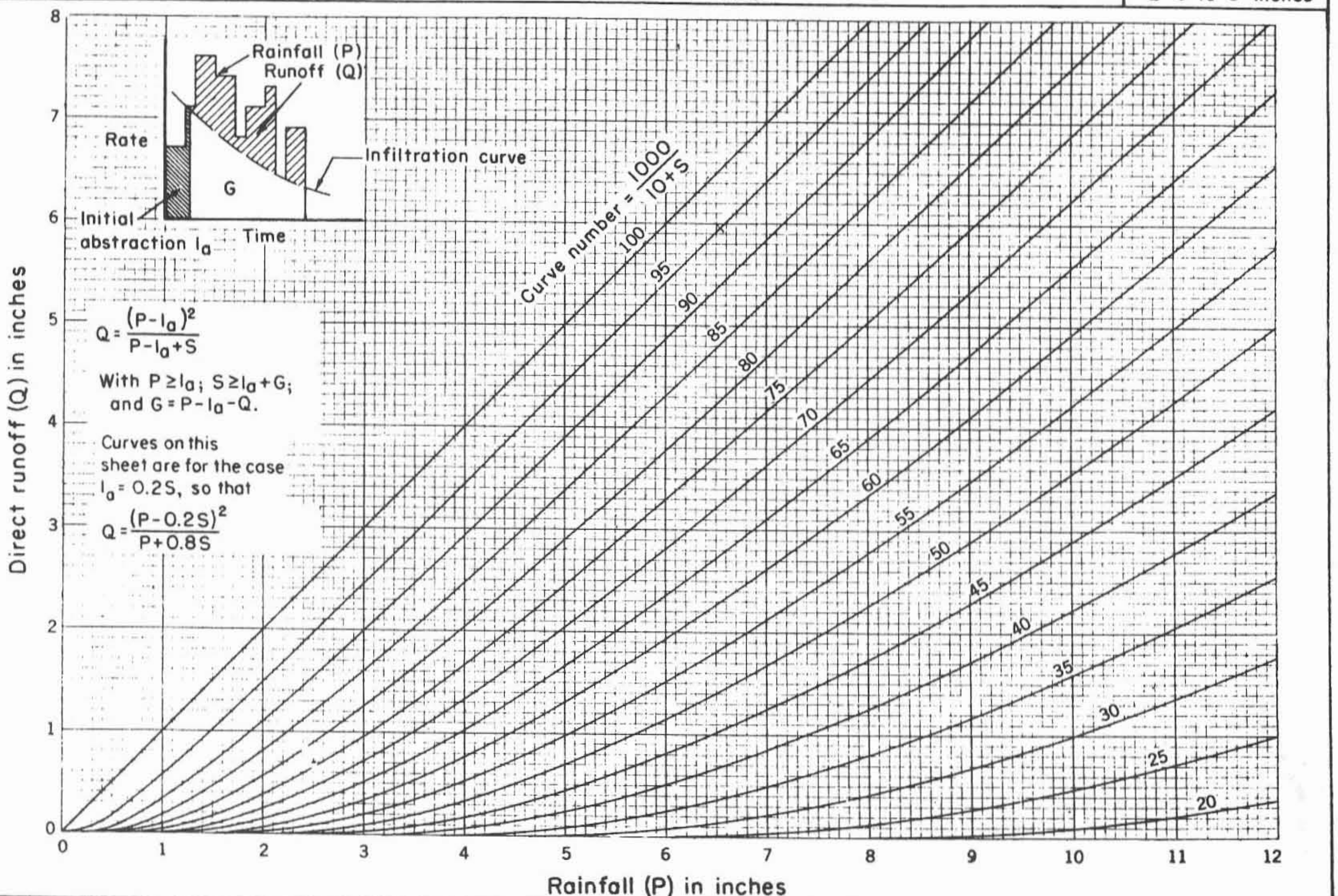
Figure 8

RAINFALL AREA REDUCTION

U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE 8-65 4-L-20499

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$

P = 0 to 12 inches
Q = 0 to 8 inches



REFERENCE

Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955.

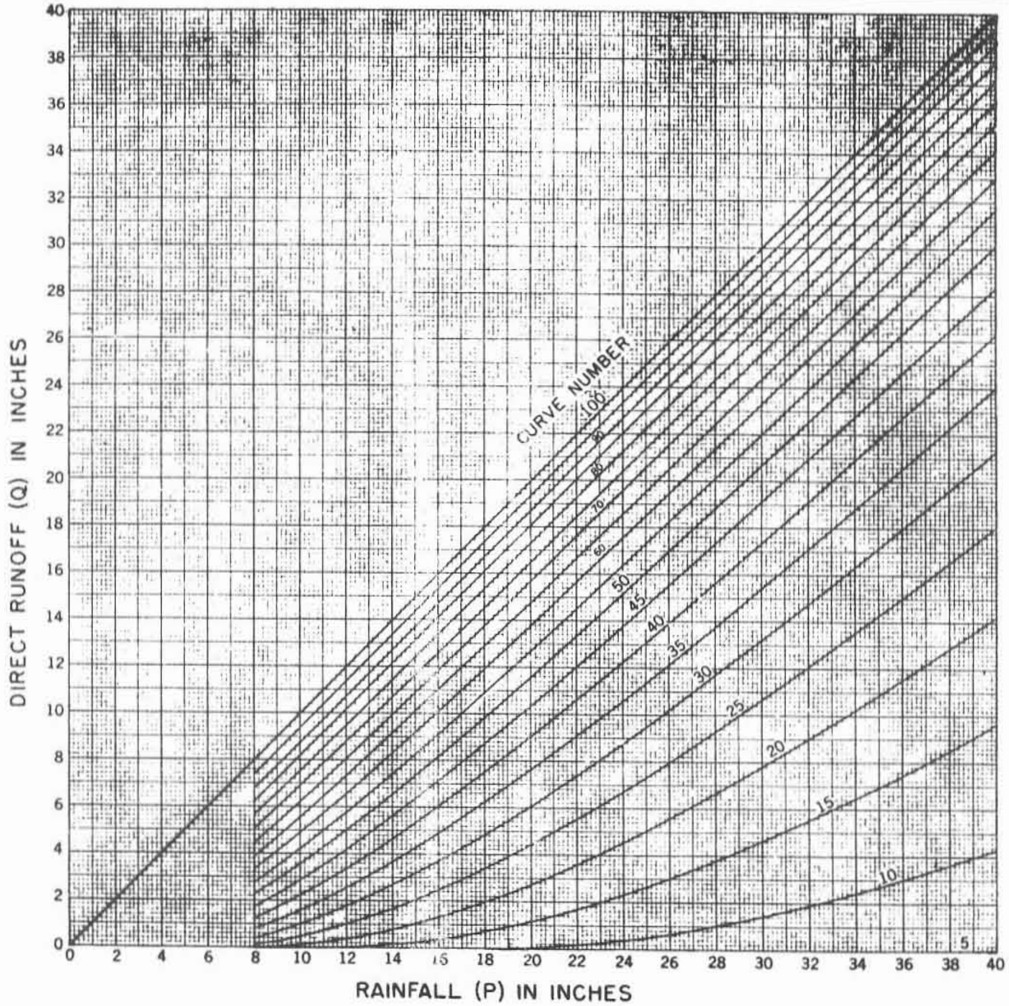
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION CENTRAL TECHNICAL UNIT

STANDARD DWG. NO. ES-1001
SHEET 1 OF 2
DATE 6-29-56

Figure 9

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q=\frac{(P-0.2S)^2}{P+0.8S}$

P = 8 to 40 inches
Q = 0. to 40 inches



REFERENCE
Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955.

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION CENTRAL TECHNICAL UNIT

STANDARD DWG NO.
ES 1001
SHEET 2 OF 2
DATE 6-27-56

Figure 9--Continued

HYDROGRAPH COMPUTATION

WATERSHED OR PROJECT Duck Creek STATE County-Dickens, Tex.

STRUCTURE SITE 5

DR. AREA 21.85 SQ. MI. T_c 3.5 HR. RUNOFF CONDITION NO. II

RUNOFF CURVE NO. 75. STORM DISTRIB. CURVE B. HYDROGRAPH FAMILY NO. 2

STORM DURATION 6 HR. RAINFALL: POINT 10.9 IN. AREAL 10.0 IN.

Q 6.87 IN. COMPUTED T_p 2.45 HR. T_o 5.13 HR.

$(T_o + T_p)$: COMPUTED 2.09 ; USED 2.0 REVISED T_p 2.56

$q_p = \frac{484\ A}{\text{REV. } T_p} =$ 4130 CFS. Qq_p = 28300 CFS.

t(COLUMN) = (t/T_p) REV. T_p. q (COLUMN) = $(q_c/q_p)\ Qq_p$.

LINE NO.	t HOURS	q CFS	LINE NO.	t HOURS	q CFS	LINE NO.	t HOURS	q CFS
1	0.	0	21	14.35	113	41		
2	0.72	113	22	15.07	85	42		
3	1.44	1130	23	15.79	57	43		
4	2.15	4810	24	16.50	28	44		
5	2.87	12100	25	17.22	0	45		
6	3.58	18300	26			46		
7	4.30	20200	27			47		
8	5.02	19200	28	Σq =	136,986	48		
9	5.74	16300	29	Δt =	0.72	49		
10	6.45	13400	30			50		
11	7.17	10500	31	Q=(0.72)	(136,986)	51		
12	7.89	6980	32	(645)	(21.85)	52		
13	8.60	4750	33			53		
14	9.32	3200	34	= 6.99	INCHES	54		
15	10.04	2120	35			55		
16	10.76	1420	36	ERROR=	1.75%	56		
17	11.48	964	37			57		
18	12.20	594	38			58		
19	12.92	396	39			59		
20	13.64	226	40			60		

Figure 10

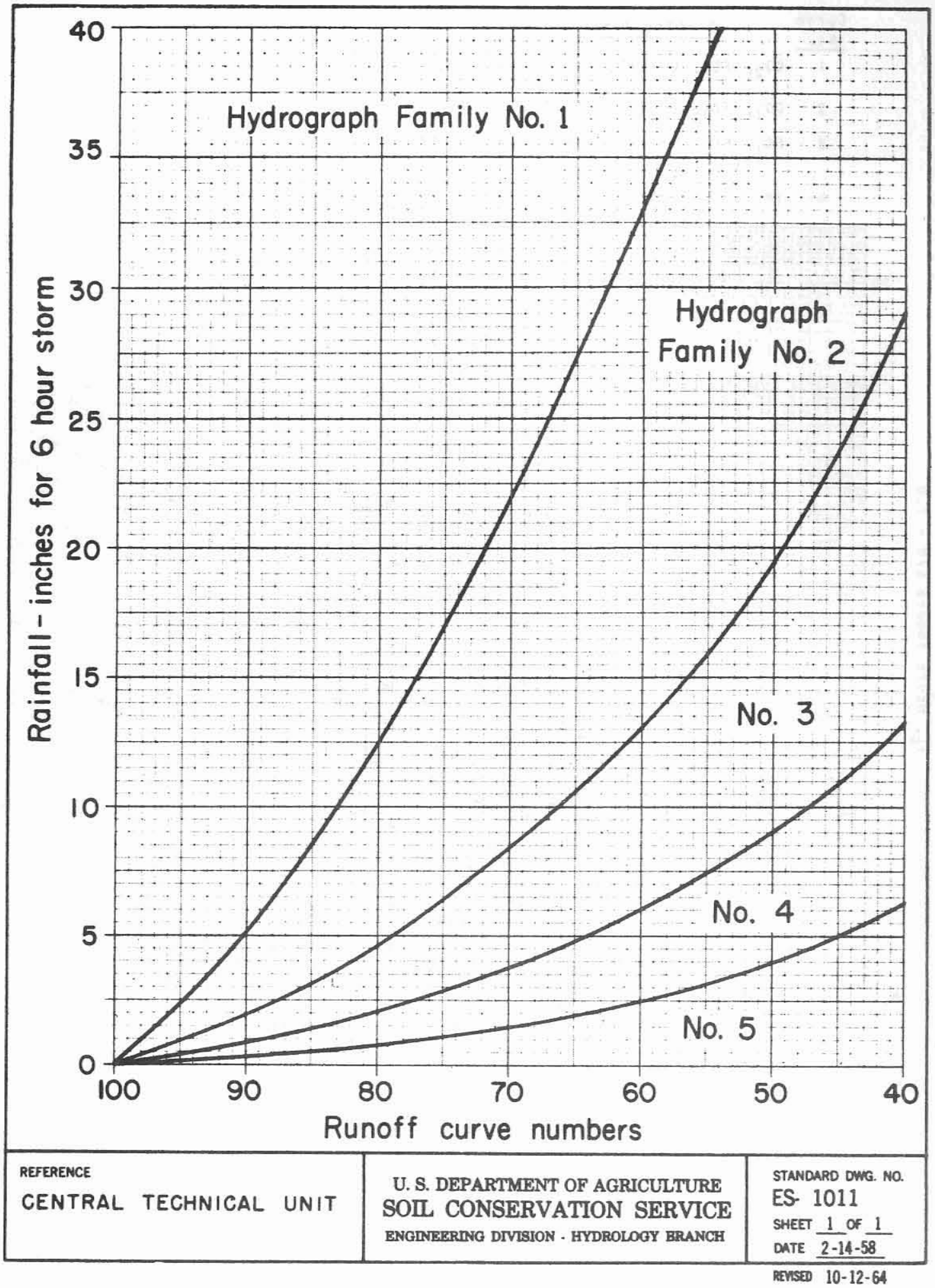
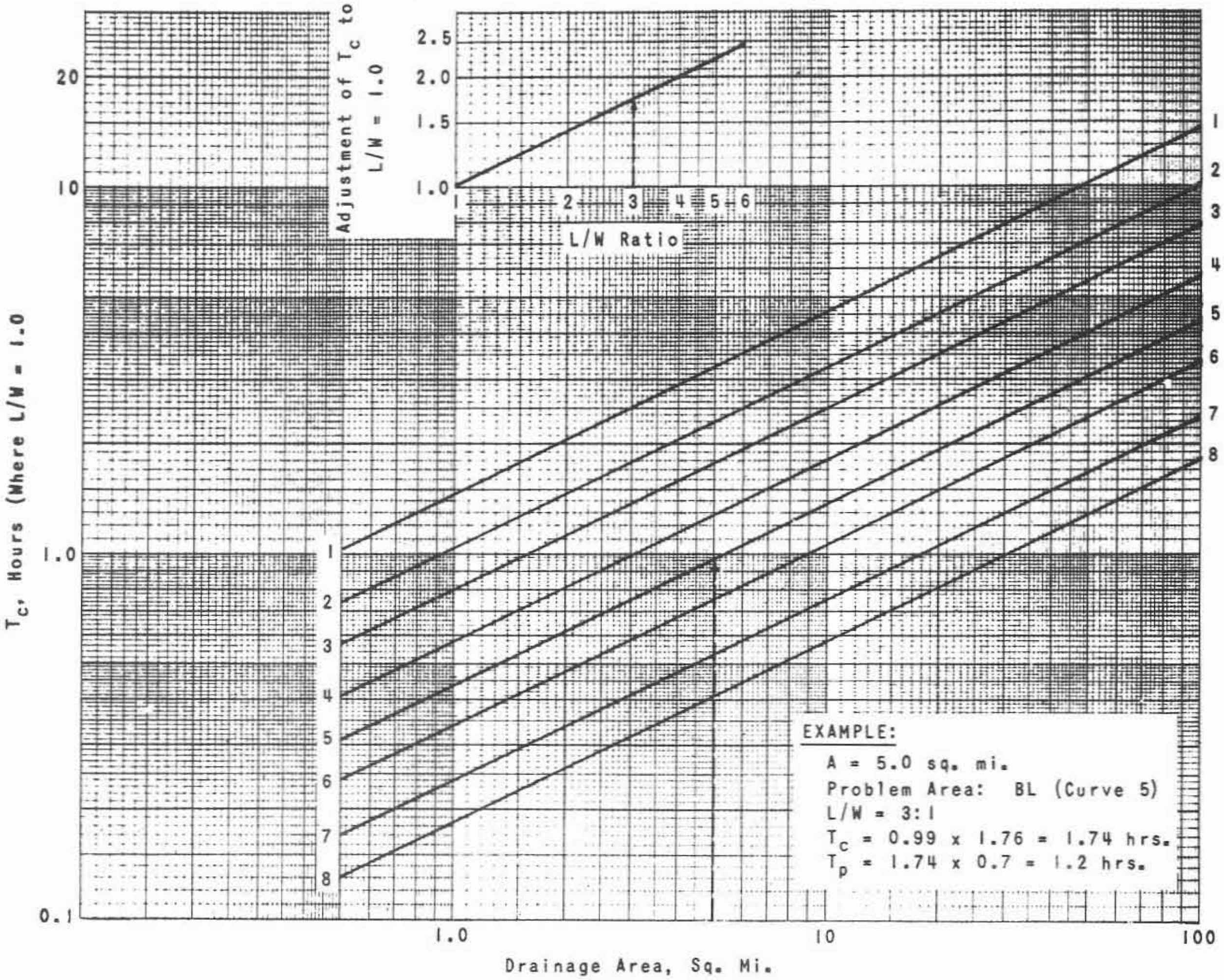


Figure 11. Chart for Selecting a Hydrograph Family for a Given 6-Hour Rainfall and Runoff Curve Number

Curve No.	Problem Areas	Curve No.	Problem Areas
1	CO_2, FC_4, LT(La)	5	BL, D^{30}, F^{10}, FC_3, GP, J^4, RG_1, RP
2	CO_1, FC_2, LH (La), LT(Ark), RG_3	6	D^{21}, D^{29}, E^9, F^9, GS(Okla), J^2, RR
3	FC_1, HP, LH(Ark), OH_2, RG_2, ZH_2	7	E^{10}, J^5
4	BH, CP, CT, OH_1, ZH_1	8	E^5, E^8, F^{11}, GS(Tex), TP, EP



DRAINAGE AREA - TIME OF CONCENTRATION
RELATIONSHIP BY PROBLEM AREAS
BASED ON CONVERSION FROM PLATE 6
IN FORT WORTH EWP-H-1 (Rev.)

Figure 12

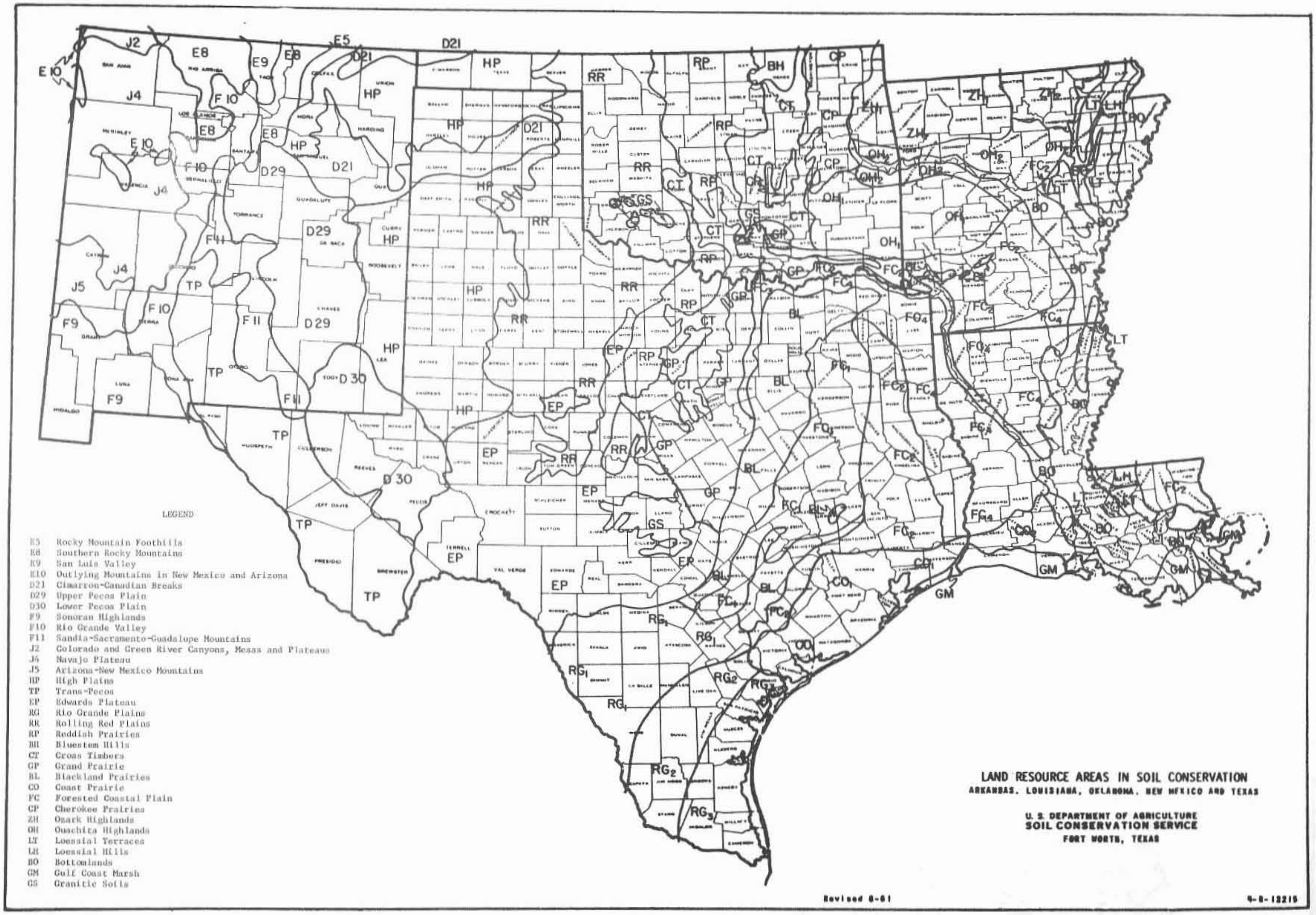

LEGEND
E5 Rocky Mountain Foothills
E8 Southern Rocky Mountains
E9 San Luis Valley
E10 Outlying Mountains in New Mexico and Arizona
D21 Cimarron-Canadian Breaks
D29 Upper Pecos Plain
D30 Lower Pecos Plain
F9 Sonoran Highlands
F10 Rio Grande Valley
F11 Sandia-Sacramento-Guadalupe Mountains
J2 Colorado and Green River Canyons, Mesas and Plateaus
J4 Navajo Plateau
J5 Arizona-New Mexico Mountains
HP High Plains
TP Trans-Pecos
EP Edwards Plateau
RG Rio Grande Plains
RR Rolling Red Plains
RP Reddish Prairies
BH Bluestem Hills
CT Cross Timbers
GP Grand Prairie
BL Blackland Prairies
CO Coast Prairie
FC Forested Coastal Plain
CP Cherokee Prairies
ZH Ozark Highlands
OH Ouachita Highlands
LT Loessial Terraces
LH Loessial Hills
BO Bottomlands
GM Gulf Coast Marsh
GS Granitic Soils
LAND RESOURCE AREAS IN SOIL CONSERVATION
ARKANSAS, LOUISIANA, OKLAHOMA, NEW MEXICO AND TEXAS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH, TEXAS
Revised 8-61
4-R-12215

Figure 13

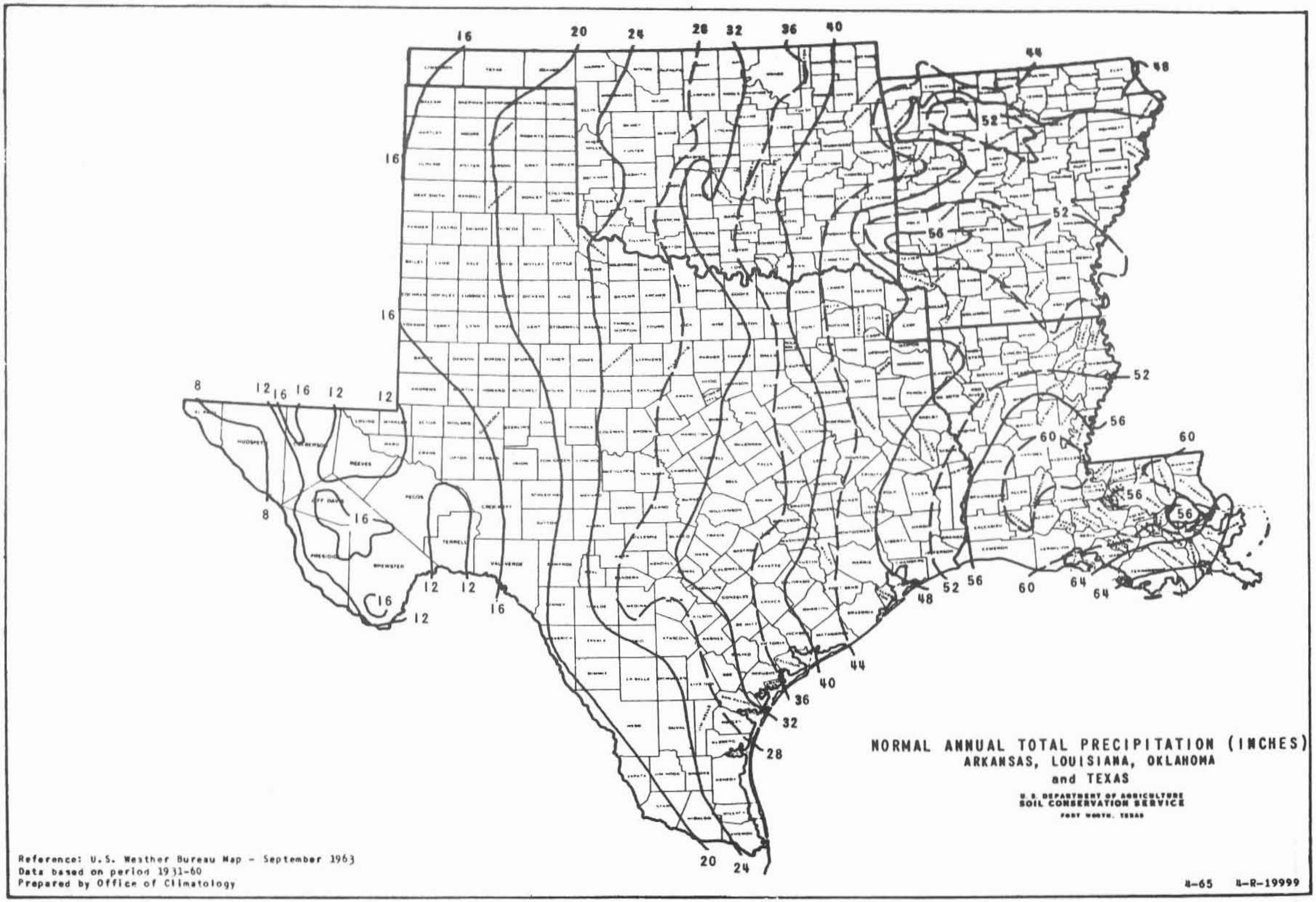

NORMAL ANNUAL TOTAL PRECIPITATION (INCHES)
ARKANSAS, LOUISIANA, OKLAHOMA
and TEXAS
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
FORT WORTH, TEXAS
Reference: U.S. Weather Bureau Map - September 1963
Data based on period 1931-60
Prepared by Office of Climatology
4-65 4-R-19999

Figure 14

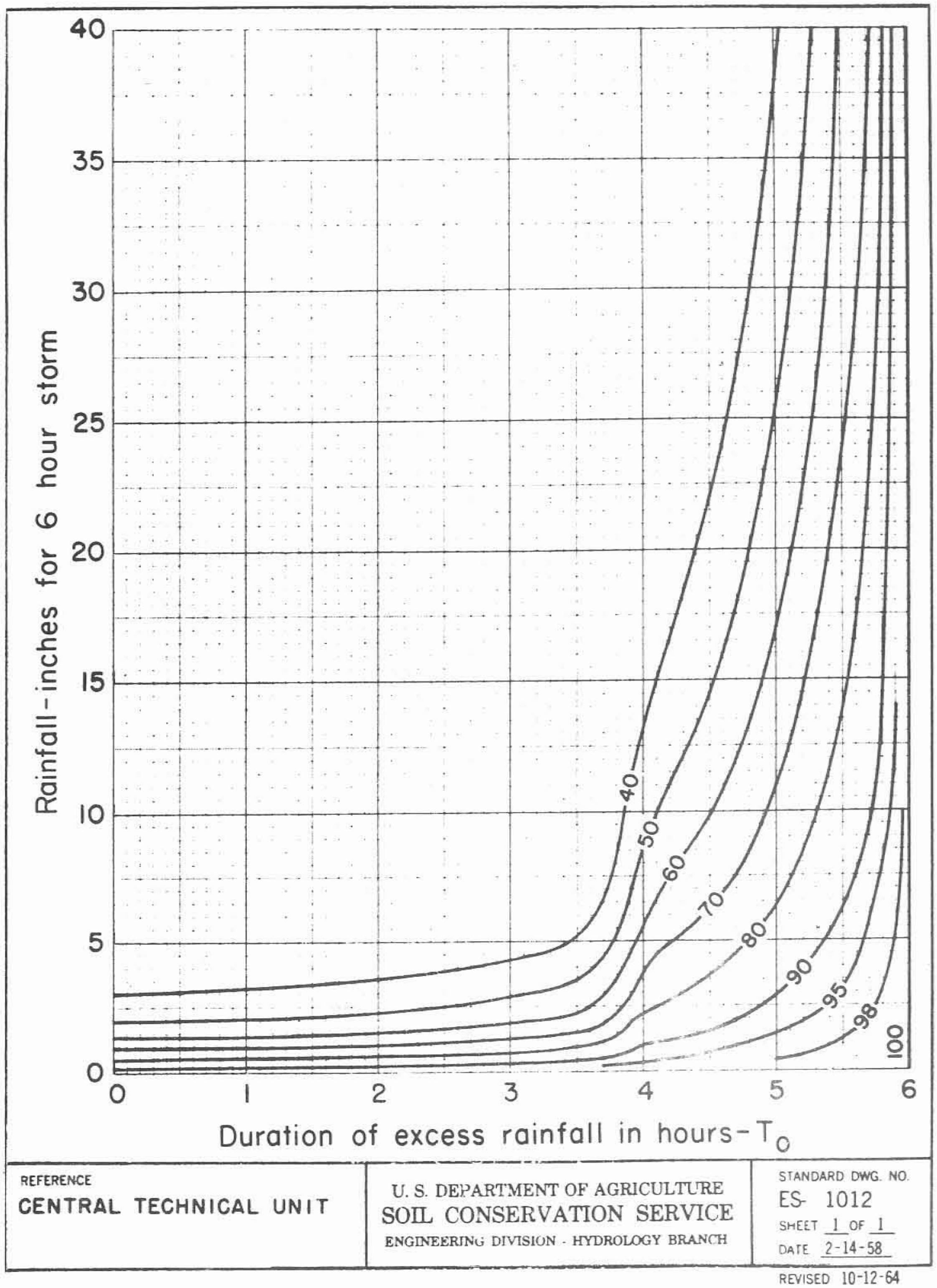


Figure 15. Duration of Excess Rainfall for Runoff Curve Numbers 40 to 100

SPILLWAY REQUIREMENTS FOR LARGE RESERVOIRS [a]

By Albert L. Cochran,[b] F. ASCE

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[a] As approved for presentation and publication by the Chief of Engineers, Department of the Army.

[b] Chief, Hydrology & Hydraulics Branch, Engineering Division, CW, Office of the Chief Engineers, U.S. Army, Washington, D.C.

Introduction

Selections of type, sizing, and the structural-mechanical design of spillways are integral elements of project design. The safety, functional effectiveness, and costs of a dam and reservoir are influenced to an important degree by these determinations.

In design studies it is usually desirable to distinguish between "regulating outlets" provided primarily for routine operation of a reservoir and "spillway" facilities intended primarily for use in discharging excess waters, inasmuch as many different design considerations are involved. However, regulating outlets and spillways usually are complementary structures. A variety of combinations have been adopted to conform with various functional needs and design advantages. Under some circumstances, regulating outlets are inoperable during severe floods because of lack of access to operating towers, or because heads exceed those for which the outlets were designed. On the other hand, some regulating outlets that are provided primarily to serve routine operating functions, before the design storage capacity of the reservoir is exceeded, are also designed to discharge "excess" waters when required, usually with the objective of reducing the frequency of emergency or limited-service spillway operations; such a structure may be designated as a "service spillway", if the capacity to discharge excess waters is a major portion of its total capacity. In contrast, some spillways are used regularly or occasionally to make routine reservoir releases associated with flood control operations or to augment downstream flows for navigation, pollution control, or other purposes.

Accordingly, in broad terms, a "spillway" may be defined as any passageway, channel, or structure designed expressly or primarily to discharge "excess" water from a reservoir after the design storage capacity and design discharge capacities of regulating outlets, turbines, and other project facilities have been used to perform normal operating functions.

The purpose of this paper is to review basic objectives, functional spillway types, alternative security standards, policy considerations, and certain assumptions affecting the final determination of spillway requirements. Your attention is invited to publications listed in the bibliography for more detailed information concerning development of spillway design flood estimates and the structural-mechanical design of spillways.

Objectives of Spillways

A spillway for a reservoir serves one or more of three principal purposes:

a. Provides security against overtopping of nonoverflow portions of the dam, acting separately or in conjunction with other outflow facilities, such as regulating outlets or power turbines.

b. Limits surcharge storage accumulations above the normal full pool elevation of the reservoir during extraordinary floods, to avoid excessive damages within reservoir flowage areas upstream from the dam.

c. Supplements or replaces regulating outlets for normal flood control operations of the project when reservoir levels are above the spillway crest elevation.

Basic Operational Types of Spillways

Controlled vs Uncontrolled Types

Many types and plans of controlled and uncontrolled spillways are used to conform with advantages and requirements of various dam and reservoir sites. A spillway is designated as an "uncontrolled" type when there are no gates, stoplogs, or other means of preventing free overflow when the reservoir exceeds the crest elevation; the terms "ungated" or "free overflow" are commonly used in the same sense. A "controlled" spillway type is equipped with crest gates, stoplogs, or other movable structures to permit various degrees of variation in outflow rates when reservoir levels exceed the spillway crest elevation; the term "gated" is usually supplemented with information to identify the structural or mechanical type of gate involved.

All spillways, regardless of type, are expected to meet the "security" objectives referred to above. Subject to this requirement, the selection of a "controlled" or "uncontrolled" spillway is governed largely by economic considerations, including due consideration of operating problems. Controlled spillways usually are more effective in limiting surcharge levels during severe floods, a very important consideration in some circumstances. For example, the gated spillway on Whitney Dam located on the Brazos River above Waco, Texas, will pass the probable maximum flood with less than 2 feet surcharge above the normal full pool level, thus reducing flowage and relocations requirements significantly. In contrast, the probable maximum flood above the Benbrook Dam, located on the Clear Fork of Trinity River west of Fort Worth, would reach a surcharge level 17 feet above the crest of its uncontrolled spillway.

Gated spillways also afford large discharge capacities for use in evacuating flood control space as rapidly as downstream channel capacities and inflow permit, often permitting a reduction in size and costs of regulating outlets. On the other hand, uncontrolled spillways have the advantage of being largely or entirely automatic in operation, and the higher surcharge levels serve to reduce peak discharge rates downstream; this is a distinct advantage under some circumstances, but the higher costs of flowage rights and relocations often preclude adoption of such plans.

Conditions and operational requirements associated with relatively small reservoirs usually favor adoption of uncontrolled spillways, but this is not always true. Quite often the combination of a moderate size "service spillway" and an uncontrolled "emergency" spillway has proven to be desirable from an operations and maintenance viewpoint, and adequate for safety.

There are no simple "pat rules" to govern the selection of an uncontrolled or controlled type of spillway for dams in specific cases. The basic functional objectives and circumstances affecting engineering feasibility and costs of alternative types of spillways should be evaluated by properly qualified engineers to assure adoption of the most economical and functionally effective overall project design for specific sites.

Spillway Service Classifications

From the standpoint of serviceability, a spillway may be designated as a service, limited service, or emergency category. A "service spillway" is any type that can be utilized without significant damage to the structure or discharge channel whenever the reservoir level exceeds the crest of the spillway. The concrete-lined spillway at Benbrook Dam would conform with this standard, as would many other large dams. Some unpaved spillways may be classed as "service" types where resistance to erosion is high enough to permit operational releases to be made through the spillway before the normal full pool elevation of the reservoir is exceeded.

A "limited service spillway" is one that may be utilized infrequently for operation of the reservoir without incurring excessive damage to the spillway structure by erosion or to downstream areas from deposition of eroded material; some extraordinary maintenance expenditures at relatively infrequent intervals would be accepted in order to reduce initial project construction costs, but not to the extent of imposing significantly adverse limitations on the optimum utilization of the controlled storage capacity of the reservoir under normal operating conditions. The spillway adopted for the Sam Rayburn Reservoir (McGee Bend Dam) on the San Angelina River is one example of a "limited service" type. An uncontrolled saddle spillway with 2200 feet crest length has a narrow concrete control weir and an unlined approach and discharge channel. To reduce the frequency of flow through the spillway and thus reduce maintenance costs, the crest of the spillway control weir was placed 3 feet higher than the normal full pool elevation of the reservoir.

An "emergency spillway" is any spillway the use of which is to be avoided as long as possible in order to prevent major damage to the spillway structure or erosion of downstream channels. Emergency spillways may involve partial control by so-called "fuse plug levees" or "flashboards," or "free overflow emergency spillways" consisting of an unpaved channel through erodible material. In some cases spillways provided with short paved control sections, with or without crest gates, may fall in the classification of "emergency spillways," if the results of operating the spillways would cause major damage to the spillway or downstream channel.

High Level (or "Perched") Spillways

Under some circumstances it is economically advantageous to utilize spillways that have crest elevations substantially higher than the normal full pool elevation adopted for the reservoir. Such spillways are commonly referred to as "high level" or "perched" types. Storage space between the adopted normal full pool elevation and the spillway crest elevation (or top of gates, if controlled) is used only when reservoir inflow exceeds the full design discharge capacity of all regulating outlets, power turbines and other facilities when the reservoir equals or exceeds normal full pool elevation. By appropriate selection of the spillway crest elevation, the storage capacity available between the normal full pool elevation and the spillway level may be made adequate to store a portion or virtually all of the spillway design flood runoff, thus greatly reducing the frequency or probability of spillway use. On this basis, an emergency or limited service spillway type is usually adequate. Adoption of high level (perched) spillways is practicable only when site and cost considerations are favorable, including flowage and relocation costs affected by the occasional high reservoir levels above normal full pool. Following are a few examples of projects in which high level spillway plans have been adopted:

Name of Reservoir	River	Location	Elevation of Normal Full Pool	Crest Elevation of High Level Spillway
Abiquiu	Chama	New Mexico	6283.5	6350.0
Bayou Bodcau	Bayou Bodcau	Louisiana	199.5	219.0
Wilson[a]	Saline	Kansas	1554.0	1582.0
Carters[a]	Coosa	Georgia	1099.0	1106.0[b]

[a] Under design.
[b] Top of gates that would control flow into unlined discharge channel.

Combinations of Spillway Types on Single Projects

Site conditions, operational advantages, and cost considerations often favor the adoption of two or more different types of spillways for a single reservoir. For example, a gated service spillway may be provided with capacity sufficient to pass floods of moderate frequency, supplemented by a separate emergency or limited service type spillway with higher crest elevation, which would be utilized only when needed to pass unusually large floods. In some cases, sections of uncontrolled spillway are designed with different crest elevations in order to provide a varying degree of flood regulation related to reservoir levels. Some low head dams are designed with a combination of gated sections and free-overflow section, designed so that flow through the gates will build up a high tailwater elevation before the reservoir level exceeds the crest of the ungated portion, thus minimizing erosion protection requirements. An innumberable number of other combinations are possible to conform economically with a range of site conditions and operational needs.

The Benbrook Dam and Reservoir project previously referred to affords an excellent example of how the functional design of regulating outlets and spillways can be coordinated to assure efficient and economical utilization of reservoir storage capacity. Gated regulating outlets are provided to permit reservoir releases to be made as required for low flow augmentation and other conservation purposes when the reservoir level is equal to or below elevation 694.0' msl, the top of conservation pool. These gated outlets are also used for flood control operations during floods of relatively frequent occurrence when the pool level is above elevation 694.0' and below 710.0', the flows being reduced to zero or as otherwise necessary for flood control purposes, particularly in the agricultural areas where non-damaging channel capacities are small. The flood control storage space between elevations 694.0' and 710.0' msl is equal to about 3.3 inches runoff from the drainage area, and is effective in regulating relatively frequent floods affecting the agricultural areas. When the reservoir level exceeds elevation 710.0' msl, flow occurs through a 100-foot wide ungated notch in the concrete spillway, with discharge rates ranging from zero at elevation 710.0' to approximately 20,000 second-feet when the pool level reaches elevation 724.0, the crest of the main spillway. If inflow rates result in a continuing rise in reservoir level, the total discharge through the spillway (including the 100-foot notch section) reaches 60,000 second-feet at a pool level of 731.0 msl, which is approximately the design discharge capacity of the Fort Worth Floodway downstream.1/ At this elevation, the reservoir would contain approximately 232,000 acre-feet of flood storage, equal to 10.1 inches of runoff from the drainage area, which would correspond to an extraordinarily large flood for the basin.

1/ Applies above mouth of West Fork of Trinity River; design capacity downstream from confluence is approximately 95,000 cfs.

The ungated notch section and spillway give a high degree of assurance that several inches of flood control storage capacity will be available in Benbrook Reservoir in the event an extraordinarily severe flood should occur that might otherwise threaten the safety of the Fort Worth Floodway levees. The gate-controlled storage capacity between elevations 694.0' to 710.0' msl assures a high degree of control of relatively moderate, frequent floods that are generally of more concern in strictly agricultural areas.

Alternative Security Standards for Spillways and Dams

The costs of dams and associated facilities are influenced substantially by the degree of security to be provided against possible failure from overtopping during floods. One objective of spillway and related safety provisions is to protect the owner's investments in the projects, and to avoid loss or interruptions in the services afforded by the project. However, an additional and usually overriding requirement for security is to protect downstream interests against hazards that might be caused by sudden failure of the dam and any ensuing artificial flood wave.

In establishing spillway requirements and other safety provisions necessary to protect an owner's economic investments and interests in a reservoir project, it is common practice to accept certain "calculated risks" in which economic losses likely to be associated with various limitations in security are balanced against the costs of increasing the security by various degrees, estimated by using some form of probability analyses. However, to assure adequate protection to downstream areas against the effects of possible failure of the dam during floods, substantially higher functional design and construction standards are usually mandatory. Functional design standards necessary to meet minimum security requirements for downstream areas at minimum costs usually conform with one of the following alternatives, the selection to be governed by circumstances associated with specific projects and downstream developments:

Standard 1: Design dam and spillway large enough to assure that the dam will not be overtopped by floods up to probable maximum categories.

Standard 2: Design the dam and appurtenances so that the structure can be overtopped without failing and, insofar as practicable, without suffering serious damage.

Standard 3: Design the dam and appurtenances in such manner as to assure that breaching of the structure from overtopping would occur at a relatively gradual rate, such that the rate and magnitude of increases in flood stages downstream would be within acceptable limits, and that damage to the dam itself would be located where it could be most economically repaired.

Standard 4: Keep the dam low enough and storage impoundments small enough that no serious hazard would exist downstream in the event of breaching, and so that repairs to the dam would be relatively inexpensive and simple to accomplish.

Policies Affecting Determination of Spillway Requirements

When a high dam, capable of impounding large quantities of water, is constructed above a populated community, a distinct hazard to that community from possible failure of the dam is created unless due care is exercised in every phase of the engineering design, construction, and operation of the project to assure complete safety. The policy of deliberately accepting a recognizable major risk in the design of a high dam simply to reduce the cost of the structure has been generally discredited from the ethical and public welfare standpoint, if the results of a failure would imperil the lives and lifesavings of the populace of the downstream flood plain. Legal and financial capability to compensate for economic losses associated with major dam failures are generally considered as inadequate justifications for accepting such risks, particularly when severe hazards to life are involved.

Accordingly, it is common practice in the United States to adhere to the policy of designing and constructing high dams that impound large volumes of water to conform with security Standard 1, as defined in the preceding subparagraphs.

Standard 2 has been confined principally to the design of run-of-river hydroelectric power or navigation dams, diversion dams, and similar structures where relatively small differentials between headwater and tailwater elevations prevail during major floods. Examples include many large structures designed and constructed by the Corps of Engineers for navigation and power on major rivers.

Standard 3 is applicable where Standards 1 and 2 are not practicable of attainment within limits of economics or other practical considerations, and where hazards to life and property downstream in the event the dam fails from overtopping would clearly be within acceptable limits. The occurrence of overtopping floods must be relatively infrequent to make Standard 3 acceptable. The "gradual rate" of breaching in such cases is usually accomplished by designing the structure to overtop where the breach of a large section of relatively erosion-resistant material would be involved, such as through a flat abutment section. The control may be obtained in some cases by permitting more rapid erosion of a short section of embankment, and less rapid lateral erosion of the remaining embankment.

Standard 4 is applicable to many small recreational type lakes and farm ponds. In such cases it is often preferable to keep freeboard allowances comparatively small, in order to assure that the volume of water impounded will never be large enough to release a major flood wave when the dam is ultimately overtopped. In some instances, adoption of Standard 4 may be mandatory, in spite of desires of the owner of a dam for a higher dam, unless much higher standards are attainable; i.e., unless appropriate safety to downstream interests can be assured, an intermediate height of structure is not justified simply to reduce the frequency of damages to the project.

Spillway Design Flood Hydrograph Estimates

The term "spillway design flood" refers to the reservoir inflow-discharge hydrograph used in estimating the maximum spillway discharge capacity and maximum surcharge elevation finally adopted as a basis for project design, assuming the initial reservoir pool elevation and general plan of water releases (through the spillway, regulating outlets, hydropower turbines and other outflow facilities) specified in the reservoir regulation plan established for use under critically severe flood conditions.

The spillway design flood used in designing projects according to functional Standard 1 should correspond to the probable maximum flood. (See References 1 to 4 for details regarding hydrograph computations.)

The Standard Project Flood (Ref. 5) has been used as the spillway design flood in connection with some major projects where functional Standards 2 or 3 are considered adequate. By definition, the Standard Project Flood (SPF) corresponds to the most critical flood event that is deemed reasonably characteristic of a particular drainage basin; in watersheds less than a few thousand square miles in size, the SPF hydrograph is usually accepted as being equal to approximately 50 percent of the Probable Maximum Flood hydrograph.

Hydrographs based on probability analyses have also been used in deriving spillway design flood hydrographs for Standards 2 and 3.

Spillway design floods for small projects corresponding to Standard 4 are usually based on rainfall-runoff probability analyses, and may represent events of fairly frequent occurrence, depending largely upon economic considerations.

Basic Assumptions Affecting Derivation and Routing of Spillway Design Floods Through Major Reservoirs

The basic objective of the overall studies of spillway requirements corresponding to Standard 1 is to establish spillway discharge capacities and freeboard allowances that will be adequate to assure that failure of a major dam from overtopping and/or wave attack on the embankment will not occur under the most severe combination of meteorological and hydrologic factors considered reasonably possible of coincident occurrence at the particular project site.

In estimating the requirements referred to above, it is considered best practice to prepare each component estimate on a reasonably conservative basis, rather than applying arbitrary "factors of safety" to final results. For example, estimates of infiltration losses should correspond to ground wetness conditions known to be conducive to extraordinary flood occurrences. Unit hydrographs should correspond to rainfall patterns favorable for rapid concentrations of runoff from the drainage basin, including consideration of probable effects of the reservoir. The reservoir level assumed to prevail at the beginning of this spillway design flood should be as high as analyses of pertinent data indicate is reasonable. Allowances for wind waves should be based on consideration of representative wind velocity characteristics of the region but not necessarily extreme short-period velocities. None of these component estimates are subject to precise determinations, but must be based on judgments governed by careful study of elements involved and general experience. It is not necessary that each component estimate correspond to the most severe magnitude possible, but each should be substantially more conservative (on the safe side) than would be reflected by averages.

The maximum reservoir level likely to be attained during the spillway design flood (inflow to reservoir) as finally estimated should be determined by appropriate routing computations. The maximum surcharge level will be governed not only by the magnitude of the flood, but also by the initial reservoir level that prevails at the beginning of inflow and the reservoir regulation plan adhered to. Flood control pools are more likely to be filled during some seasons of the year than in others, and the potential magnitude of the spillway design flood also varies by seasons. Moreover, unanticipated future changes in allocations of reservoir storage space to flood control, or revisions in permissible outflow rates because of changing flood control needs in the future, are likely to alter probabilities that a reservoir would be filled to various levels at the beginning of the spillway design flood inflow into the reservoir. Various combinations of these variables are possible.

When necessary to assure safety of a high dam, the reservoir should be assumed filled to normal full pool level at the beginning of the spillway design flood or to the highest level that is shown to be reasonably possible by appropriate studies; restrictions on reservoir releases should be assumed as the most severe that might be called for during occurrence of the spillway design flood, even though the probabilities of such severe occurrences are relatively small.

Although the safety of a major dam against failure that would jeopardize downstream interests must be assured in all cases, somewhat less severe assumptions than referred to above are useful in analyzing freeboard requirements. Under some circumstances, reservoir stage-probability estimates may be useful guides in selecting the reservoir level to be assumed as prevailing at the beginning of the spillway design flood. However, in many instances the assumption of initial reservoir levels corresponding to arbitrarily selected percentages of the flood control capacity will serve to demonstrate the effects that alternative assumptions would have on maximum reservoir surcharge levels, and may eliminate the need for more detailed studies of probable initial pool levels when the effects are relatively small or moderate. In this connection, it is usually desirable to assume, for one routing of the spillway design flood, that the design flood control capacity is 50 percent filled at the beginning of inflow.

There are several reasons for concluding that the flood control design storage capacity of a reservoir is likely to be at least 50 percent filled at the beginning of the spillway design flood, regardless of the size of the capacity involved. Normally there will be a relatively large number of floods capable of filling at least one-half the design flood control space, and most reservoir regulation plans call for optimum control of these moderate floods. In many cases, reservoir capacities originally assigned to flood control are reassigned in part to conservation or similar uses, further increasing the likelihood that at least 50 percent of the original design capacity will be filled at the beginning of the spillway design flood. It is also probable that hydrologic and meteorological conditions required for development of the maximum probable flood will be preceded by small or moderate flood runoff that would partially deplete available flood control capacities.

A comparison of surcharge elevations computed under alternative assumptions discussed above usually will reveal whether or not more detailed analyses should be made to establish the most logical starting pool level to be assumed in routing the spillway design flood. If the design flood control capacity is relatively small, there will be little difference between estimated maximum surcharge levels; on the other hand, if the flood control capacity is unusually large in comparison with normal flood runoff quantities, the assumption that the reservoir will be only half filled at the beginning of the spillway design flood would be reasonable in most circumstances. The apparent likelihood that either of these initial pool levels (full or half full) would prevail at the beginning of the spillway design flood can be taken into consideration when final decisions are reached regarding freeboard requirements for the dam, based on comparison of the effects of alternative assumptions, and other pertinent information.

In routing the spillway design flood through the reservoir under the alternative assumptions regarding initial pool level, a conservative regulation plan should be followed, conforming basically with project design objectives but recognizing that future conditions are likely to require more restrictive releases than normally anticipated during planning studies. Reservoir regulating outlets should be assumed as operating according to normal plans during the spillway design flood, provided there is reasonable assurance that the gates would actually be attended during extreme flood emergencies; if there is reasonable doubt that time intervening between occurrence of the spillway design storm rainfall and occurrence of critical reservoir stages would be long enough to assure safe operation of the outlets (particularly during nighttime), the outlets should be assumed inoperative during the spillway design flood.

Final Selection of Spillway Types and Design Capacities

Some dam and reservoir sites are so distinctly favorable for selection of specific types of spillways that alternatives need not be studied in detail. In such instances final selection of spillway lengths, crest controls, and discharge characteristics are usually based primarily on economic considerations, within reasonable operating limits. However, in all cases it is advisable to plot a curve of overall project costs against surcharge elevations that would be attained in the reservoir during the spillway design flood, assuming various differences in size or details of the spillway type under consideration. If the curve shows a distinct break in the cost relation, the most economical plan is indicated. On the other hand, if the change in cost is moderate over a substantial range in spillway length or other features, greater weight can be given to factors that are not fully reflected in construction cost comparisons.

In connection with the design of large multiple-purpose projects, investigation of several alternative spillway schemes is usually justified. Studies of alternatives should include comparisons of overall project costs, reliability and safety of operation under both routine and emergency conditions, probable operation and maintenance costs, and other pertinent factors. In some cases the importance of assuring a high degree of control over reservoir surcharge level may dictate selection of a spillway associated with somewhat higher overall project costs than would otherwise be appropriate; for example, the existence of leveed urban areas within potential reservoir flowage limits, mine entrances, major railroads or other critical concentrations of values upstream from the dam often influence the selection of spillway schemes to limit surcharge elevations to a greater extent than direct economic comparisons based on average annual or initial construction cost estimates. In other cases, the necessity of assuring a high degree of security to downstream interest against unexpected emergencies has resulted in giving preference to spillways that assure the most dependable operation under extreme flood conditions, even though higher initial costs may be involved.

All comparisons of alternative spillway schemes should be based on studies of overall project performance and economic relations, and not on separate analyses of spillways alone.

Following preliminary selection of spillway types and limiting discharge requirements corresponding to representative floods, minimum spillway discharge capacities required at a few key reservoir levels should be established as a limiting basis for computation of stage-discharge relations required for preliminary structural and mechanical design of spillway facilities to be used in establishing approximate project cost estimates. After final selection of the spillway type and general plan, the spillway stage-discharge relation should be refined as necessary to conform with the most economical and otherwise satisfactory structure design established through detailed hydraulic analyses, including model tests if practicable. The stage-discharge curve thus established should be used in recomputing the maximum reservoir level to be expected during the spillway design flood under the proposed plan of project operation, to verify the adequacy of proposed spillway design and related project features.

Freeboard on Dams

The design discharge requirements for a spillway should be based on thorough studies of flood potentialities and other factors previously discussed, without direct reliance on "freeboard" allowances above the estimated maximum reservoir surcharge level during the spillway design flood. However, freeboard provisions to provide some factor of safety against overtopping of a dam during extreme floods, which usually influences to some extent the degree of conservatism incorporated in estimates of spillway requirements.

"Freeboard" is defined as the vertical distance from a designated "freeboard reference level" in a reservoir to the design grade of an embankment, or to any specified design feature of a structure, such as the floor of an operating platform or machinery level. Hence, it is important that the freeboard reference level and the project design feature involved be specifically identified in each study.

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4	Hydrology of Spillway Design: Large Structures - Adequate Data, by Franklin F. Snyder, Journal of the Hydraulics Division, Proceedings of ASCE, May 1964 (Paper 3915).
5	Standard Project Flood Determinations, Civil Works Engineer Bulletin 52-8, dated 26 March 1952, Corps of Engineers, U.S. Army (Office, Chief of Engineers, Washington, D.C.).
6	Freeboard Allowances for Waves in Inland Reservoirs, by Thorndike Saville, Jr., E. W. McClendon, and A. L. Cochran, Transactions ASCE, Vol. 128, 1963, Part IV.

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> Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams, Civil Works Engineer Circular 1110-2-27, dated 1 August 1966, Department of the Army, Office of the Chief of Engineers, Washington, D.C.
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SOME EFFECTS OF URBANIZATION ON DESIGN FLOOD CRITERIA FOR SMALL WATERSHEDS

by

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Introduction

For the past two years the Center for Water Resources Research at the University of Texas has been engaged in a study evaluating the effects of urbanization on some storm runoff characteristics of a small urban watershed (Waller Creek located in Austin, Texas). The results of this study have just recently been published in the Water Resources Research Center Report series. Wider distribution will soon be made possible by publishing the same report as a Texas Water Development Board Report. Hopefully, in the near future an appendix to this report will be published (also as a TWDB Report) which will contain complete unit hydrographic data for each of the watersheds studied.

The purpose of this paper is to show the effects of urbanization on the unit hydrograph characteristics of the Waller Creek watershed and demonstrate the practical design aspects of the derived urban relationship.

Development of Equations

In order to describe the effects of the past and future urban development on the Waller Creek watershed it was necessary to develop empirical equations describing the hydrograph under conditions before urbanization and then under different states of urban development. This was accomplished by analyzing data from twenty-two urban and eleven rural watersheds located throughout the United States (Tables 1 and 2). The data were presented in terms of 30-minute unit hydrographs. The hydrograph properties selected are shown in Figure 1 and represent fairly conventional properties such as the time of rise (T_R), the width of the hydrograph at 0, 50, and 75 percent of the peak discharge (respectively, T_B, W_{50}, W_{75}) and the peak discharge (Q).

Time of Rise

Statistical analysis indicated that the length, slope, impervious cover and a channel parameter, Φ, best described the period of rise for an urban watershed. The channel parameter, Φ, was found necessary to describe the shorter period of rise for those urban watersheds with channel improvement. Shown in Table 3 are the three Φ classifications: 0.6 for the extensive channel improvement, 0.8 for some channel improvements, and 1.0 for natural conditions. The resulting linear regression equation based on the twenty-two urban watersheds is

$$T_{RU} = 20.8\ \Phi\ L^{0.29}\ S^{-0.11}\ I^{-0.61} \quad \ldots.(1)$$

For the eleven rural watersheds the time of rise could best be described as a function of the length and slope of the watershed

$$T_{RR} = 2.65\ L^{0.12}\ S^{-0.52} \quad \ldots.(2)$$

Peak Discharge

Statistical analysis indicated that the peak discharge for both the urban and rural watersheds could best be expressed as a function of the drainage area and the time of rise. The resulting equations are

$$Q_U = 1.93 \times 10^4\ A^{0.91}\ T_{RU}^{-0.94} \quad \ldots.(3)$$

$$Q_R = 1.70 \times 10^3\ A^{0.88}\ T_{RU}^{-0.30} \quad \ldots.(4)$$

Hydrograph Widths

Statistical analysis also indicated that the hydrograph widths for both urban and rural watersheds could best be expressed as a function of the drainage area and peak discharge. The resulting equations based on the twenty-two urban watersheds are

$$T_{BU} = 4.44 \times 10^5\ A^{1.17}\ Q_u^{-1.19} \quad \ldots.(5)$$

$$W_{50U} = 4.14 \times 10^4\ A^{1.03}\ Q_u^{-1.04} \quad \ldots.(6)$$

$$W_{75U} = 1.34 \times 10^4\ A^{0.92}\ Q_u^{-0.94} \quad \ldots.(7)$$

and for the eleven rural watersheds

$$T_{BR} = 7.41 \times 10^3 A^{0.64} Q_R^{-0.53} \quad \ldots.(8)$$

$$W_{50R} = 7.37 \times 10^4 A^{1.11} Q_R^{-1.13} \quad \ldots.(9)$$

$$W_{75R} = 4.46 \times 10^4 A^{1.06} Q_R^{-1.13} \quad \ldots.(10)$$

TABLE 1.--Data on Rural Watersheds

No.	Watershed	Storm Data	Unit Hydro-graph Data	Sources of Data
A	Calaveras, Tex.	X		U.S. Geological Survey*
B	Deep Creek No. 3, Tex.	X		U.S. Geological Survey*
C	Deep Creek No. 8, Tex.	X		U.S. Geological Survey*
D	Escondido No. 1, Tex.	X		U.S. Geological Survey*
E	Honey Creek No. 11, Tex.	X		U.S. Geological Survey*
F	Honey Creek No. 12, Tex.	X		U.S. Geological Survey*
G	Cow Bayou, No. 4, Tex.	X		U.S. Geological Survey*
H	Albuquerque, N.M.	X		Agricultural Res. Ser.(1960)
I	Bentonville, Okla.	X		Agricultural Res. Ser.(1960)
J	Guthrie, Okla.	X		Agricultural Res. Ser.(1960)
K	Stillwater, Okla.	X		Agricultural Res. Ser.(1960)
L	Freeman Field, D, Ind.**	X	X	U.S. Corps of Engrs.(1947)
M	St. Anne, 2, Ind.**	X	X	U.S. Corps of Engrs.(1947)

* Data furnished by Austin District.
** Only used for lag time and general relationships.

TABLE 2.--Data on Urban Watersheds

No.	Watershed	Storm Data	Unit Hydro-graph Data	Sources of Data
1	Anacostia, N. W., Illinois		X	U.S. Corps of Engrs.(1954)
2	Anacostia, N. E., Illinois		X	U.S. Corps of Engrs.(1954)
3	Boneyard, Illinois		X	Chow (1952)
4	Brays Bayou, Texas	X	X	Van Sickle (1964)
5	Greens Bayou, Texas	X	X	Van Sickle (1964)
6	Halls Bayou, Texas	X	X	Van Sickle (1964)
7	Sims Bayou, Texas	X	X	Van Sickle (1964)
8	White Oak Bayou, Texas	X	X	Van Sickle (1964)
9	Red Run, Michigan	X		Wiitala (1963)
10	Waller Creek at 38th Street, Texas	X		U.S. Geological Survey*
11	Waller Creek at 23rd Street, Texas	X		U.S. Geological Survey*
12	Salt Fork, West Branch, Illinois		X	Mitchell (1948)
13	Louisville, 17th Street, Kentucky		X	U.S. Corps of Engrs.(1949), Snyder(1958), Eagleson(1962)
14	Louisville, N.W. Trunk, Kentucky		X	U.S. Corps of Engrs.(1949), Snyder(1958), Eagleson(1962)
15	Louisville, Western Outfall, Kentucky		X	U.S. Corps of Engrs.(1949), Snyder(1958), Eagleson(1962)
16	Louisville, Southern Outfall, Kentucky		X	U.S. Corps of Engrs.(1949), Snyder(1958), Eagleson(1962)
17	Louisville, S. W. Outfall, Kentucky		X	U.S. Corps of Engrs.(1949), Snyder(1958), Eagleson(1962)
18	Freeman, A, Indiana	X	X	U.S. Corps of Engrs.(1947)
19	Freeman, B+A, Indiana	X	X	U.S. Corps of Engrs.(1947)
20	Freeman, B+T, Indiana	X	X	U.S. Corps of Engrs.(1947)
21	Lockbourne, 2, Ohio**	X	X	U.S. Corps of Engrs.(1947)
22	Lockbourne, 3T, Ohio**	X	X	U.S. Corps of Engrs.(1947)
23	St. Anne, 1, Indiana	X	X	U.S. Corps of Engrs.(1947)
24	Godman, 1, Kentucky	X	X	U.S. Corps of Engrs.(1947)

* Data furnished by Austin District
** Used only for lag time and general relationships.

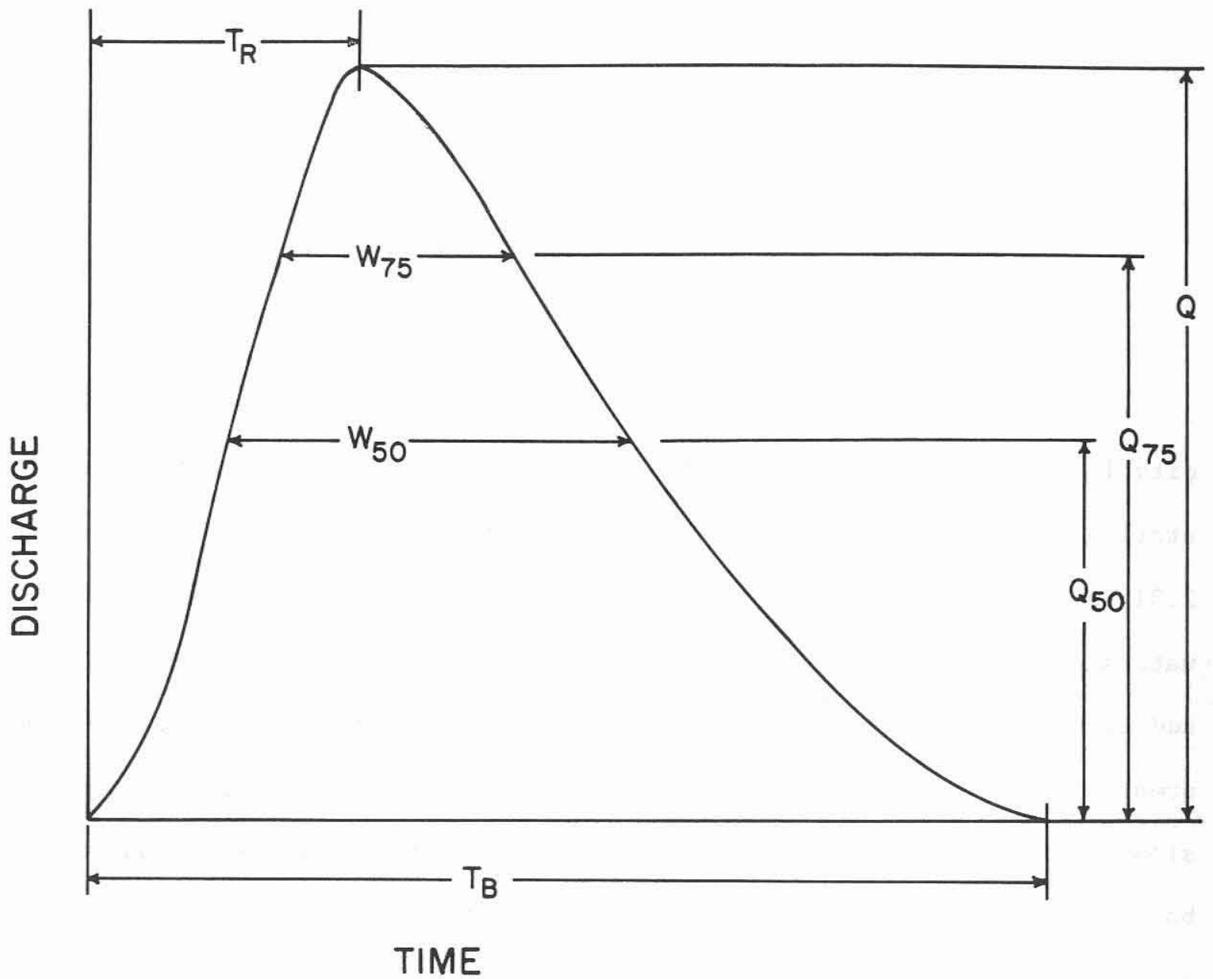


Figure 1. Definition of Hydrograph Properties

TABLE 3.--Channel Parameter, Φ, Classifications

Φ	CLASSIFICATION
0.6	Extensive channel improvement and storm sewer system, closed conduit channel system.
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.

Effects of Urbanization - Waller Creek

Waller Creek, a tributary of the Colorado River, lies entirely within the city limits of Austin, Texas. The drainage area above the Twenty-third Street station is 4.13 square miles and above the Thirty-eighth Street station is 2.31 square miles (Figure 2). Two streamflow stations are located in the watershed; one at Twenty-third Street and one at Thirty-eighth Street. In addition, there are two non-recording and three recording rain gages. The area is relatively long and narrow with a length of 27,600 feet and an average slope of 0.009 ft/ft. An extensive study was made of the impervious cover based on aerial photographs and building records. Figure 3 summarizes the chronological development of the impervious cover for Twenty-third and Thirty-eighth Streets and the area located between the Twenty-third and Thirty-eighth Street stations, ΔA, drainage areas.

Waller Creek

In order to evaluate the effects of the present urban development, the rural equations 2, 4, 8, 9, and 10 were used to determine the unit hydrograph characteristics of the Waller Creek watershed as they might have existed under undeveloped conditions. The effects of future development were evaluated for different values of impervious cover by application of the derived urban

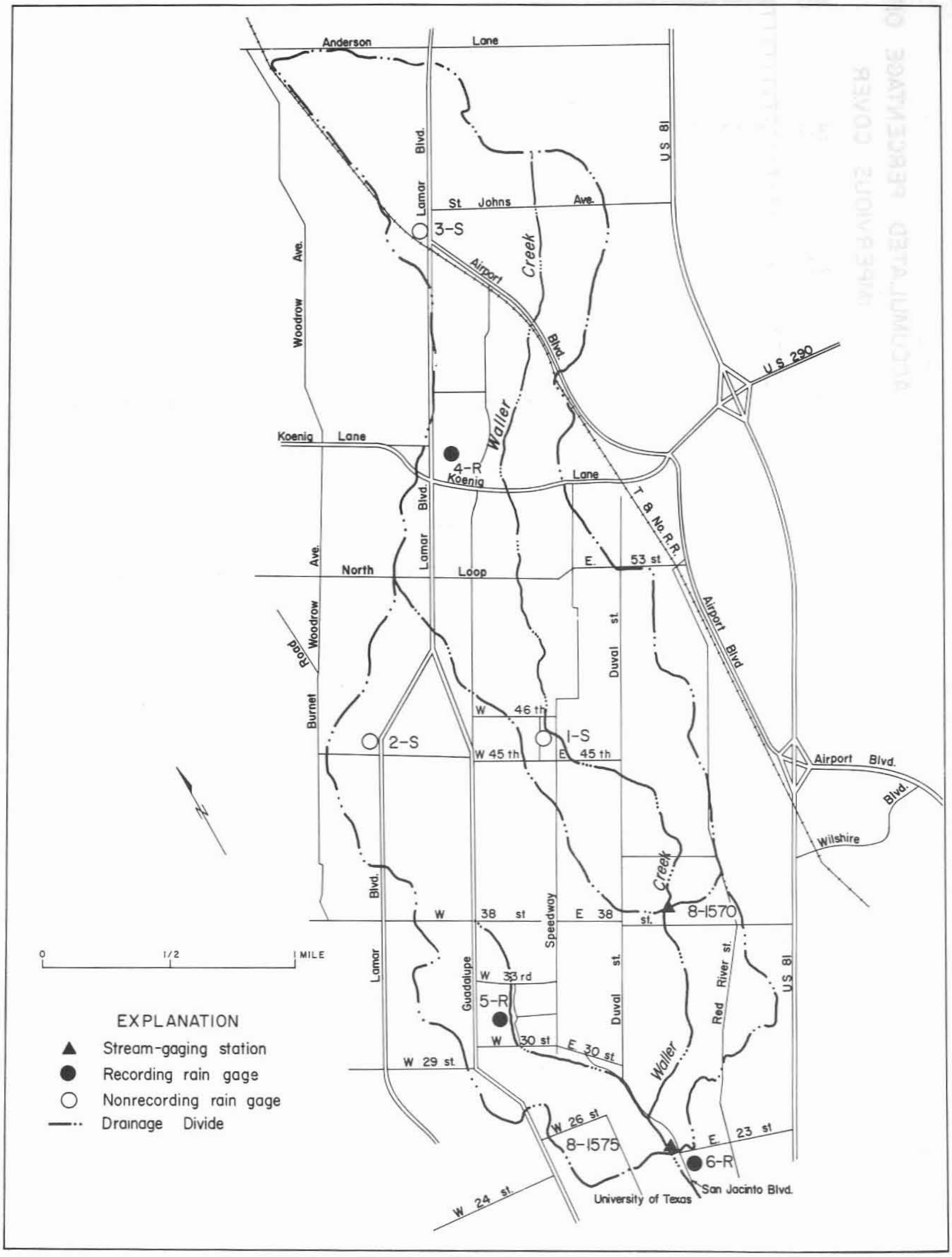


Figure 2. Instrumentation of Waller Creek Watershed

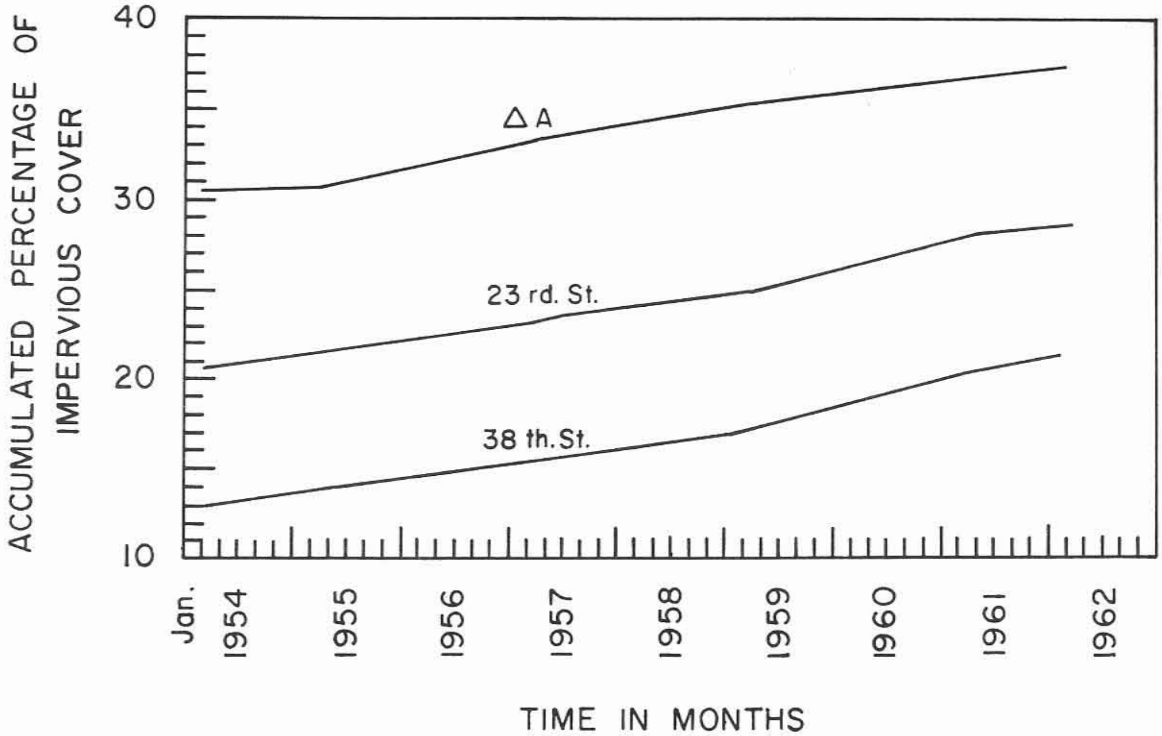


Figure 3. Impervious Cover of Waller Creek Watershed

equations 1, 3, 5, 6, and 7. The results from both the urban and rural equations are then combined and presented graphically. These results are computed for both the Twenty-third and Thirty-eighth Street gaging station locations.

1. <u>23rd Street Station.</u> The results of the application of the rural and urban equations to Waller Creek watershed at Twenty-third Street are shown in Figure 4. The effects of projected future development on the time of rise, T_{RU} were introduced by increasing the impervious cover, I, in equation 1 from its present value of 27 percent to 50 percent. The value of Φ of 0.8 is assumed constant during this development. Comparison of the present unit hydrograph and the rural hydrograph indicates that the peak discharge has increased approximately 51 percent and the time of rise has decreased 46 percent due to present urbanization as compared with rural conditions. The effect of future development will continue this same trend of increased peak discharge and reduced time of rise resulting in an increase in the peak discharge of 62 percent and a reduction in the time of rise of 52 percent at 50 percent impervious cover. The results of the effects of urbanization on the hydrograph characteristics of the Waller Creek watershed at Twenty-third Street are summarized in Table 4.

TABLE 4.--Summary of Some Effects of Present and Future Urban Development on the Waller Creek Watershed at Twenty-third Street

Stage of Development	Time of Rise (minutes)	Difference, Percent of Rural Values	Peak Discharge (cfs)	Difference, Percent of Rural Values
Rural I = 0 Percent	105	0	1,460	0
Present I = 27 Percent	57	-46	2,200	+51
Future I = 50 Percent	50	-52	2,360	+62

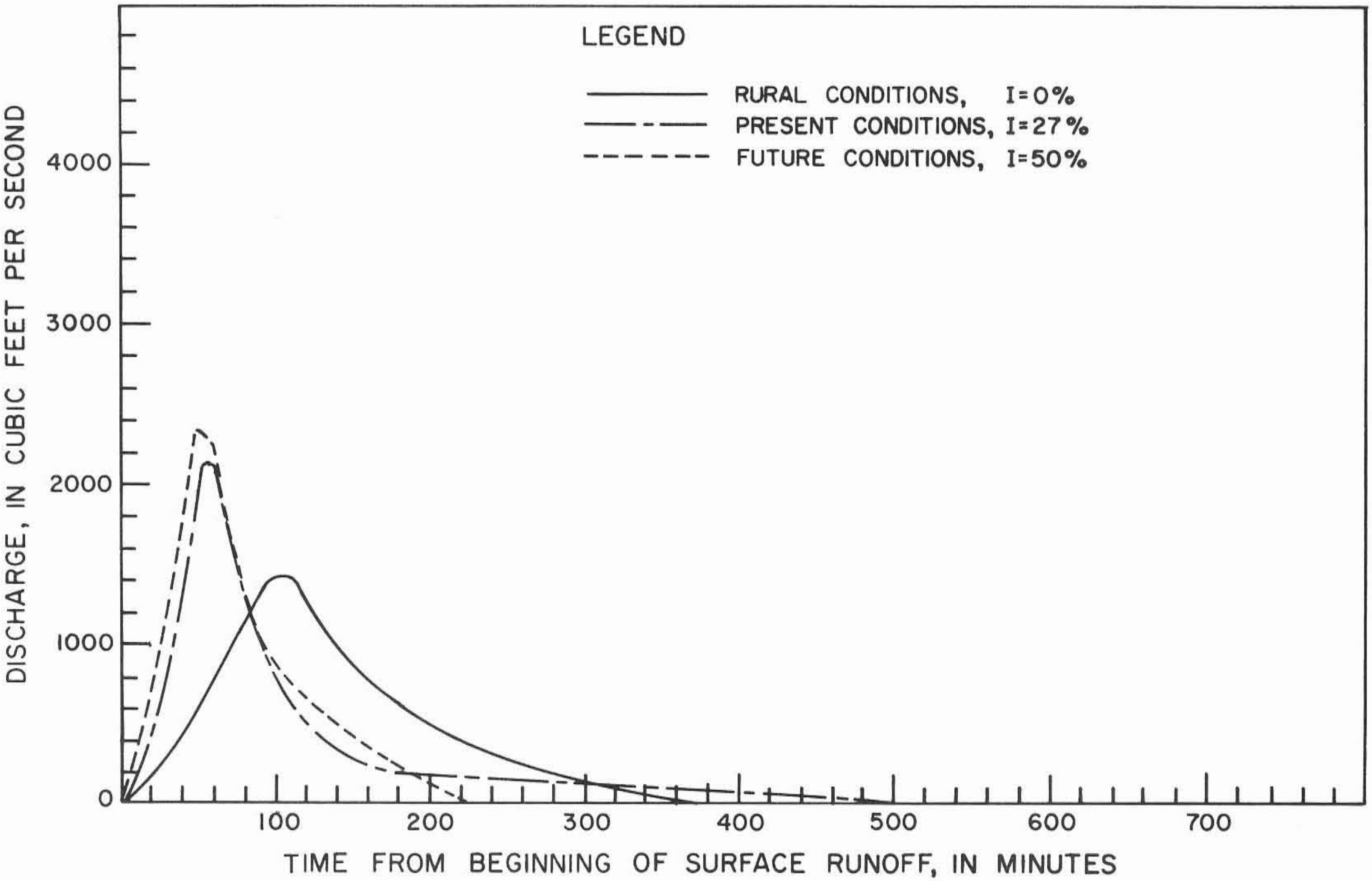


Figure 4. Effects of Future Urban Development on the Unit Hydrograph at the 23rd Street Gaging Station

2. <u>38th Street Station.</u> Similarly the rural and urban equations were applied to the less developed upper part of the watershed. Again values of the impervious cover were increased from the present value of 21 percent to 50 percent. The value of Φ of 0.8 was again assumed constant during the development. Comparison of the measured and rural unit hydrographs indicated that the peak discharge has only been increased 6 percent while the time of rise has been reduced 47 percent due to present conditions of urbanization. Future development would continue this trend resulting in a 54 percent reduction in the time of rise and 66 percent increase in the peak discharge at 50 percent impervious cover (Figure 5). The effects of urbanization on the hydrologic characteristics of the 38th Street watershed are summarized in Table 5.

TABLE 5.--Summary of Some Effects of Present and Future Urban Development on the Waller Creek Watershed at Thirty-eighth Street

Stage of Development	Time of Rise (minutes)	Difference, Percent of Rural Values	Peak Discharge (cfs)	Difference, Percent of Rural Values
Rural I = 0 Percent	103	0	880	0
Present I = 21 Percent	55	-47	930	+6
Future I = 50 Percent	47	-54	1,460	+66

Application of Urban Equations

The derived urban equations were applied to a small urban watershed which had not been used in the study in order to further test their accuracy. Unit hydrograph data were available for a small urban watershed which was part of the Louisville, Kentucky, flood control program (Figure 6). The Beargrass Creek basin contains two types of areas with totally different runoff characteristics: (1) The rural and suburban areas in the upstream portions of the

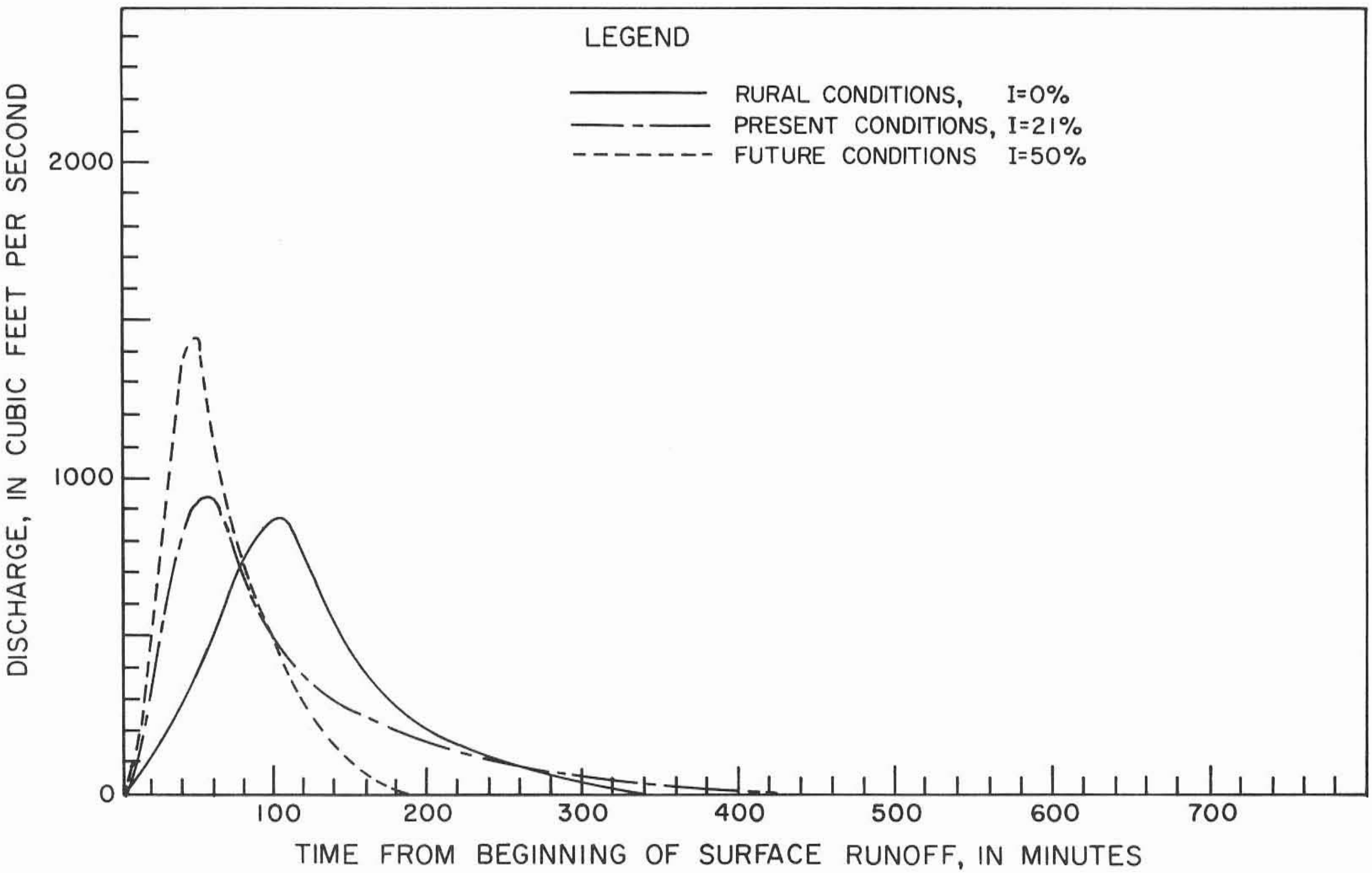
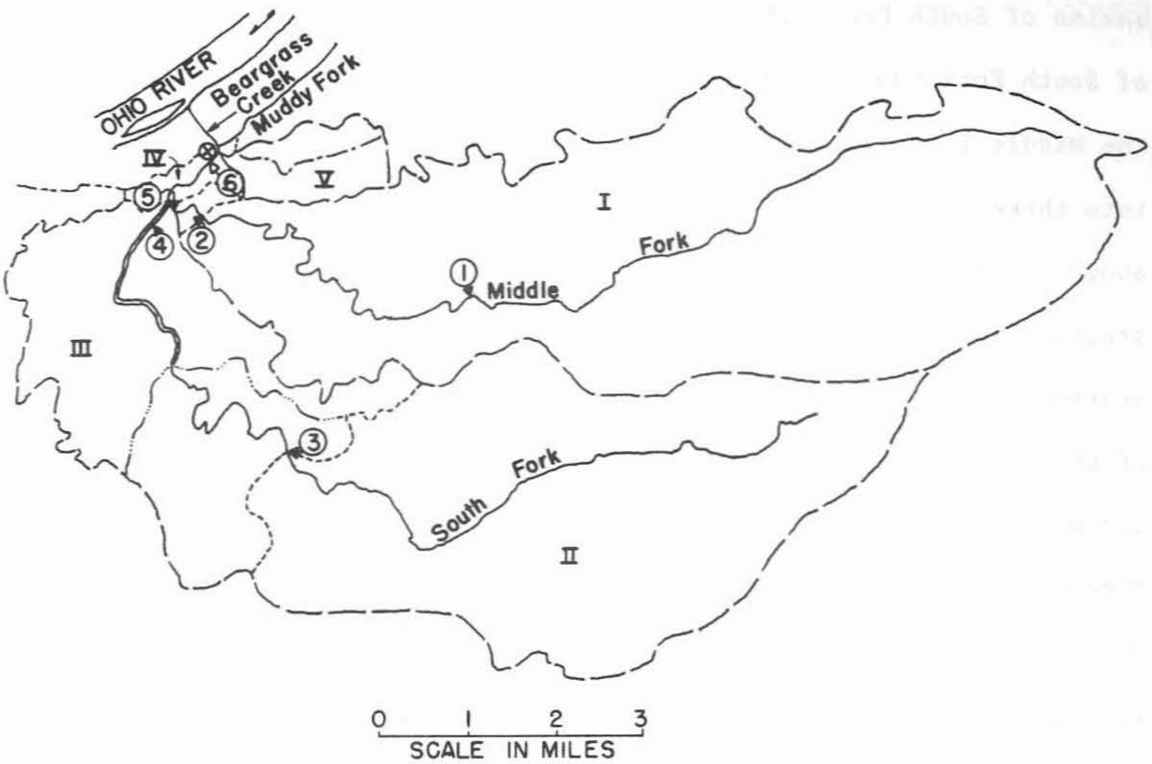


Figure 5. Effects of Future Urban Development on the Unit Hydrograph at the 38th Street Gaging Station



GAGING STATIONS

1. Middle Fork at Cannons Lane
2. Middle Fork at Payne Street
3. South Fork at Trevillian Way
4. South Fork at Baxter Avenue
5. Beargrass Creek at Main Street
6. Beargrass Creek at Brownsboro Road

LEGEND

- Drainage Divide
- Drainage Sub-divide
- Upstream Limit of Sewered Area
- Brownsboro Road Sewer
- Creek (Paved Section)
- Line of Protection
- ⊗ Proposed Pumping Plant
- ▼ Recording Stream Gage
- ▽ Non-recording Stream Gage

DRAINAGE AREAS

I Middle Fork above Payne Street (25.1 Sq. Mi.)
II South Fork above Trevillian Way (17.6 Sq. Mi.)
III Area above Main Street and below Payne Street and Trevillian Way (9.7 Sq. Mi.)
IV Area between Pumping Plant and Main Street (0.3 Sq. Mi.)
V Brownsboro Road Trunk Sewer (1.2 Sq. Mi.)

NOTE:
Unit Hydrographs have been developed from stream-flow records for areas I, II, and III.

Figure 6. Beargrass Creek Drainage Area
(After USCE, 1946)

basins of South Fork and Middle Fork; (2) Urban area in the downstream portion of South Fork basin, along the main channel, and in the downstream portion of the Middle Fork basin. The Beargrass Creek basin above Main Street was divided into three areas for the development of unit hydrographs: (1) South Fork basin above Trevillian Way gaging station; (2) Middle Fork above Payne Street gaging station; (3) Area upstream from Main Street and downstream from the gaging station at Trevillian Way and Payne Street. The rapid runoff characteristics of the areas downstream from the Payne Street and Trevillian Way gaging stations and upstream from the Main Street gaging station allowed the separation of the discharge hydrograph for the urban area between Main Street and Trevillian Way (Figure 6). This area contains 9.7 square miles, of which 6.3 square miles is intensively developed urban area with a well-developed storm sewer system. Table 6 contains various physical factors which were used in the derived urban equations to predict the 30-minute unit hydrograph. An average value of 0.7

TABLE 6.-- Physical Characteristics of the Beargrass Creek Watershed

Factor	Value
Area, A	9.7 $mil.^2$
Length, L	5.6 mil.
Slope, S	0.0012
Impervious Cover	70 percent

was selected for Φ because of the extensive channel improvement in the lower part of the watershed and sewer lines located throughout the watershed. The predicted 30-minute unit hydrograph for Beargrass Creek based on the derived urban equations and adjusted to represent one inch runoff, is compared with the measured unit hydrograph in Table 7 and Figure 7. The predicted and measured unit hydrographs were found to be in good agreement.

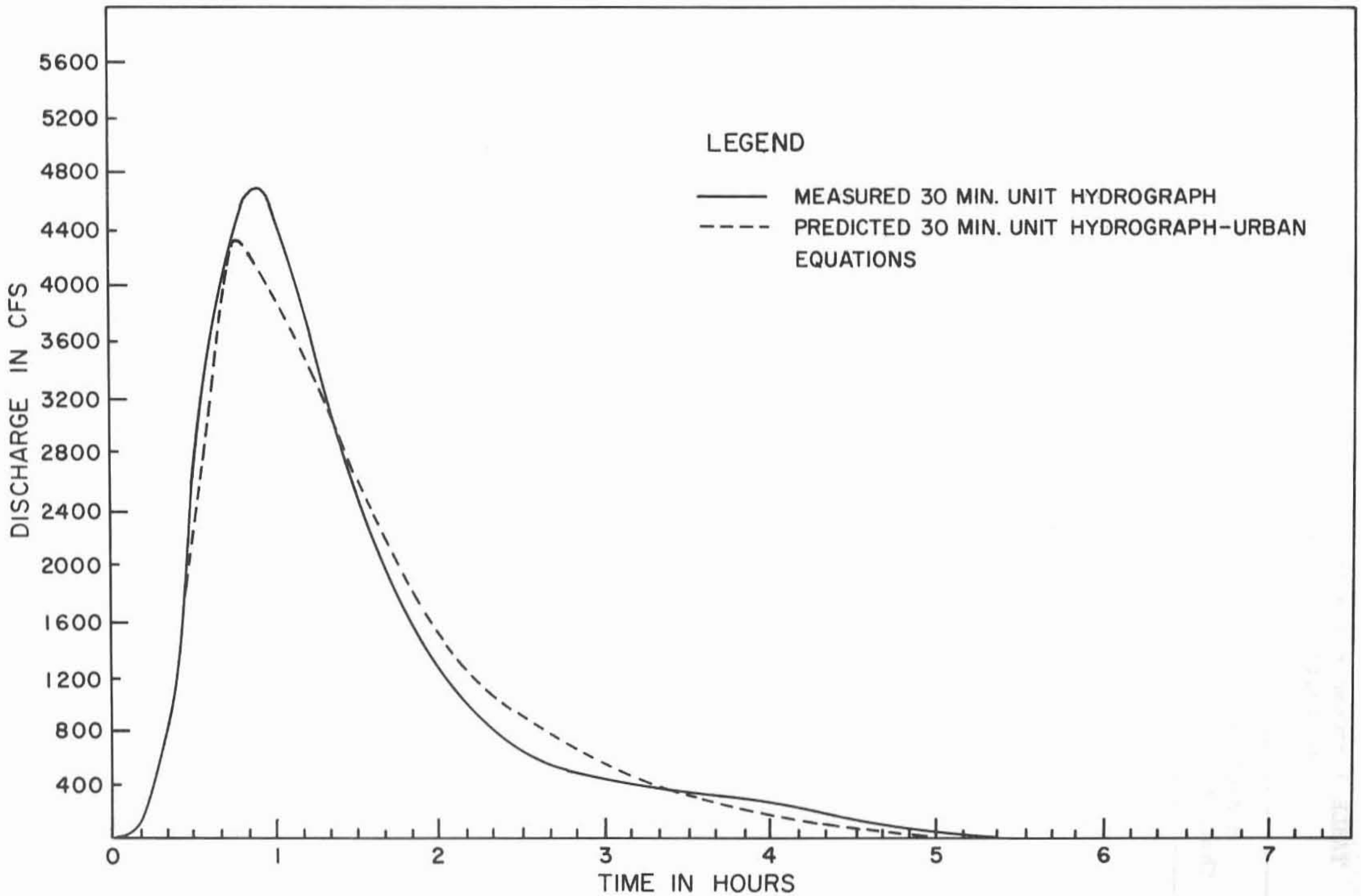


Figure 7. Comparison of Predicted and Measured Unit Hydrograph for Beargrass Creek Above Main Street and Below Payne Street and Trevillian Street

TABLE 7.--Comparison of Predicted and Measured Unit Hydrograph Characteristics for Beargrass Creek Watershed

Hydrologic Characteristics	Measured Values	Predicted Values	Difference, Percent of Measured Value
Q, cfs.	4,700	4,400	-6
T_R, min.	62	43	-31
T_B, min.	350	294	-19
W_{50}, min.	68	70	+3
W_{75}, min.	40	42	+5

Conclusions

In conclusion, the following observations can be made regarding this study:

1. Urbanization can result in considerable reduction in the time characteristics of runoff. The time of rise was reduced approximately 47 percent for both Twenty-third and Thirty-eighth Street Stations as a result of the present urban development. At a stage of urban development represented by 50 percent impervious cover, the time of rise will be reduced approximately 53 percent for both the Twenty-third and Thirty-eighth Street Stations.

2. The peak discharge of the unit hydrograph has increased 51 and 6 percent for the Twenty-third and Thirty-eighth Street Stations respectively under the present conditions of urbanization. At a stage of urban development represented by 50 percent impervious cover the peak discharge will be increased approximately 65 percent for both the Twenty-third and Thirty-eighth Street Stations.

3. The derived urban relationships can be used to predict unit hydrograph properties within ±25 percent two-thirds of the time.

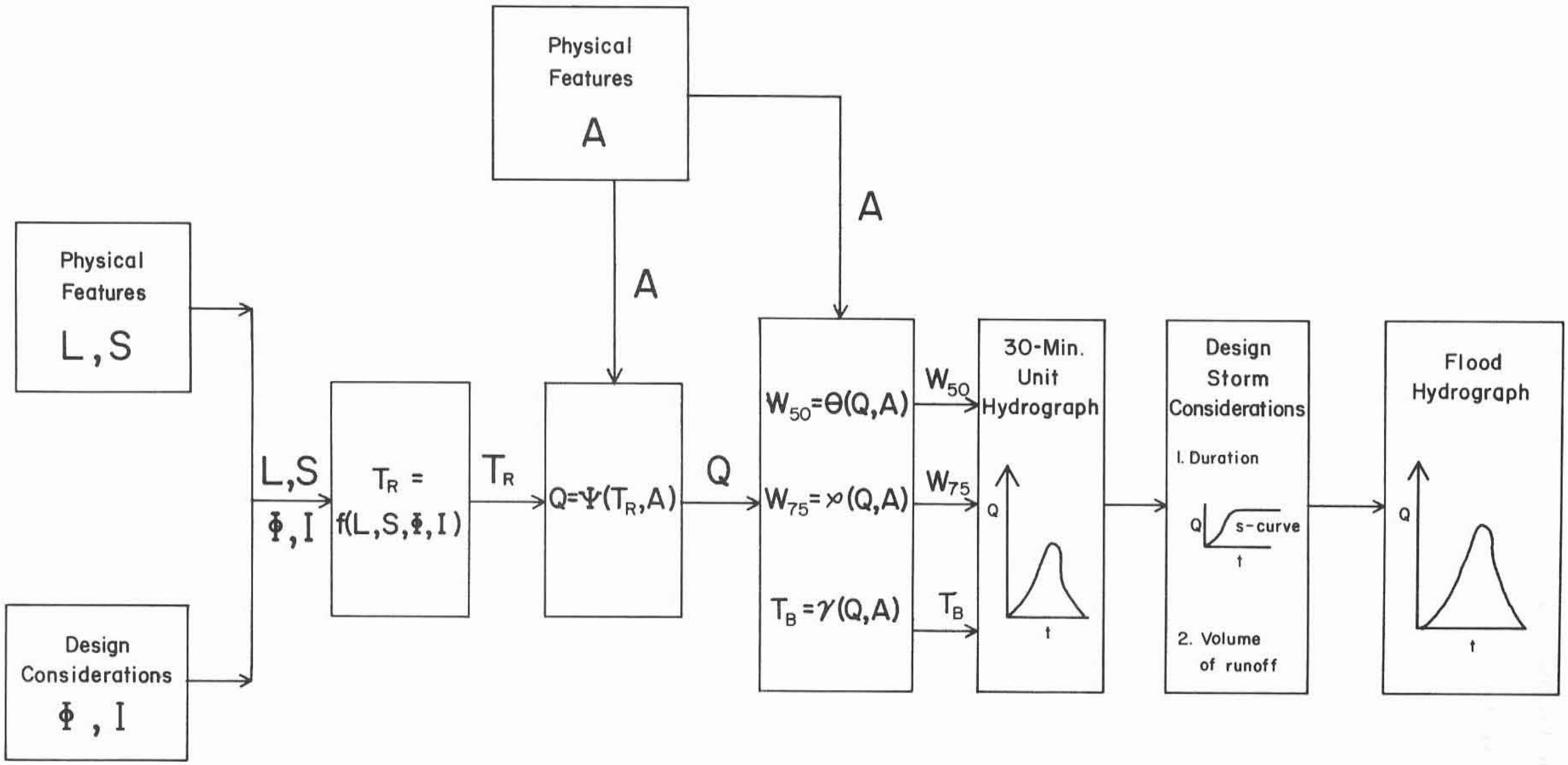


FIGURE 8. Simulation of Computational Procedure

4. The urban factors, Φ - channel improvement and I - impervious cover, can be used to reflect the effects of urban development on the unit hydrograph characteristics of a small watershed.

5. By determining only three physical factors; area, length, and slope of the watershed and selecting the channel factor Φ and impervious cover I based on present or future expected development, the 30-minute unit hydrograph can be constructed. The calculation procedure is shown in Figure 8.

It is hoped that in the near future the design equation can be extended further to cover longer duration storms and to present these equations in the form of design curves. It is planned that these results will be made available through the Texas Water Rights Commission and the Texas Water Development Board.

SOME OBSERVATIONS ON FLOOD FORMULAS AND FLOOD FREQUENCIES

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Many and varied are the formulas which have been proposed for estimating flood peaks to be expected, and more are the formulas or methods which have been put forward as the last word on estimating flood frequencies. Some of these formulas are deceptively simple, others are even more complicated than the well-known Kutter formula for the flow of water in open channels. It is not my intention to add to this list of formulas or methods, but I do wish to point out some of the pitfalls likely to be encountered in the use of published formulas and methods.

It is an unfortunate fact that engineers, like other people, are attracted by the simple formula. It is also an unfortunate fact that some of us are given to proposing formulas or methods which, as Trautwine[1] puts it, are "...buried out of sight under heaps of mathematical rubbish."

A widely used formula for flood peaks is the Myers "maximum", usually given in the form

$$Q = 10{,}000 \sqrt{M}$$

in which Q is the peak runoff in cubic feet per second, and M is the drainage area in square miles. This formula seems to have had its origin in a formula for the size of bridge openings for railroads, and the later assumption of a velocity of ten feet per second. This is a good example of the deceptively simple formula. The use of the square root of the area in square miles seems to give a measure of the distance the water will travel, and 10,000 cubic feet per second is a nice round figure to handle.

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[1] Trautwine, John C., The Civil Engineers Reference Book, 21st Edition, Trautwine Company, Ithaca, N.Y., 1937. (A reprint from the Preface to, Trautwine, John C., Civil Engineers Pocketbook, First Edition, Philadelphia, Pa., 1872.)

In 1932, 1935, and 1936, we had some very high floods in Texas, some of them so far above this Myers "maximum" as to cause me to make an intensive study of the Myers, and a number of other formulas. When I plotted peak flood flows against drainage areas on logarithmic paper, I found that the Myers "maximum" gave results twice as large as recorded flows at one square mile, about one-third of recorded flows at 400 square miles, and about four times too large at one million square miles. The plotted points indicated a bend at about 800 or 1,000 square miles, indicating that floods on the larger drainage areas followed a different law from those on the areas less than 800 to 1,000 square miles. A review of the literature available, and of the various formulas proposed showed that an exponential formula (which plots as a straight line on logarithmic paper) was the preferred type. Where the Myers formula has an exponent of 0.5 for the drainage area in square miles, the other formulas had exponents of from 0.6 to 0.875.

Some engineers have attempted to get around the discrepancies in the Myers formula by applying a multiplier, the Myers "rating" to make the predicted flood agree with the recorded flood. However, in so doing, they lose sight of the principle that if the "rating" apply to an area, it must necessarily apply to sub-areas. If, as some engineers contend, the rating changes for the sub-areas, they at once admit that the peak flood does not vary as the square root of the drainage area. To put it another way, they cannot have the rating, and at the same time have the square-root principle.

When one attempts to apply the Myers formula, and convert it to runoff rates, some peculiarities show up. If we consider an area of one square mile, the Myers "maximum" gives a peak flow of 10,000 cubic feet per second, or 15.625 cubic feet per second per acre. Applying the approximate relation that one cubic foot per second is equal to a runoff of one inch per acre per hour, this peak discharge of 10,000 cubic feet per second on one square mile would equal a runoff at the rate of 15.625 inches per hour.

On a drainage area of 402 square miles, the West Nueces River near Brackettville, Texas, in 1935, had a peak discharge of 580,000 cubic feet per second, or 2.9 times the Myers "maximum". If we apply this rating to one square mile, it would mean a runoff of over 45 inches per hour. It would be a very hardy soul who would be willing to admit such a runoff, yet it is the logical result of using the Myers formula.

It would seem that the only flood formulas giving logical results would have to be confined to a limited area of country, where climatological factors are similar, and should also be limited to a certain number of square miles.

A fundamental difficulty in making a formula to fit the larger areas, say above 1,000 square miles is that the larger the drainage area, the fewer the streams, and also the less the likelihood of a single storm covering the entire drainage area. For instance, the Ohio River may be in flood, and the Missouri at very low flow. The resulting flood in the Mississippi would be due almost entirely to the flood in the Ohio. To carry this farther, the highest floods known in the Tennessee River have come from only a part of the drainage basin. Coming closer to home, one of the greatest floods in Texas was on the Little River near Cameron, Texas, in 1921, yet the flood producing rainfall covered only about the lower one-third of the drainage area. It will be many years before we have sufficient data on which to base a curve or formula applying to these larger basins.

As the situation shapes up at present, we have to rely on published records, which are, of course, more numerous on the smaller streams. Future generations will undoubtedly procude better data, and better curves or formulas.

As to flood frequencies, again we have a plethora of formulas or methods. Generally speaking, these are based on using the record of a single stream, and by some mathematical legerdemain, predicting the size of the flood having a recurrence interval of from one to as much as 1,000 years. Some years ago, the U.S. Geological Survey published the results of studies using ten well-known flood frequency methods. These ten methods were used to predict the so-called 10-year, 50-year, 100-year, 1,000-year, and even the 10,000-year flood on the Tennessee River at Chattanooga, Tennessee, and all were based on the same 57-year record, 1875-1931. The results obtained by the ten methods differed widely, for example, the 1000-year flood varied from 372,400 to 709,300 cubic feet per second. These results were obtained despite the recorded occurrence of a flood of about 393,000 cubic feet per second in 1867. With this wide divergence in results, one may question, and with good reason, any of these formulas.

The Texas Water Commission recently issued a bulletin, No. 6311[2/], on flood peaks in Texas. In this bulletin, the estimated 50-year flood for the Neches River at Rockland, Texas, is 60,200 cubic feet per second. I have no reason to doubt the mathematics by which this result was obtained, because in the recorded history of Texas, no major tropical storm has centered over the Neches basin. This same bulletin shows the 50-year flood for stations on the San Jacinto River, with smaller drainage areas, to be from 2-1/2 to 3-1/2 times as large. The mere fact that no major flood has occurred on the Neches River since the settlement of Texas is no reason for this discrepancy. Remember, Waco could not have a tornado, but it did. The mere fact that a stream has not had a "frog-strangling" flood is no sign that it cannot have one. It may mean that a major flood is long overdue. This is not intended as a criticism of my friends on the staff of the Texas Water Commission, for

[2/] Patterson, James L., Floods in Texas, Magnitude and Frequency of Peak Flows, U.S. Geological Survey in cooperation with the Texas Water Commission, Austin, Texas, Texas Water Commission Bulletin 6311, December 1963.

they did not prepare this bulletin. However, personally, I would not advise the building of a spillway for a dam at Rockland on the basis of these data.

I am only too well aware that some of the statements I may make will cause mathematicians to say that I know nothing of mathematics. I admit the charge. I think it possible to get so engrossed with a pretty mathematical curve as to cause one to lose sight of realities. To illustrate this point, let us revert to our childhood, and have some fun with marbles.

Let us suppose that we have a box containing 10,000 marbles, of which 30 are colored. Now suppose that we take 100 samples, of 100 marbles each. It is plain that 70 or more of these samples cannot possibly contain a colored marble, since there are only 30 colored marbles to begin with. In this case, we know the total number of marbles, the number of colored marbles, and the size of the sample. Now, instead of taking 100 samples, let us take one and only one sample of 100 marbles. Who can, from this one sample, state the number of colored marbles in the box?

Now let us suppose that we have a box containing an unknown number of marbles, of which an unknown number are colored. Again, we will take one sample of 100 marbles. Now, who is willing to stick his neck out and tell us how many marbles are in the box, and how many of them are colored.

Now we can let the marbles in the box represent annual floods, and the colored marbles the largest floods. The total number of marbles then will represent the number of years for which climatological and other factors have been substantially unchanged. How many years is that? With streamflow records, even of extreme floods generally covering less than 100 years, who can say how representative the record is of the much longer period it is supposed to represent.

It is a fact which needs no demonstration that the smaller floods occur more often than do larger floods, and that the larger the flood, the fewer the occurrences. In any flood record there is always one flood that is the largest, the single, isolated event. Some flood frequency methods, notably the California method, treat this one event as having a frequency equal to that of once in the period of record. A little reflection will show that the true recurrence interval may be only once in many times the length of record, or that it may be (in a long term record) several times in the period of historic record. To put it in another way, the single, isolated event is meaningless, so far as frequency or recurrence interval are concerned. All that it means is that a flood of that size has occurred.

It seems appropriate that a few criteria for flood frequency methods be proposed. A few of these are:

1. Given a good record of floods on a stream, say for thirty or more years, the addition of a few more years of record should not make any substantial difference in the frequency analysis. Should it do so, how do we know that the second curve is any better than the first, or that, given a few more years added to the record, a third curve will not show up.
2. Given a flood record of thirty or forty years, cut the record in half, compute the frequency from the first half, and from it predict the second half.
3. If it be possible to predict the 100-year flood from a record of twenty years, then one must be able to predict the same 50-year flood from any 10 consecutive years in this 20-year record. This may cause my mathematically minded friends to recoil in horror, but in each case, it is a matter of extrapolation to a period five times the length of record, and if the method of extrapolation be valid, it must apply to both cases. Should it fail to do so, it is purely a

case of mathematical legerdemain, and merely proves that the method is not correct. We have had only too many cases of 100-year floods being exceeded three times in 4 consecutive years.

To design a dam, or a spillway, or a floodway, or a bridge on such data, unless tempered with a large admixture of recognition of realities, is merely to invite disaster. Nature has neither rewards nor punishments, only consequences.