FLOODS

Their Hydrology and Control

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PREFACE

During the last three decades, recorded flood damages in the United States have totaled over two billion dollars. Floods have also resulted in the loss of many lives. Losses reached an amount of over half a billion dollars for the single year of 1937, chiefly in the Mississippi and Ohio River Basins, and have averaged probably a hundred million dollars yearly, including the total area of the United States subject to floods.

Over half the total flood losses occurred in the three years 1936–1938, causing a strong popular demand for remedial legislation by Congress and resulting in the recognition of flood control as a proper function of the Federal government.

In the decade 1936 to 1946, authorizations by Congress of Federal appropriations for flood-control projects have totaled about three billion dollars, with another billion for proposed postwar work. Actual funds appropriated have been about 1.5 billion dollars. Thus a total amount of about four billion dollars now represents the proposed flood-control program, now about one-third funded. This amount is likely to be materially exceeded in the future because of the adoption of further projects.

A vast amount of project investigation has been undertaken by the U.S. Engineer Corps and published in part in the many Congressional documents relating to flood control. The author has attempted to review these in order to show the methods and some of the routine of their work and to make it available to other engineers. Obviously, in such a broad field, limitation to some of the more interesting and typical projects has been necessary. The results are in the nature of a progress report, since much of the program has not yet reached the construction stage.

The basic features of flood hydrology and their application to the problems relating to control projects are included, and a somewhat detailed description is given of the more important great floods in the United States and their resulting damage.

An analysis of the use of reservoirs for flood control is given in some detail, as well as the features of local flood-protection work by means of walls and levees and pumping plants. A group of typical problems relating to flood-control projects is also appended.

The author extends grateful acknowledgment to the U.S. Engineer Corps, including the office of the Chief of Engineers as well as many of the

district offices, for much information and assistance, freely given; to the Mississippi River Commission and to Gerard H. Matthes, formerly head engineer and director of the Vicksburg Experiment Station, for assistance relating to the Lower Mississippi River; to the engineers of the Tennessee Valley Authority for their kind cooperation; to the Honorable Edith Nourse Rogers, of the House of Representatives, Washington, D.C., for her invaluable help in securing Congressional and other documents; and to the many others who have assisted in supplying information.

H. K. Barrows

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CHAPTER 1

HISTORICAL—TYPES OF FLOODS

Historical

No systematic records of discharge exist on any river in any country for a period of as much as a hundred years. In Europe, records of flood heights for several hundred years are available on many rivers, but they are limited to notably high floods in the earlier centuries. Even longer stage records have been kept in India and China, but these are of little or no value as an index of discharge, because of uncertainty as to the effect of channel variations during the period of record.

On the Seine River[1] in France, water stages have been observed at La Tournelle bridge in Paris as follows:

Year	Stage, ft	Year	Stage, ft
1649	25.2	1764	24.0
1651	25.7	1802	24.4
1658	28.9	1836	21.0
1690	24.8	1876	21.3
1711	25.0	1910	27.6
1740	26.0		

The maximum flood occurred in 1658, with a stage of 28.9 ft. Some evidence exists, however, of a somewhat greater flood in 1611 or 1615, with a stage of about 30 ft. The 1658 record is called the maximum, however. The next largest flood, according to this table, was that of 1910. Some authorities claim, however, that there were intervening floods of greater discharge although of less height on the gauge because the channel of the Seine had a greater carrying capacity than in 1910.

Records of floods on the Danube[2] extend back to the year 1000, with the greatest in 1501, the second greatest in 1787, and the third greatest in 1899.

The Hydrographic Bureau of Austria concluded that a record of 100 years is not long enough to fix the maximum flood and that a record of 500 years or more is scarcely sufficient. For example, the 1899 flood,

[1] *Miami Conservancy District Tech. Rept.*, 2d ed., Part V, p. 320.

[2] *Ibid.*, p. 318.

the maximum of the nineteenth century, might be taken as the maximum to determine the channel capacity. It was the maximum for over 100 years, yet the flood of 1501 had a 33 per cent greater maximum discharge at Vienna.

On the Tiber River[1] at Rome, the first flood recorded was 413 B.C., and the greatest was that of A.D. 1598. The second greatest flood was in 1870.

There seems to be no tangible evidence in any of these records over long periods, taking them for what they are worth, that there have been any progressive changes in the causes or extent of possible floods. There is certainly no definite pattern of their time of occurrence.

The longest continuous record of *annual* flood heights now available was made at the Roda gauge, or Nilometer, on the River Nile at Cairo, Egypt, covering some 13 centuries of time. This record was studied by C. S. Jarvis,[2] whose results showed no apparent well-defined or progressive changes during this long period except such as could be explained by the effects of sedimentation, erosion, or regulation upon gauge readings.

Systematic records of river discharge in this country for the most part date back only about 40 years. In New England, the discharge of the Connecticut River at Holyoke, Mass., and vicinity has been determined since 1880, or about 60 years. Gauge-height records for yearly floods at Hartford, Conn., since 1828 are available continuously, with some intermittent information prior to that date. A record of discharge has been made from 1848 to 1861 and 1866 to the present time, or nearly 90 years, on the Merrimack at Lowell, Mass. This is the longest record of discharge now available in the United States.

The Susquehanna River at Harrisburg, Pa., has records of discharge since 1890; the Potomac at Point of Rocks, Md., since 1895; the Roanoke River at Roanoke, Va., since 1896; the James River at Buchanan, Va., since 1895; the Tennessee River at Chattanooga from 1874 to 1913 and 1915 to the present—about 65 years.

Records of yearly flood stages are available on the Mississippi River at St. Louis since 1861, with some prior data; at Memphis, Tenn., and Vicksburg, Miss., since 1871. These records of stage, owing to unstable channel conditions, are only approximately related to discharge.

The Upper Mississippi at St. Paul has a stage record since 1887, with data of daily discharge since 1892, or nearly 50 years.

In the West, the Colorado River at Yuma, Ariz., has a stage record since 1878, with daily discharge records since 1902.

The work of stream gauging and determination of daily-flow records is

[1] *Ibid.*, p. 324.

[2] Flood Stage Records of the River Nile, *Trans. A.S.C.E.*, Vol. 101, pp. 1012–1071 1936.

now being carried on in the United States in a comprehensive manner by the Water Resources Branch of the U.S. Geological Survey; and in July, 1945, there were 5,600 gauging stations in use for this purpose. The results of its work are published yearly in the *Water Supply Papers* and will become, as time goes on, the essential and increasingly useful basis for the study of river flows and floods in this country.

The work of river gauging is also being supplemented cooperatively by an increasing number of states; and more accurate determination of precipitation is now attained by the U.S. Weather Bureau and other agencies, including its extent and distribution and types of storm patterns. This makes it possible to study, in connection with runoff data, maximum storm rainfalls and limitations in floods likely to occur upon a given area, information that is often essential where past records of unusual floods are not available.

Flood history in the United States may well be said to have begun on the Lower Mississippi River (see Chap. 10) where as early as 1717 some of the first levees to hold back floodwaters were built by the French. Since that time, periodic great floods, as in 1849–1850, 1927, and 1937, have stimulated efforts toward more efficient flood protection. These culminated in the Federal Flood Control Acts of 1928 and later, whereby levees were made the principal line of defense against floods, supplemented by a comprehensive system of flood-control reservoirs upon the Ohio, Upper Mississippi, Missouri, Arkansas, Red, and White Rivers, and other tributaries.

The Mississippi River Commission, formed in 1879, functioned for many years in the control of the Mississippi River Basin; but, since 1935, the U.S. Engineer Corps has been in control.

One of the early flood investigations was made in 1912 by the Pittsburgh Flood Commission. This included a proposed reservoir system and a river wall at Pittsburgh, which lies at the junction of the Allegheny and Monongahela Rivers and is subject to periodic damaging floods. This project was not, however, carried out.

Los Angeles, Calif., and its vicinity has long had serious flood problems and in 1915 enacted the Los Angeles County Flood Control District. This agency (including the neighboring cities and towns) has planned and constructed many flood-control dams and many miles of channel improvement, and the work is now under the direction of the U.S. Engineer Corps (see Chap. 15, page 318).

The Ohio flood of March, 1913, resulted in the first large-scale attempt in the East (aside from the Mississippi River) to prevent floods and in the formation of the Miami Conservancy District. This agency between 1917 and 1925 constructed five large retarding-basin type of reservoirs as well as some local channel works in the Ohio River Basin, for the protection of Dayton, Hamilton, and other municipalities. The project was excellently planned and carried out and has successfully accomplished its purpose (see Chap. 11, page 233).

After the flood of June, 1921, upon the Upper Colorado River near Pueblo, Colo., the flood problem was met and solved by the construction of a barrier dam and local channel works. It is of interest to note that this work was under the direction of the Morgan Engineering Co., who were the engineers for the Miami project (see Chap. 11, page 238).

The flood of 1927 in Vermont resulted in action by the state to investigate this disaster, which included the loss of 84 lives and damages in Vermont aggregating over 30 million dollars. The Advisory Committee of Engineers appointed in 1928 made a comprehensive study of the flood[1] and planned flood-relief reservoir systems for all the rivers of importance. It was found that power reservoirs were best adapted with flood control as a by-product. Three of these reservoirs have since been constructed by the U.S. Engineer Corps; but only one is utilized for power, the others being of the retarding-basin type.

The state of New Hampshire during 1933–1934 had an investigation made of flood-control reservoirs and formed a Water Resource Board to act in such matters. Under its direction, the large Pittsburg Reservoir upon the Upper Connecticut River was constructed for power storage and incidental flood benefits, financed by the utility company upon the river, but built under state direction.

During the last decade, the country as a whole has become "flood-conscious" because of the prevalence of unusual and destructive floods over wide areas. These began with the New England flood of November, 1927, in Vermont and western Massachusetts, followed by the New York State and Texas floods of 1935, the March, 1936, floods in the Atlantic states and New England, the Ohio and Mississippi River floods of 1936 and 1937, and finally the Los Angeles flood in March, 1938, and the so-called "hurricane flood" in New England in September, 1938.

This has resulted in legislation by Congress making flood control a matter for Federal action, with the preparation of a widespread program of flood-relief projects, now being planned and in part under way, under the direction of the U.S. Engineer Corps.

Types of Floods

A flood may be defined as an overflow or inundation coming from a river or other body of water. In the case of a river, it may be due to such

[1] *Reports of Advisory Committee of Engineers upon Flood Control*, Dec. 15, 1928, and Dec. 15, 1930.

natural causes as excessive rainfall or melting snow or temporary ice obstructions in the form of jams. There have also been floods due to overtopping or failure of reservoir dams, with the sudden release of large volumes of water, causing great damage and often loss of life.

Adjacent to the ocean, floods may be caused by abnormally high tides or tidal waves due to wind and storms or hurricanes, as at Galveston in 1905 and in southern New England in September, 1938. In this book, attention will be confined, however, to the study of floods occurring upon rivers and watercourses.

Evidently a river may be in flood over a wide range of stages. The average yearly flow of a stream in the humid regions of the United States is equaled or exceeded from one-third to one-fourth the time, which includes all periods of high water and floods. A "bank-full" stream, or the stage at which a river is about to overflow to its flood plain, is sometimes considered as indicating the flood stage of a river.

In measures for flood relief, the stage at which damage of consequence begins to occur is of importance, and lesser floods may be disregarded. The range of floods between this "no-damage" stage and the maximum or great flood constitutes those floods which must be taken into account in studying flood benefits from flood-relief projects.

Floods Due to Natural Causes. The usual cause of floods is excessive rainfall, which, if intense and prolonged, as it has often been in some of the great storms of the past, may result in catastrophes of major importance. When the storms occur with snow upon the ground accompanied by melting temperatures, as in March, 1936, in the Atlantic states, flood flows may be greatly increased and damage widespread. In this flood, ice jams also occurred at numerous points, increasing river stages and adding to the resulting damages.

The condition of the ground at the time of a storm is also of importance in respect to flood runoff. Thus, in the Vermont flood of November, 1927, the previous month had been one of generally ample rainfall, the ground was well saturated, and hence the flood runoff was high.

In contrast with this, in October, 1895, a storm occurred in eastern Massachusetts and Connecticut, as well as Rhode Island, with approximately the same amount of rainfall as that in Vermont in 1927.[1] Preceding the October, 1895, storm, however, had been a period of 55 days with about half the usual rainfall, and rainfall considerably below the average as well during the earlier portion of the year. The significant result was little or no flood damage due to the October storm, since runoff was relatively low because of previous droughts.

Snow on the ground in the 1936 flood represented a water content of

[1] *Jour. N.E.W.W. Assoc.*, June, 1928, pp. 167–168.

from 2 to 4 in. in the Middle Atlantic states and 5 to 10 in. in New England and northern New York, most of which contributed to the flood runoff.

Floods have been caused in some cases by storms of the "cloudburst" type—usually of local extent, accompanied by thunderstorms. The floods in New York State in July, 1935, were notably of this type.

Floods Due to Reservoir Failures. Some of the worst floods in respect to loss of life have been those due to the failure of reservoir dams. That at Johnstown, Pa., in 1889, with some 2,200 deaths, heads the list (see page 136). In 1874, the failure of a dam upon Mill River near Williamsburg, Mass., resulted in 144 deaths (see page 137).

The flood wave due to sudden failure of a reservoir dam is likely to be tremendous in height and effects. A small reservoir dam failed in 1932 at the Balsams near Dixville Notch, N.H., where some 20 million cu ft of water was released in about 10 min, resulting in a flood wave of 30,000 cfs or more upon a small stream of about 2 sq miles drainage area. This literally cleaned out the valley for several miles downstream. Fortunately warning was given, so that no lives were lost. From this example on a small stream, it will be seen that the failure of a large reservoir, if located upon a populated river, would result in a major disaster. Proper design and construction of reservoir dams are therefore of great importance. Most states now provide laws requiring adequate engineering supervision of such structures.

Observation of a Reservoir Failure. An interesting opportunity to observe the failure of a reservoir and the progress of the flood wave downstream occurred in India in 1894.[1] Early in September, 1893, the River Bireh Ganga was completely blocked by a landslide near Gohna in Garhwal. A great mass of earth and rock filled the valley of the stream to a depth of 800 ft above the river bed and extending for a length of 2 miles. The stream flow from 90 sq miles of drainage area was completely held back for nearly a year. The river is a mountain stream with a slope of about 250 ft to the mile.

The Indian Corps of Engineers took charge, surveyed the lake that was formed, and found its capacity when ready to overflow would be about 16.5 billion cu ft. The probable date of breach was first set at May 15, 1894; but, after more data were available, it was changed to Aug. 15. Actually it occurred at 11:30 P.M. Aug. 25, 1894, although water began to escape freely by percolation early in August.

At 4 A.M. of Aug. 26, the water level had fallen 390 ft, and more than 10 billion cu ft of outflow had occurred. This is an average flow for 4.5 hr of about 620,000 cfs (about three times the flow of the Niagara River),

[1] *Jour. of the Society of Arts*, London, March, 1896, p. 431; also Frizell, "Water Power Engineering," 1st ed., pp. 222–223.

or about 7,000 csm, and is the equivalent of 48 in. of depth over the 90 sq miles of drainage area. Following are data of stream rise below the reservoir and elapsed times:

Distance from reservoir, miles	Max. rise above normal river level, ft	Elapsed time to	
		Beginning of rise	Max. rise
Just below	260		
13	160	......	25^m
20	......	47^m	
30	130	$1^h\ 20^m$	$2^h\ 15^m$
51	140	$2^h\ 30^m$	$4^h\ 15^m$
72	42	$3^h\ 45^m$	$5^h\ 10^m$
150	11	$9^h\ 15^m$	12^n

Arrangements were so well made and warnings were so efficiently given that no lives were lost in this extraordinary flood.

CHAPTER 2

METEOROLOGY OF STORMS—PRECIPITATION IN THE UNITED STATES

Energy of Wind Circulation[1]

The amount and distribution of rainfall over the surface of the earth are controlled almost entirely by the circulation of the earth's atmosphere, or winds. The source of energy maintaining this circulation is the radiant energy received from the sun, which strikes that portion of the earth turned toward the sun in a nearly constant stream. Compensation by radiation into space is continual and practically equalizes the amounts of energy received and lost. The amount of insulation or energy of the sun's rays depends upon their obliquity and is also continually changing owing to the daily rotation of the earth and its annual revolution around the sun, but the total during a year is much greater in equatorial regions than near the poles. The amount of energy lost by radiation, although variable in different locations, is more nearly constant than the total amount received. This is made possible by the transfer of energy from equatorial to polar regions by winds, assisted by ocean currents. Thus winds act as an agency that automatically tends to even up over the entire earth the supply of radiant energy.

Wind Circulation in the United States

The immediate cause of wind circulation is the variation in air temperature. Thus, considering two points some distance apart where the air temperatures average 60 and 80°, respectively, with barometric pressure 0.1 in. greater at the colder point. At the colder point, the pressure would decrease upward at an initial rate of about 0.1 in. for each 100 ft of vertical rise, while at the warmer point this decrease would be initially about 4 per cent less, owing to the lesser weight of air at 80° as compared with 60°. Although the two assumed temperatures represent approximate surface conditions, and temperatures would in general lessen some 3 or 4° per 1,000 ft of altitude, the assumed differential of 20° may continue for some distance vertically. This assumed situation would tend to cause a surface wind from the colder to the warmer region and at high altitudes a

[1] Storm Rainfall of the Eastern U.S., *Miami Conservancy District Tech. Rept.*, 2d ed., Part V, 1936, Chap. III.

wind from above the warmer region toward the colder region. Such conditions exist over a large part of the earth's surface for much of the time.

The high temperatures in the equatorial region lead to an unequal pressure distribution, with the result that there is, around the earth, a fairly permanent belt of high pressure near the surface in latitude 30 to 35°N, accompanied by corresponding low pressure at great altitudes above the earth. The resulting surface winds blowing constantly toward the equator over the oceans are called the "trade winds." They reach as a maximum an altitude of about $2\frac{1}{2}$ miles. The belt of high surface pressure is affected by the seasons as the sun moves north and south of the equator and by the relations of continents and oceans, since land areas are warmer than water areas in summer and correspondingly colder during the winter.

The region of the trade winds barely touches the most southern part of the United States, but our winds are affected by their influence. Because of the earth's rotation, any wind in the Northern Hemisphere tends to be deflected toward the right. Hence the trade winds, in blowing toward the equator, turn toward the southwest. Similarly, the air currents at high altitudes moving north from the equator blow toward the northeast; and, since they are probably much less retarded by friction because of their separation from the earth's surface, their easterly component of velocity persists sufficiently to give a decided drift toward the east to all the atmosphere throughout the range of latitude occupied by the United States. The United States is said to lie in the region of the prevailing westerlies, and this constant steady drift of our atmosphere toward the east is the basis of all our weather predictions.

Throughout the region of the prevailing westerlies, the air temperatures, and their resulting winds, are much affected by the presence of the water vapor and clouds in the air and by the vertical movements in the air masses. The mechanism controlling changes in these elements has not yet been completely determined, and for this reason long-range weather forecasts are still in only a partly developed state.

The general circulation of the atmosphere between tropical and extratropical regions by means of high-altitude currents in one direction and low-altitude currents in the reverse direction might be spoken of as a continuous circulation in vertical planes. In the latitudes of the United States, this simple circulation, during much of the time, becomes converted into what might be called, for contrast, a "circulation in horizontal planes." Over a large area, the air will be moving toward the north, at a particular instant; while, over a different but correspondingly large area at the same time, the air will be moving toward the south. The whole system has a continual drift toward the east in correspondence with the prevailing westerlies of these latitudes, with the result that, at a given geographical station, the winds are intermittently southerly and northerly.

Neither the origin of these changes nor the mechanism that controls their movements is well enough worked out yet so that accurate predictions of their movements can be made for any extended advance period.

Storms

A storm may be defined as a movement of air accompanied by precipitation—the latter, however, commonly being very variable in distribution. Precipitation is dependent fundamentally upon the evaporation of water into the air, when as water vapor it may be transported long distances by winds. To cause precipitation, condensation must be effected by cooling. The cooling may be caused by contact with cold surfaces or with cold layers of air, by adiabatic expansion while rising to greater altitudes, by radiation, and by mixing with colder air. The oceanic areas of the Torrid Zone are the most extensive source for evaporation, although it should be remembered that, even on land surfaces, a large fraction of the annual rainfall is reevaporated into the air. The water precipitated as rain therefore comes from these two distinct sources, each of which may preponderate according to the nature of the storm. Thus thunderstorms derive their supply mainly from land-surface evaporation, while cyclonic storms, which are caused primarily by warm moisture-bearing winds blowing from the tropics, are fed largely by evaporation from the ocean.

Yearly mean rainfall in the United States (Fig. 2·7, page 20) varies greatly, owing chiefly to the factors of latitude, proximity to oceans, and altitude. It tends to diminish as latitude increases, varying from about 60 in. near the Gulf Coast to about 30 in. along the Canadian line.

Rains caused by moisture-laden air from the Gulf of Mexico and the Atlantic Ocean exert a marked influence in swelling the annual precipitation in the southeastern parts of the United States. Conversely, as the distance from these bodies of water increases, this influence becomes less. This is particularly noticeable in the Great Plains region immediately west of the Mississipi River, where the lack of moisture from the Gulf region causes the annual rainfall to diminish rapidly in a westward direction. The isohyetals here run nearly north and south with a marked easterly inclination in the northerly latitudes caused by the influence of these latitudes, which tends to decrease rainfall.

Increase in altitude is often accompanied by increased precipitation, but this is usually local in character rather than country-wide. Mountain ranges tend to force air currents to higher altitudes and thus induce precipitation upon the sides exposed to prevailing winds. On the leeward sides, even at high elevations, precipitation may materially decrease.

Storms may be classed as (1) cyclonic, (2) hurricanes, and (3) thunderstorms.

Cyclonic Storms. These are often called "extratropical cyclones," to distinguish them from the tropical cyclones or West Indian hurricanes. They are the disturbances of varying intensity which pass over the United States with more or less regularity in a general easterly direction and are usually accompanied by precipitation over large areas. The barometric pressure, temperature, and other conditions accompanying the movement of such storms, or LOWS, as well as succeeding HIGHS, or conditions of fair weather, are shown upon the daily maps and forecasts of the U.S. Weather Bureau (Fig. 2·5). The LOWS and HIGHS are usually from 500 to 1,000 miles apart and pass over the country in a more or less regular succession from west to east at an average rate of about 20 mph in summer and 40 mph in winter. LOWS are generally more definite and distinct in form than HIGHS.

Hurricanes. The hurricanes, or tropical cyclones, that reach the United States are formed in a region of calm or very light variable winds from 8 to 12°N latitude and move northeasterly to about latitude 30°, thence northeasterly (Fig. 2·6). When the hurricane reaches the North American continent, wind velocities are reduced, and its characteristics are rapidly changed to those of the extratropical cyclone. About 10 such hurricanes touch the United States each year, mostly in the late summer, and on an average less than one is violent or destructive.

Thunderstorms. Thunderstorms are usually local in extent, with a cloud area rarely more than 200 miles long by 50 miles wide, and seldom causing precipitation over a length of more than 300 miles. They occur most frequently in the hottest season of the year and warmest part of the day and may be caused by (1) convection; (2) overrunning or underrunning of cool dry and warm moist air currents; or (3) contact of warm moist air with mountain slopes. They usually last only a few hours.

Precipitation in the United States

Major Influences. Several major influences exercise important effects upon the amount and distribution of precipitation in the United States. These are as follows:

Latitude. The range of latitude in continental United States is from about 25 to 49°, a distance of approximately 1,600 miles. Such a variation in latitude is bound to produce climatic differences greatly affecting both precipitation and runoff.

Land and Water. The controlling water areas in the United States are principally the Pacific Ocean, the Gulf of Mexico, the Atlantic Ocean, and to some extent the Great Lakes. The influence of these great sources of water vapor upon precipitation is materially affected by the configuration and extent of the land areas, by the direction of the prevailing winds, and by the passage of cyclonic storms.

Prevailing Winds and Mountain Barriers. Most of continental United States is within the zone of prevailing westerly winds. This general direction of wind movement is particularly effective during the winter

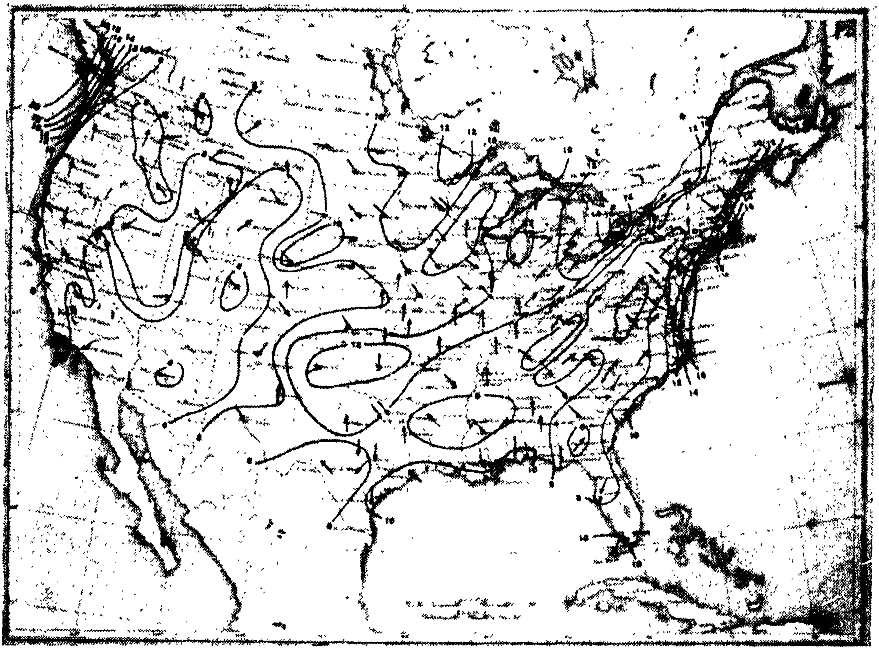


Fig. 2·1 Prevailing winds in the United States during January.

months, as shown in Fig. 2·1. It accounts for relatively high precipitation on the windward side of the principal mountain barriers, clearly indicated on the map of average annual precipitation. The great interior basins comprising the Colorado, Rio Grande, and Mississippi systems are affected, especially during the summer months, because the interior land areas comprising them are remote from large bodies of water.

Having an essentially continental climate, the land area of these basins tends to become colder than the oceans and Gulf in winter and warmer in summer. Hence air pressure over the continent's interior is, on the whole, higher in winter and lower in summer than over the water masses. Because winds incline outward from a region of higher pressure and inward toward a region of lower pressure, there is a general tendency (interrupted by cyclonic disturbances to be described later) for winds during winter to move gradually out of the high-pressure area over the western part of the interior toward the south, east, and southeast, displaced by cold and relatively dry air from Canada. This is shown on Fig. 2·1. As the winds, in general, except for the Ohio Basin, flow southward, they become heated, and the vapor capacity per unit volume is increased. Hence we find, everywhere in the interior of the United States, the least precipitation

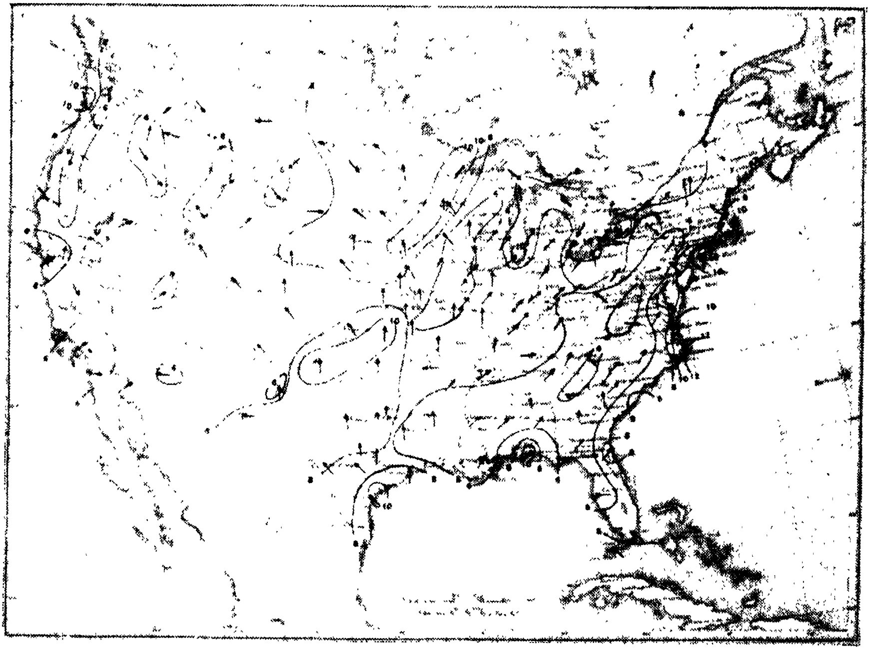


FIG. 2·2 Prevailing winds in the United States during July.

during the winter, except for relatively small areas in the Ohio and Tennessee valleys.

Conversely, during the summer, pressure over the interior of the continent is lower than over the oceans, and there is a general tendency for winds to flow inward. Because winds from the Atlantic and Pacific are cut off by mountain ranges, virtually the only gateway to the interior is from the Gulf. Hence warm winds that have passed over wide expanses of warm water enter and pass in a general northward direction up the several river basins, as shown in Fig. 2·2. As these winds flow northward, they become cooled by contact with land areas, by contact with colder air to the north, and by being forced to rise over mountain slopes or over masses of denser, cooler air, by which the temperature is reduced through expansion.

Moisture brought from the Pacific Ocean by the westerlies is precipitated as these winds rise over the Coast Ranges and the Rocky Mountains on their way eastward. Because of these mountain barriers, therefore, little or no moisture from the Pacific ever reaches the Great Plains. The Appalachians, being much lower, have less effect in preventing moisture-laden easterly winds from entering the interior. It is most fortunate for the United States that North America has no transverse mountain range

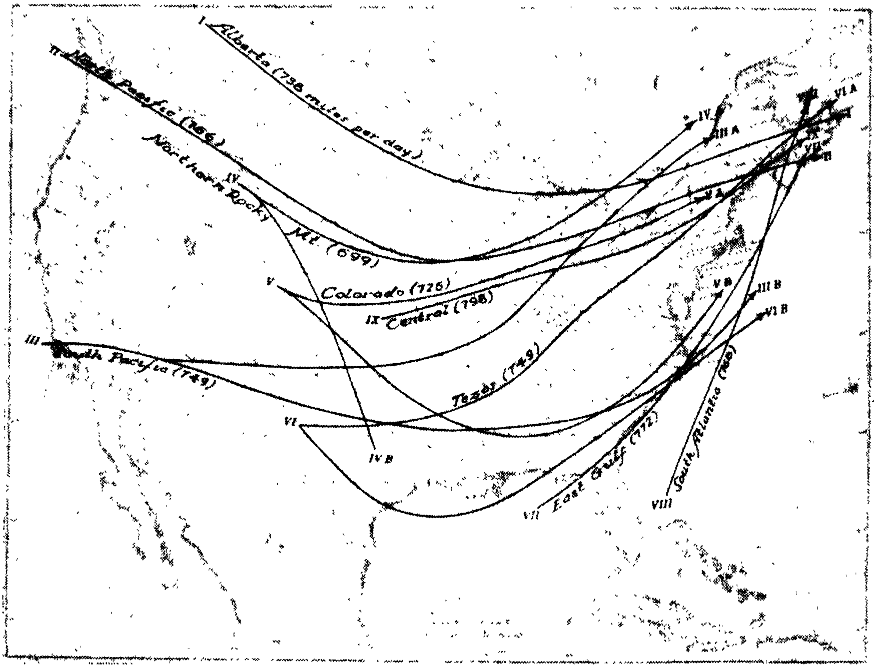


FIG. 2·3. January storm partly in the United States.

comparable with the Himalayas of Asia. Such a range would divorce the Middle West from the mild, damp Gulf of Mexico winds and cause arid or semiarid conditions in all of our North-Central states.

Between the Sierra Nevada-Cascade Mountains and the Rocky Mountains is a vast semiarid and arid region formerly known as the "Great American Desert," now the "Great Basin." Moisture-bearing winds from the west and from the east are blocked by mountains; the south opens on the desert of northern Mexico. Local convectional storms, the westerlies, and occasional winds from the Gulf of Mexico bring a little rain to this area, but agriculture is possible only where water from the mountains is available for irrigation.

Altitude. In addition to their barrier effect, mountains and highlands in themselves constitute distinctive climatic provinces. The drop in temperature previously mentioned, of about 3°F for every 1,000 ft in elevation, results in upland temperatures often many degrees below those of adjacent lowland areas. This cooling effect consequent upon expansion of air as it rises chills moisture-bearing winds, causing them to precipitate appreciable quantities of rain and snow. Both the Rocky Mountains and

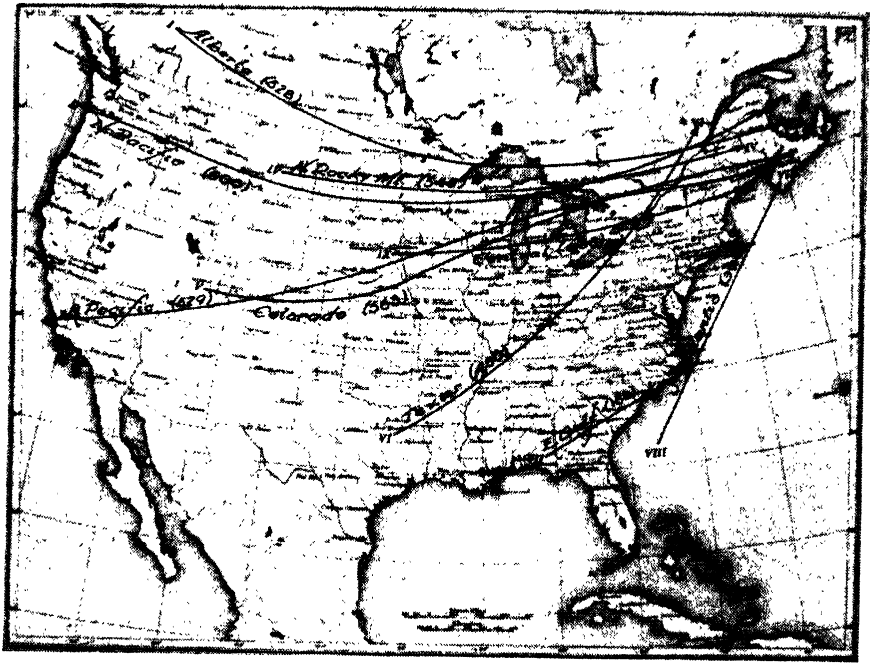
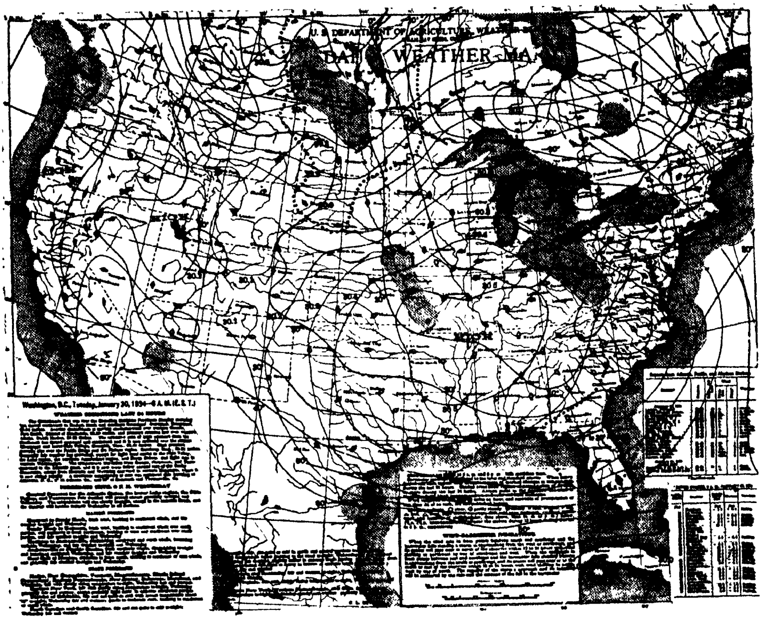


FIG. 2·4. July storm partly in the United States.

the Appalachian highlands are therefore cooler and more humid than near-by parts of the interior plains. Precipitation is also higher on the eastern slopes of the Appalachians than nearer the Atlantic coast.

Cyclonic Storms. Cyclonic storms are perhaps the most significant of the modifying influences that affect the precipitation of the United States. These cyclones, not to be confused with destructive tornadoes, are roughly circular low-pressure areas that drift across North America from west to east carried by the prevailing westerlies. Cyclones, or "LOWS," average nearly 1,000 miles in diameter and travel at a rate of some 600 miles a day. Normally each cyclone is followed by an anticyclone, or high-pressure area. This irregular procession of cyclones and anticyclones usually enters from the North Pacific or Gulf regions, as shown on Figs. 2·3 and 2·4, crosses



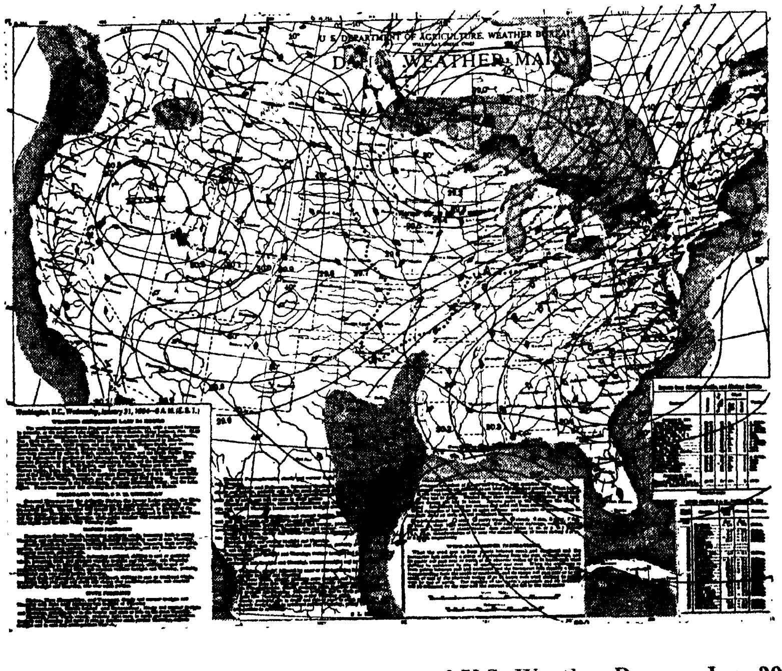
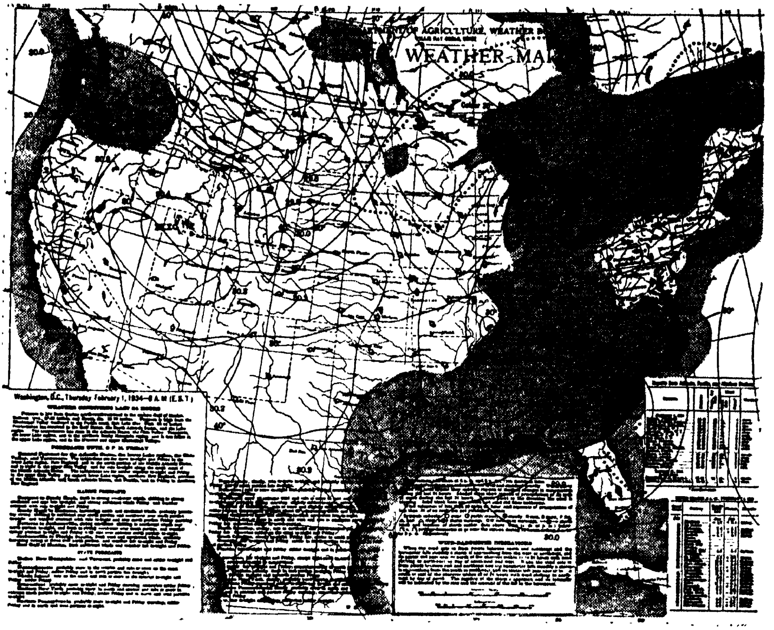
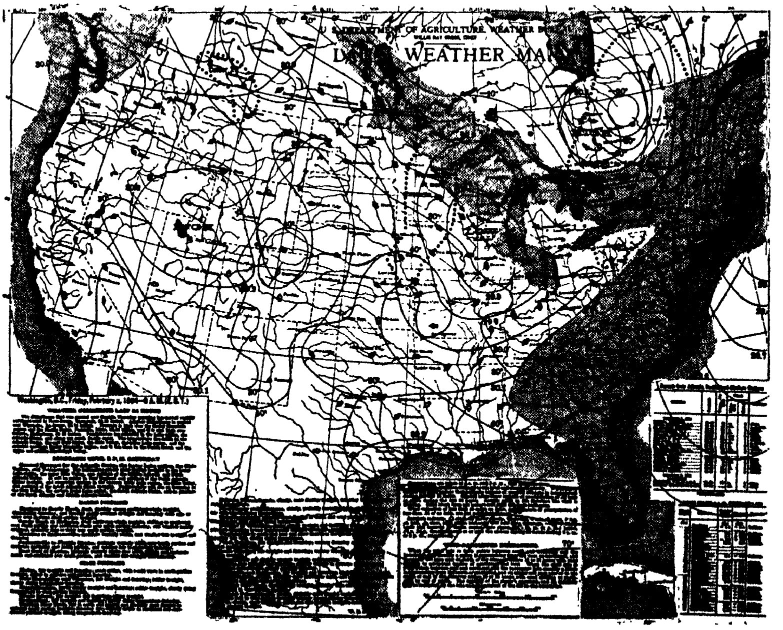


Fig. 2·5. Daily weather maps of U.S. Weather Bureau, Jan. 30



c



d

to Feb. 2, 1934. *a*, Jan. 30; *b*, Jan. 31; *c*, Feb. 1; *d*, Feb. 2.

the Mississippi Valley, and, following the route of least resistance, usually leaves by way of the Great Lakes and the St. Lawrence lowland. Some of these storms, especially in winter, pass across central New York into northern and central New England.

This phenomenon is further illustrated by daily weather maps of the U.S. Weather Bureau in Fig. 2·5, for the period of four successive days, Jan. 30 to Feb. 2, inclusive, 1934. The following more or less typical features are shown:

1. January 30. Entire country covered by high pressure with generally fair and cool weather.
2. January 31. Low areas entering from Canadian northwest and Gulf, accompanied by rain and rising temperature.
3. February 1. Both LOWS have moved eastward along typical paths, with increased intensity, and moist, relatively warm winds from the water areas are flowing into the LOWS, producing heavy precipitation and rising temperature.
4. February 2. The centers of the two LOWS have united off the Maine coast, with a "rain shadow" to the rear. Areas of high pressure, or anticyclones, with clear and cool weather cover the remainder of the country.

Abrupt changes in wind direction, in temperature, and in precipitation characterize the passage of a well-defined cyclone. Except for their general eastward movement, cyclones obliterate all evidence of the prevailing westerlies. Winds locally blow toward the center of a low-pressure area in a counterclockwise direction. This means that warm damp southern air flows into the southern and eastern parts of the cyclone, while cold and usually drier northern air moves into the northern and western sections.

Practically all the winter precipitation and much of the summer rainfall in the United States are caused by cyclonic storms. Most of the rapid changes in weather so characteristic of this area are also directly accounted for by the passage of cyclones and anticyclones.

The tropical hurricanes, which originate in the Gulf of Mexico or the Caribbean, are predominant as heavy rain producers in the late summer and fall. They move westward and then to the northeast, often causing excessive precipitation and floods along the Gulf and Atlantic coast and sometimes affecting also the headwaters of the Ohio and Tennessee basins.

Average Yearly Precipitation

In Fig. 2·7 is a map showing the average yearly precipitation over the United States. These are average figures. In some years, they will be considerably greater and, in others, considerably less. Moreover, the relative distribution shown on the map will rarely occur in any one year. The map gives a picture of the annual average precipitation that may be expected in a long term of years. It also accurately delineates the relative wetness or dryness of various parts of the United States.

North Pacific Drainage. Were it not for mountain ranges, the prevailing westerly winds from the Pacific would produce a fairly high and

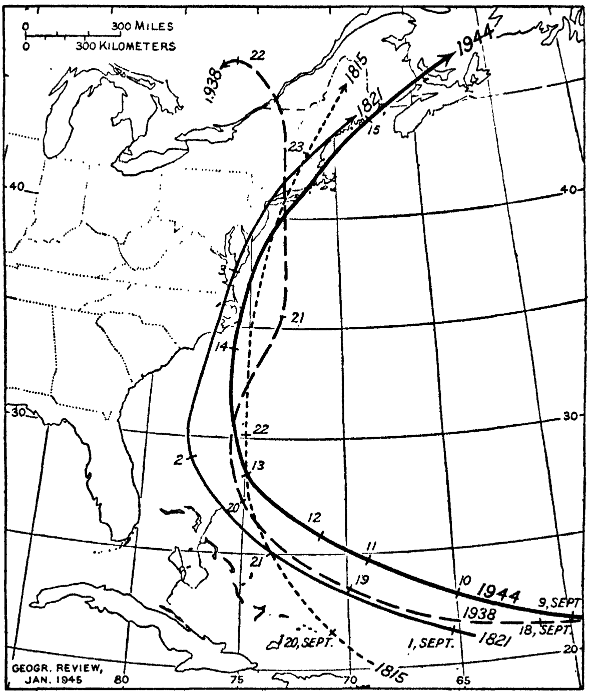
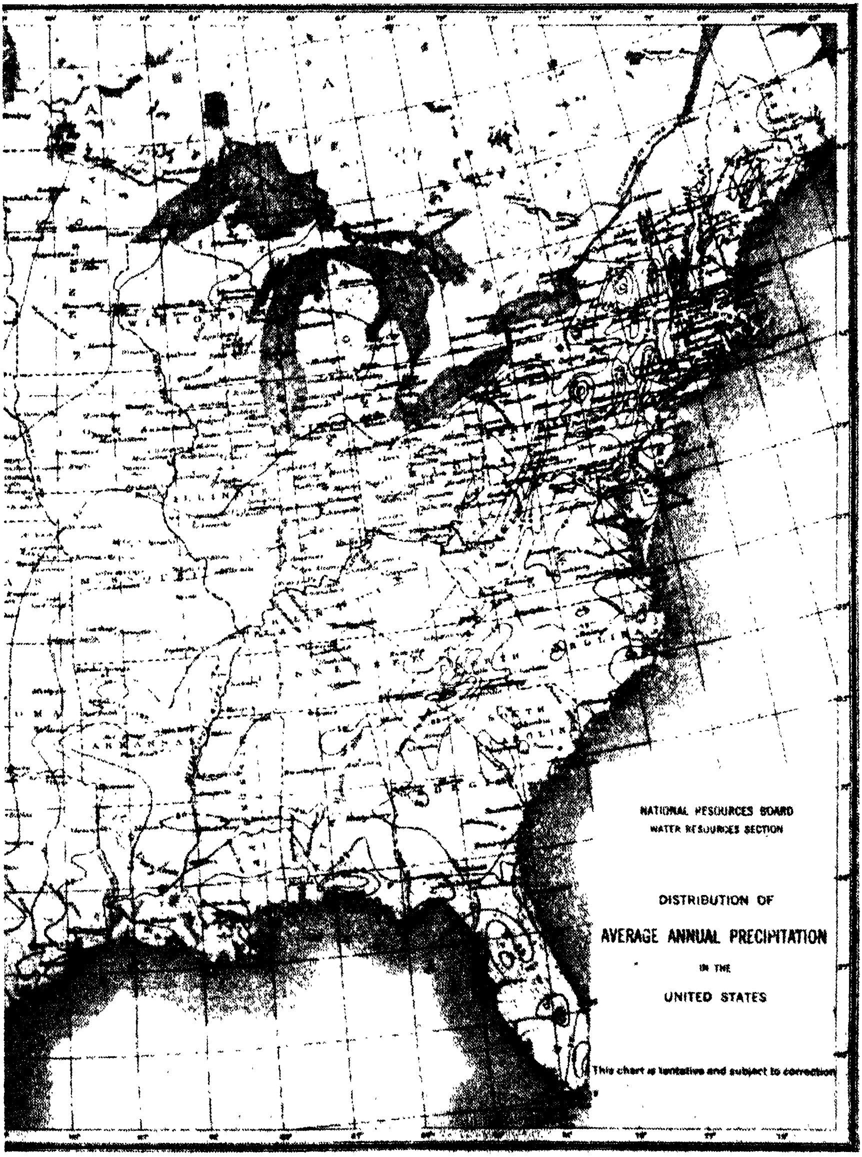


FIG. 2·6. Paths of West Indian hurricanes.

uniform precipitation, decreasing with distance from the coast. The Cascade Range, however, introduces a barrier producing extreme precipitation conditions, and on the westerly slopes of these mountains occurs the highest annual rainfall in the United States, probably reaching as much as 250 in. at high altitudes. Average yearly values of 75 to 125 in. are fairly general west of the Cascades, with snow depth often reaching 50 ft. yearly at the higher elevations.

Passing eastward from the crest of this range, the precipitation rapidly decreases to less than 10 in. on the lower portions of eastern Washington and Oregon and southern Idaho. As the Rockies of the Continental Divide are reached, there is a progressive increase in precipitation because

FIG. 2·7. Average yearly precipi



tation in the United States.

of altitude, but the winds have then lost so much of their moisture tha annual precipitation is only from 20 to 50 in. on the average over most o this territory. Over much of the Columbia Basin, the precipitation is from 10 to 20 in., the high average flow of the Columbia River being du to heavy precipitation upon headwater streams in the mountains.

South Pacific Drainage. Along the northern California coast, wher average annual precipitation exceeds 40 in., the range is from 10 to 100 in. In the San Francisco Bay region, an average of from 20 to 30 in. is sufficient for agriculture. Farther south, the 10 to 20 in. of annual rainfall, combined with high temperatures, creates semiarid conditions requiring irrigation for most agricultural purposes. This is also the case in much of the Sacramento and San Joaquin basins.

Great Basin. With no outlet to the oceans, the topographic conditions of this region operate to produce an extremely low rainfall, varying on the average from 5 to 10 in. annually. Much of this area, including the Great Salt Lake Basin, is arid and nonagricultural in character and consists of desert waste and barren hills. Local sources of water are heavily drawn upon for irrigation needs, and localities of any size, such as Salt Lake City, are frequently concerned with securing additional supplies from subsurface sources or even from more adequately watered areas outside of the Basin.

Colorado River Basin. Over most of this basin, average annual precipitation ranges from 5 to 20 in., varying in general with elevation. Local areas at high elevations, particularly in Wyoming and Colorado, have as much as 20 to 30 in. in the form of rain and snow. Precipitation over the basin as a whole will hardly exceed 10 in., and conservation of water by storage is therefore essential for the irrigation requirements of agriculture.

Western Gulf Drainage. Average annual rainfall in this region varies from 10 to 15 in. over most of the Rio Grande Basin, increasing progressively and fairly uniformly eastward to as much as 55 in. in portions of Louisiana. High rainfall in the southeastern part of this drainage district is caused by proximity to the Gulf and the climatic influences consequent thereon.

Mississippi River Basin. An inspection of the map of average annual precipitation (Fig. 2·7) will show the following salient facts:

1. The lines of equal annual precipitation tend to run in a general north-and-south direction. This tendency is most pronounced west of the Mississippi. East of the river, they tend to run northeast-southwest.

2. The annual rainfall is smallest on the west of the Mississippi Valley and increases gradually to a maximum on the east.

3. The only part of the Mississippi Valley having an average annual rainfall less than 10 in. is a small area centering in Big Horn County on the headwaters of the Big Horn River in the central western portion of

Wyoming. This region is in a deep valley (elevation 3,800 to 4,800 ft) surrounded by mountain ranges of 7,000 ft. The average annual rainfall is about 6 in. The minimum annual rainfall of record (1.20 in.) occurred at Hyattville in 1900. At this station, there are generally not more than 28 days per year when precipitation occurs, and for the area as a whole not more than 40 days per year having precipitation.

4. The Great Plains region has for the greater part an average annual precipitation between 10 and 20 in. The northwestern portion has between 10 and 15 in.

5. Farther east, there is a relatively uniform band representing average annual precipitation between 20 and 30 in. extending nearly north and south. If this amount of rainfall could be depended upon each year, crop production would be secure. Unfortunately, there are large variations from the average, often amounting to 50 per cent or more, in the rainfall for any one year.

6. Throughout the central lowland region, the precipitation varies from 30 to 40 in.

7. As the Appalachian Plateau is reached, we find over the Ohio and Tennessee basins average annual precipitation from 40 to 60 in., increasing to the east. This is the best watered portion of the Mississippi Valley affected by moisture-laden winds from both the Gulf of Mexico and the Atlantic Ocean. The rainfall is not only larger on a total annual basis but is more equally distributed throughout the year than in regions to the west.

8. Table 2·1 shows the distribution of annual precipitation over the principal subbasins of the Mississippi. It will be noted from the last two columns that the relative water-producing abilities of the different portions of the valley differ materially. Thus the Ohio Basin, with less than 40 per cent of the area of the Missouri, produces 86 per cent as much precipitation on an areal basis.

9. The maximum average annual precipitation (about 82 in.) occurs at Highlands, N.C., in the Blue Ridge Mountains. It is caused almost entirely by local topographic features inducing heavy rainfall from tropical hurricanes and from local thunderstorms in summer. The maximum annual rainfall of record was 111.20 in. in 1915.

Red River, Great Lakes, and St. Lawrence Drainage. The basin of the Red River of the North comprises a region of relatively low precipitation, varying from 20 to 25 in. annually. The precipitation characteristics of the area tributary to the Upper Great Lakes are affected significantly by proximity to the lakes themselves. A fairly even distribution of from 30 to 35 in. obtains over most of this region. The precipitation over areas tributary to Lakes Erie and Ontario and the St. Lawrence Basin is vari-

Table 2·1 Annual Precipitation in the Mississippi Valley

Subbasin	Drainage area, sq miles	% of entire basin	Average annual rainfall, in.	Average annual snowfall (water equivalent), in.	Total precipitation, in.	Equivalent proportion of total	
						In.	%
Missouri	529,000	42.2	18.91	3.90	22.81	9.62	30.9
Upper Mississippi	187,000	15.0	28.72	3.10	31.82	4.78	15.4
Ohio	204,000	16.4	42.10	2.60	44.70	7.29	23.4
Arkansas, White	186,000	14.9	28.00	1.30	29 30	4.36	14.0
Red, Ouachita	90,000	7.2	39.40	0 60	40.00	2.88	9.3
Lower Mississippi	54,000	4.3	49.37	0.50	49.87	2.17	7.0
Total	1,250,000	100.0	......	....	......	31.10	100.0
Average	....	......	28.31 91%	2.79 9%	31.10 100%		

able, owing to topographic factors. Climatic conditions are complex because of the influence of both cyclonic storms from the west and coast storms to the east. The average annual precipitation varies from 35 in. in the lower altitudes to 45 in. over the headwater streams of the Adirondacks.

North Atlantic Drainage. A tendency of storm paths from the west and south to converge over portions of this area produces a relatively high annual precipitation of marked uniformity throughout the year, and amounting to from 40 to 50 in. on the average. Precipitation is higher in the mountain areas, reaching a maximum of about 80 in. on Mount Washington.

South Atlantic and Eastern Gulf Drainage. The low-lying coastal plains of this region, rising to the Piedmont Plateau and Appalachian Mountains, produce topographic features profoundly affecting precipitation. The average annual precipitation is fairly uniform over the region, amounting to from 50 to 60 in. near the coast, 40 to 45 in. over the Piedmont Plateau, and reaching 50 to 60 in. in the mountains.

CHAPTER 3

STORM RAINFALL IN THE UNITED STATES

Maximum Rainfalls

In planning flood-control works, it is fundamentally necessary to determine what flood magnitude shall be used as a basis for their design. Selection of this flood should be based upon flood experiences of the past on the river under consideration, where such information is available. It is often the case, however, that extreme floods have not been included in the past records of a given river basin, so that recourse must be made to estimates of possible floods based upon higher rainfalls than those which have occurred.

Even where great floods have been experienced over a river basin, it is usually desirable to relate them to rainfall and study in this way the possibility of even greater floods. The consideration of maximum storm rainfalls is therefore an important adjunct in flood-control studies.

In the use of data of maximum rainfall, three aspects are important, *viz.*, (1) The amount of rainfall for given periods of time, as 1, 2, 3 days, etc.; (2) the area over which the rainfall occurs; and (3) the frequency of occurrence.

The most extended study of maximum storm rainfalls in the country was made by the engineers of the Miami Conservancy District, Dayton, Ohio.[1] All station records of five consecutive years or more in the United States east of 103° of longitude, the easterly boundary line of New Mexico (about 3,000 stations), were included through the period 1870–1933, with the following limitations:

Where the normal annual precipitation exceeded 20 in., they considered only storms where the total daily rainfall equaled 10 per cent or more of the normal annual, or where the total storm rainfall equaled 15 per cent of the normal annual. Where the normal annual precipitation was less than 20 in., they considered storms where there was a total rainfall of 4 in. or more.

For convenience, the United States was divided into 2-deg quadrangles bounded by the odd-degree meridians and parallels; and, as one feature

[1] Storm Rainfall of the Eastern U.S., *Miami Conservancy District Tech. Rept.*, 2d ed., Part V, Dayton, Ohio, 1934.

Table 3·1 Maximum Rainfalls in Eastern United States

Quadrangle	Number of stations	Aggregate years of record	Location	Days					
				1	2	3	4	5	6
				Maximum rainfall, in.					
3–D	75	1,508	Mass., R. I., Conn.	11.8	14.6	14.7	14.7	14.7	14.7
4–D	47	752	N. Y.	9.7	11.8	12.1	12.7	12.7	12.7
6–E	40	644	Pa., Md., Va.	8.5	9.4	9.4	9.8	10.4	10.9
6–G	23	344	N. C., Va.	10.5	13.5	14.5	14.5	17.0	17.0
8–H	33	529	S. C., Ga.	11.6	14.9	15.4	16.9	16.9	18.5
10–G	28	470	Tenn., Ky.	9.0	9.5	10.0	11.0	11.1	11.2
8–I	13	194	Ga.	10.4	13.5	14.1	14.3	16.9	17.8
8–J	24	408	Fla., Ga.	18.0	19.1	19.1	19.1	19.1	19.1
11–I	21	369	Ala., Miss.	12.6	13.8	13.9	12.8	11.1	14.9
13–H	16	283	Ark.	12.0	13.3	13.9	14.1	14.1	14.4
13–I	19	255	La.	21.4	28.6	28.8	29.9	30.3	31.6
15–G	13	195	Okla.	10.0	11.0	14.5	15.5	16.2	16.2
16–I	22	284	Tex.	11.0	13.2	15.0	15.7	15.7	15.7
18–H	8	69	Tex.	8.3	8.3	8.6	8.8	8.8	10.3
9–E	50	831	Ohio	6.5	9.5	10.4	11.1	11.4	11.7
12–E	30	545	Ill.	10.2	10.2	10.2	10.2	10.2	10.2
14–D	38	691	Iowa	14.6	14.7	15.5	15.5	15.5	15.5
12–B	15	140	Wis.	11.2	11.7	12.8	13.2	13.2	13.2
17–A	16	171	N. D.	5.2	5.2	5.2	5.5	5.5	6.4
17–D	20	276	Kans.	7.0	7.0	7.0	7.2	7.4	7.5

of the investigation, maps were prepared showing maximum 1- to 6-day rainfalls in each quadrangle.[1]

Maximum Station Rainfalls. In Table 3·1 are given, from the Miami report, values of maximum rainfalls in the Eastern United States showing their approximate location and amounts for each day up to 6 days.

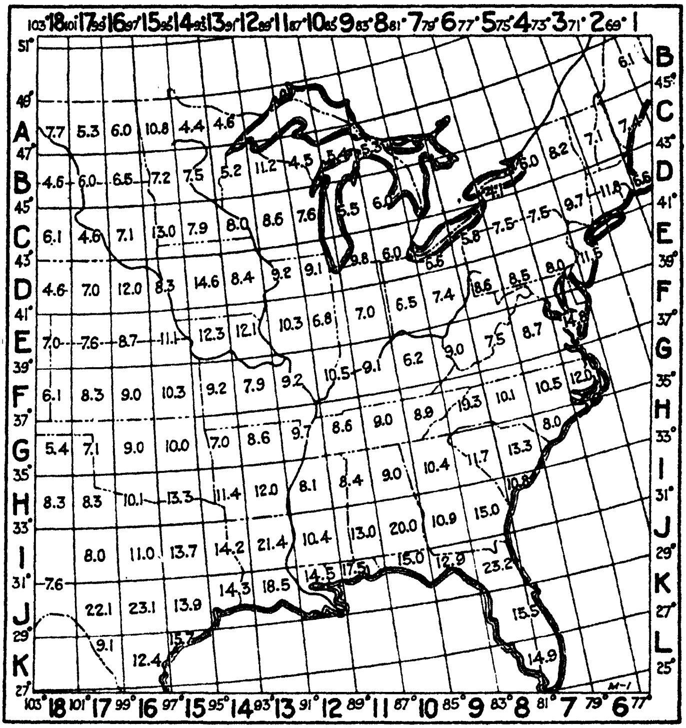


FIG. 3·1. Maximum 1-day rainfalls in Eastern United States.

Table 3·1 was prepared from Figs. 7, 37, and 38–43, inclusive, of the Miami report, selecting 20 typical quadrangles out of the 130 shown. These, together with Fig. 3·1, which shows data of rainfall for all the quadrangles for a 1-day storm, give an approximate picture of maximum rainfalls for the United States east of Colorado. It should be noted that rainfall records are generally longer and more numerous in the North Atlantic and Central states than in the south and west portions of this district.

Area Covered by Maximum Rainfalls. The data of maximum rainfalls in Table 3·1 represent the results of observations at rainfall stations or for limited areas. It is essential, therefore, to consider the areas covered

[1] *Ibid.*, pp. 103–105.

Table 3·2 Extent of Rainfall in Great Storms—United States

Storm	Center	Period, hr	Greatest rainfall, in., over area, sq miles					
			1	500	1,000	2,000	4,000	6,000
Oct. 3–4, 1869	Connecticut	48	12.4	10.4	9.7	8.9	8.1	7.8
July 12–14, 1897	Connecticut	24	8.0	7.3	6.8	6.2	5.6	5.2
		48	10.3	9.5	9.1	8.5	7.5	6.9
		72	10.3	9.6	9.3	8.6	7.7	7.0
Sept. 19–21, 1938	New England	24	4.9	4.0	3.8	3.6	3.2	3.0
		48	11.2	9.2	8.6	8.0	7.2	6.7
		72	19.7	16.0	15.2	14.2	13.1	12.4
Oct. 8–9, 1903	New Jersey	24	11.5	9.3	8.4	7.6	6.8	6.3
		48	15.0	11.9	10.9	9.9	9.0	8.4
Mar. 23–27, 1913	Ohio	24	7.0	6.7	6.5	6.2	5.8	5.7
		48	9.5	8.8	8.4	8.2	8.0	7.8
		72	10.2	9.7	9.5	9.2	9.0	8.7
		96	11.1	10.4	10.0	9.6	9.4	9.3
		120	11.2	10.9	10.8	10.5	10.1	9.9
Aug. 25–28, 1903	Iowa	24	11.2	10.6	10.0	9.1	7.8	6.8
		48	14.7	11.8	10.8	9.7	8.3	7.7
		72	15.5	12.2	11.4	10.5	9.4	8.7
		96	15.5	12.4	11.7	10.7	9.6	8.8
June 9–10, 1905	Iowa	24	12.1	10.9	10.0	8.9	7.8	6.9
July 14–16, 1916	No. Carolina	24	19.3	12.2	11.3	9.9	8.4	7.5
		48	23.2	18.4	17.1	15.5	13.7	12.5
		72	23.7	18.4	17.3	15.6	13.8	12.7
June 29–July 3, 1909	Florida	24	12.0	11.5	10.3	8.8	7.6	6.8
		48	15.5	14.2	13.0	11.8	10.8	9.9
		72	15.8	14.6	13.4	12.4	11.4	10.5
		96	17.7	15.4	14.9	14.1	12.9	12.0
		120	17.8	16.4	16.0	15.5	14.8	14.2
July 6–10, 1916	Alabama	24	11.1	10.7	10.2	9.5	8.8	8.2
		48	17.3	15.8	14.8	13.7	12.3	11.3
		72	19.7	18.7	17.8	16.5	14.7	13.1
		96	20.3	20.0	19.8	19.3	18.4	17.4
		120	22.3	20.4	19.9	19.4	18.6	17.7
Aug. 17–20, 1915	Texas	24	9.1	8.7	8.0	7.9	7.7	7.5
		48	18.6	16.2	15.0	13.6	12.3	11.7
		72	19.7	19.6	19.5	19.2	18.6	17.9

by great storms and the variation in amount of rainfall within these areas. Such information was included in the studies of the Miami Conservancy District (pages 278–281 of their 1936 report), from which Table 3·2 has been prepared (with the addition of some records available since that time).

A noteworthy storm in Table 3·2 (not available for the Miami report) is that of Sept. 17–21, 1938 (the so-called "hurricane flood"), which for

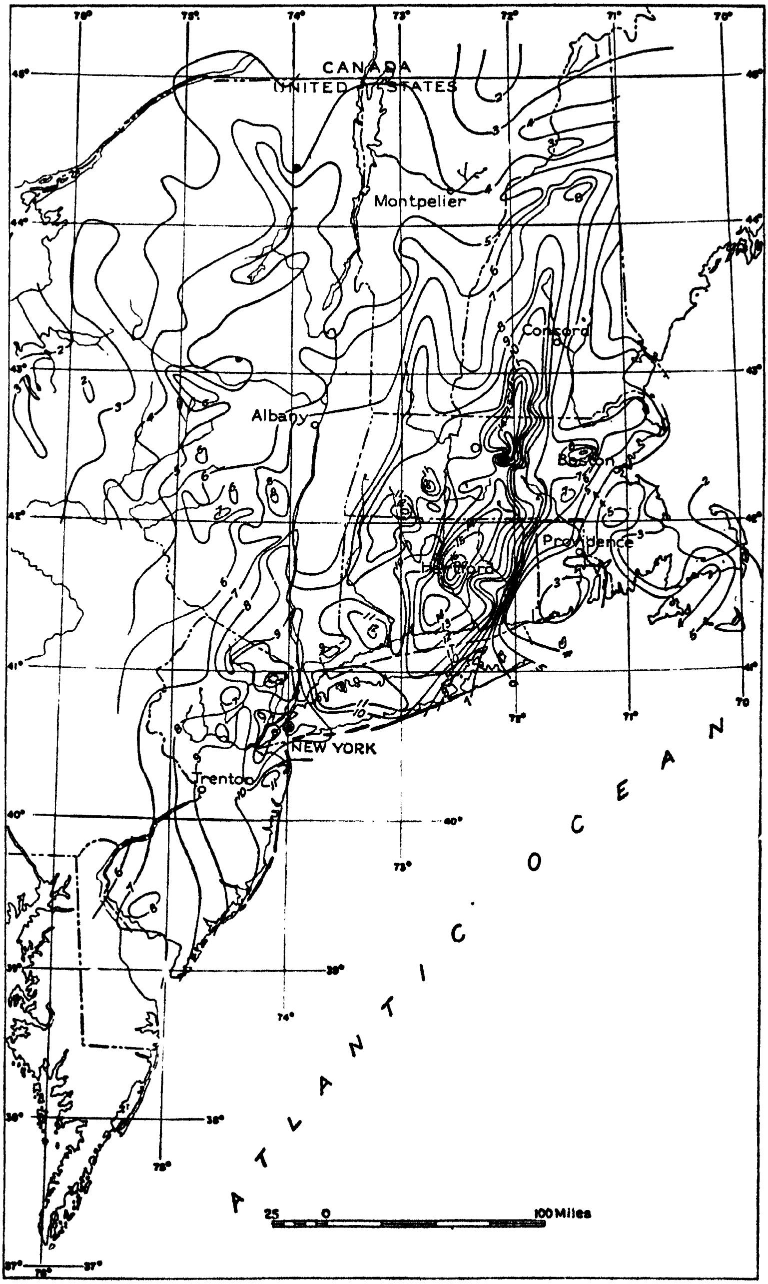


FIG. 3·2. Isohyetal map for storm of Sept. 17–21, 1938.

rainfalls of 8 in. or more exceeded all records in the Northeastern United States in respect to areas covered. These were as follows:

Rainfall, In.	Area, Sq Miles
4	63,000
6	38,000
8	21,000
10	8,300
12	2,600
14	800
16	100

The maximum of 16 in. on 100 sq miles is especially noteworthy as well as station records of 17 in. at two points. An isohyetal map of this storm is shown in Fig. 3·2 (see also page 29).

A study of the data given in Table 3·2 indicates the results as shown in Table 3·3, as the approximate maximum rainfalls that may be expected to occur sooner or later (1) in the Northern and Central states (east of the 105th meridian) and (2) in the Southern coastal states.

These values are only approximate but will serve reasonably well to define in a general way limiting values of rainfall in these districts. West of the 105th meridian, no comprehensive study has as yet been made to serve as a basis for similar data applicable to that part of the country.

Frequency of High Rainfalls. The third factor of importance in the study of maximum rainfalls is that of frequency, or the number of times

Table 3·3 Approximate Maximum Rainfalls

Time, hr	Greatest rainfall, in., over area, sq miles		
	1	500–1,000	5,000
Northern and Central States			
24	12	10	7
48	15	12	8
72	20	16	13
96 and over	20	16	13
Southern (Coastal) States			
24	19	12	9
48	23	15	12
72	23	18	14
96 and over	23	20	17

within a selected period of years that any particular amount of rainfall has occurred. Dividing the period by the number of such occurrences would give the average length of time in years during which the particular rainfall has occurred once, also (less accurately) called "frequency." It must be kept clearly in mind that no regular interval of occurrence is implied—only an average one.

In studying frequency, a term called "pluvial index" was derived as follows: In quadrangle 9-E (Ohio), there were 28 rainfall stations, with an aggregate record of 713 years (excluding stations with less than 10 years of record). It was found that 7 stations showed a rainfall of 5.40 in. or more in 1 day or on an average once in $\frac{713}{7} = 100$ years at any point in the quadrangle. This is the "pluvial index," or the 24-hr rainfall equaled or exceeded in a 100-year period. Other similar indexes were computed for 2- to 6-day periods and times of 15, 25, 50, and 100 years and plotted upon isopluvial charts (Figs. 14–47 of the Miami report).

From these charts, Table 3·4 was prepared with data for four of the quadrangles. Thus, for quadrangle 3-D (in southern New England), 4.5 in. of rainfall in 1 day occurs on an average every 20 years; 5.5 in. every 50 years; 6.5 in. every 100 years. For a 6-day storm, these figures would be 6.2, 7.6, and 8.5 in., respectively.

It is of interest to note that, assuming the values for the 100-year time

Table 3·4 Frequency of Excessive Rainfalls

Quadrangle	Average number of years between periods of excessive rainfall	Average depth, in., for periods of days			
		1	2	4	6
3–D (Massachusetts, Rhode Island,	20	4.5	5.0	5.8	6.2
and Connecticut)	50	5.5	6.6	7.3	7.6
	100	6.5	7.6	8.1	8.5
9–E (Ohio)	20	4.2	5.4	7.0	7.3
	50	5.0	6.5	8.2	8.9
	100	5.5	6.9	8.8	9.3
12–J (Louisiana and Mississippi)	20	7.8	9.0	10.3	11.5
	50	8.9	9.8	11.5	12.3
	100	9.2	10.2	12.2	14.4
15–E (Kansas and Nebraska)	20	5.4	6.0	7.1	8.1
	50	6.5	7.1	8.5	9.6
	100	7.5	8.1	9.3	10.4

interval at 1.00, the value ratios for other times are approximately alike, irrespective of the periods or number of days, and average as follows:

20 years	0.76
50 years	0.91
100 years	1.00

In other words, based upon this Miami study, 20-year rainfalls are approximately three-fourths and 50-year rainfalls nine-tenths of 100-year rainfalls.

Again keep in mind, however, that these are average figures, in respect to time of occurrence and rainfall amount. There is nothing to indicate how soon a rainfall equal to or greater than any of these tabular values may actually occur.

Yearly Distribution of Great Storms

The distribution of great storms during the year is of importance in connection with the use of multiple-purpose reservoirs and chiefly with power reservoirs for flood-control purposes, as will be further discussed.

In Table 3·5, based upon the studies of the Miami Conservancy District,[1] are given the distribution by months of the year and the number of great storms (as located by 6-in. isohyetals) that have occurred in the United States east of the 105th meridian for the period 1887–1933. The great storm of March, 1936, in the northeastern section, has also been included in this table.

Table 3·5 shows that, in New England and New York, and essentially south to Virginia, great storms have occurred either in the early spring or in the fall months. In the Southern states, great storms have occurred in nearly all the months of the year. In the North-Central states, they have been generally confined to the summer months. In the Great Plains district —from the Dakotas to Oklahoma—they have occurred in all but the earlier months of the year. Detailed information of this kind is not available for the Western states; but, since in this section of the country the greater portion of the rainfall occurs in the winter and spring months, in general, the great storms have occurred at those times of the year.

It is seen, therefore, that much diversity exists in the distribution of great storms during the year in different sections of the country.

Seasonal Occurrence of High Rainfalls. A study of seasonal distribution of great storms was made in the Miami Conservancy report (Figs. 207 and 209), which divided these into Northern and Southern storms. The Southern group shows these occurring generally in any month of the year.

[1] *Ibid.*, pp. 106–121.

Table 3·5 Distribution and Number of Great Storms in Eastern United States by Months of Year, as Limited by 6-in. Isohyetals, for the Period 1887–1936

State	J	F	M	A	M	J	J	A	S	O	N	D	Total
Maine			1						1				2
New Hampshire			1						1		1		3
Vermont											1		1
Massachusetts			1				1		2	1	1		6
Rhode Island									1				1
Connecticut							1		1				2
New York								1	1	2			4
Pennsylvania			1		2		1	2	1	2			9
New Jersey					1		2	3	2	1			9
Maryland								4	3				7
Delaware								3	1				4
Virginia			1				2	2	1	2			8
West Virginia			1				1		1				3
North Carolina			1		1		3	6	7	4			22
South Carolina	1		1		1		2	5	6	4		1	21
Georgia	4	5	4	3	2	1	2	7	11	5		3	47
Florida	2	1	3	2	4	4	3	7	15	7	1	2	51
Michigan													
Indiana			1							2			3
Ohio			1		1		1			1			4
Kentucky	5							1		1	1	1	9
Tennessee	3		4	1	1		1	2		1	1	2	16
Mississippi	5	2	7	8	8	2	2	3	6	3	3	8	57
Alabama	4	4	6	6	3	3	3	2	6	4	2	4	47
Minnesota							2		1				3
Wisconsin							2	2	1				5
Iowa						3	4	1	4				12
Illinois			2		1	2	2	2		3	1		13
Missouri	3		1	1	2	2	5	3	6	3	2	1	29
Arkansas	3		3	4	2	1	2	4	4	2	4	3	32
Louisiana	4	2	6	4	6	1	5	4	7	3	4	6	52
North Dakota													
South Dakota							1						1
Nebraska						1		2	3		1		7
Kansas					1	2	1		6	1	2		13
Oklahoma				2	2	1	2	3	2	2		2	16
Texas			1	7	7	6	5	4	7	5	4	3	49

For the Northern storms, however, limiting those considered to amounts over 10 in. of rainfall of 72 hr duration, the following is the situation:

Northern Great Storms—Distribution by Months of the Year

Month	No. of storms	% of total
January	0	0
February	0	0
March	0	0
April	0	0
May	0	0
June	3	12
July	8	31
August	4	15
September	4	15
October	4	15
November	2	8
December	1	4
Year	26	100

This lack of occurrence of great storms in the North during the winter and spring months is of importance in planning the use of multiple-use reservoirs for flood control, since needs for flood control are in harmony with the normal operation of such reservoirs for power use, as will be further discussed (see page 175).

CHAPTER 4

METHODS OF ESTIMATING FLOOD FLOWS—DISCHARGE OF RIVERS OF THE UNITED STATES

Historical

Although river stages have been observed and occasionally recorded in different parts of the world for centuries, efforts to estimate the corresponding river flows date back less than a century. Hence relatively little information of value in studying past floods is available from the existing older records, and dependence must be almost wholly upon such information as has been gathered during the past 50 years or so.

Kuichling made one of the earliest investigations of floods in a report of the New York State Barge Canal in 1901, suggesting formulas for "rare" and "frequent" floods and giving a table of floods in this country and abroad.

Weston E. Fuller made the first really comprehensive study of floods in 1913 and for the first time suggested a time or frequency factor as an essential feature of flood formulas. This was supplemented by Hazen, who suggested the application of statistical methods to flood studies. This subject has been further developed by Foster, Slade, and others.

The use of the flood hydrograph as a basis for the study of flood flows was urged by the Flood Committee of the Boston Society of Civil Engineers, who suggested the fundamental concept of a constant time base at a given river station. Sherman and Bernard developed this principle into the unit-hydrograph and distribution-graph methods of analyzing both flood flows and ordinary flows, which have resulted in important progress in methods of studying flood flows.

The U.S. Geological Survey is now systematically observing and reporting upon flood flows and obtaining information that will serve as a basis for future studies. The Section of Hydrology of the American Geophysical Union is another important agency in the analysis of flood flows, as well as in many other aspects of the hydrologic cycle.

Gauging Stations of U.S. Geological Survey[1]

The standard equipment for gauging stations includes a recording gauge, usually installed in a reinforced-concrete gauge house set upon the river

[1] Information furnished by the District Engineers of the U.S. Geological Survey.

bed at an elevation above maximum flood stage and extending by a well to below low-water elevation, connecting with river level by an intake pipe. This provides for a continuous record of water elevations.

A discharge-rating curve is developed by current meter gaugings made at different river stages. A bridge is used, if one is available for this purpose; or in many cases a cable is strung across the river, and an operating car is used for gauging purposes.

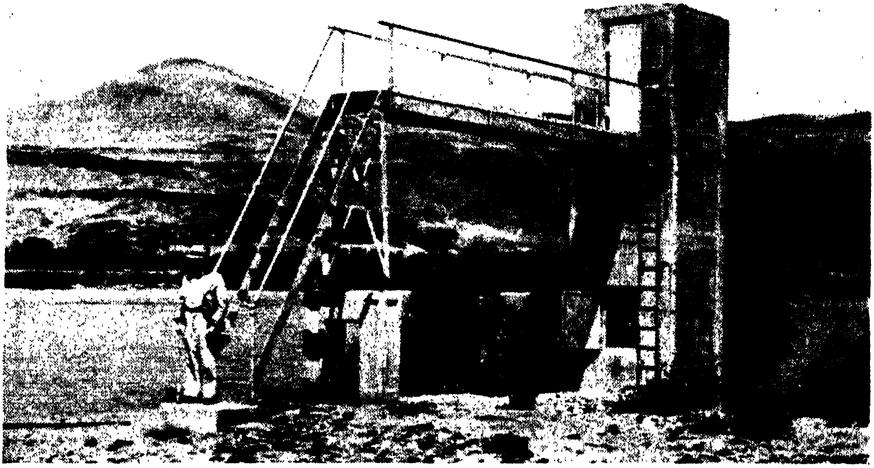


FIG. 4·1. Recording-gauge installation of the U.S. Geological Survey at Celilo Falls, near The Dalles, Oreg.

In Figs. 4·1 and 4·2 are shown a gauge and cable installation for a large river, *viz.*, the Columbia River at Celilo Falls, near The Dalles, Oreg., constructed in 1931. This station was first established in June, 1878, and for many years records were kept at a staff gauge at The Dalles. The drainage area is 237,000 sq miles.

The gauge well and shelter are of reinforced concrete, 6 ft square, outside dimensions. The well walls are 8 in. thick and the shelter 5⅝-in. The height of the well is 40 ft from the top of the footing to the floor of the shelter, 13 ft of the well being above ground. The well is connected to the river with two 2½-in. galvanized wrought-iron pipes, the lower one 3 ft and the upper one 7 ft above the bottom of the well.

The approach walk was constructed with a length greater than usual in order to obtain a good foundation and to afford better landing facilities with a boat during extreme flood conditions.

The cost of the recorder installation was $5,000.

The steel towers for the cableway are 101 ft above the concrete piers. The tops of the piers on the Oregon shore are 13 ft and those on the Washington shore 5 ft above the ground surface. The elevation, mean sea level, of the tops of all piers is 174.00, or 2 ft above the stage of the 1894 flood.

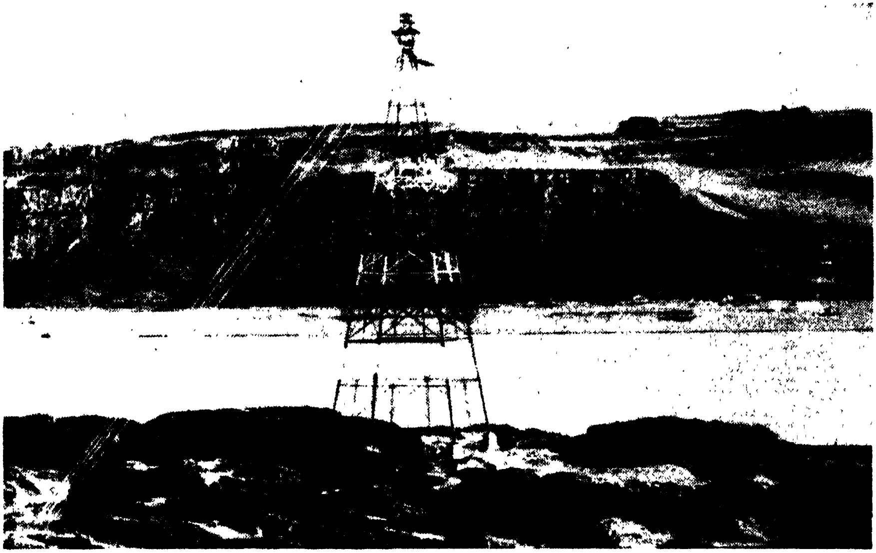


FIG. 4·2. Cableway installation of the U.S. Geological Survey, upstream from Celilo Falls.

The clear span between center of towers is 1,727.0 ft. The lower cable for the gauging car is a 1⅜-in.-diameter galvanized improved plow steel, 37-wire tramway track strand; the upper cable is ⅝-in.-diameter galvanized improved plow steel, 19-wire tramway track strand insulated from the tower by strand insulators and used as one electrical conductor to carry current across the river. The other side of the circuit is grounded to the main cable. This cable was used during erection as a messenger cable for the main cable. After erection of the main cable was completed, this cable was cut to length, field-socketed, and erected for an electrical conductor. Stockbridge vibration dampers are installed on both cables to prevent destructive resonant vibrations.

The cost of this cable installation was $15,000, including two gasoline-engine-driven electric generator plants, air-conditioned powerhouse, astronomical clock and time switch, switchboards and wiring for airways, obstruction markings, and warehouse with living quarters for engineers visiting the station for the purpose of making discharge measurements. Because of the strong winds prevalent in this area from about 10 A.M. to 8 P.M., it is necessary to begin a discharge measurement as soon as there is light enough to see.

Figure 4·3 shows the gauge installation on the Allegheny River at Franklin, Pa., which is located at Eighth St. Bridge, 1,000 ft below

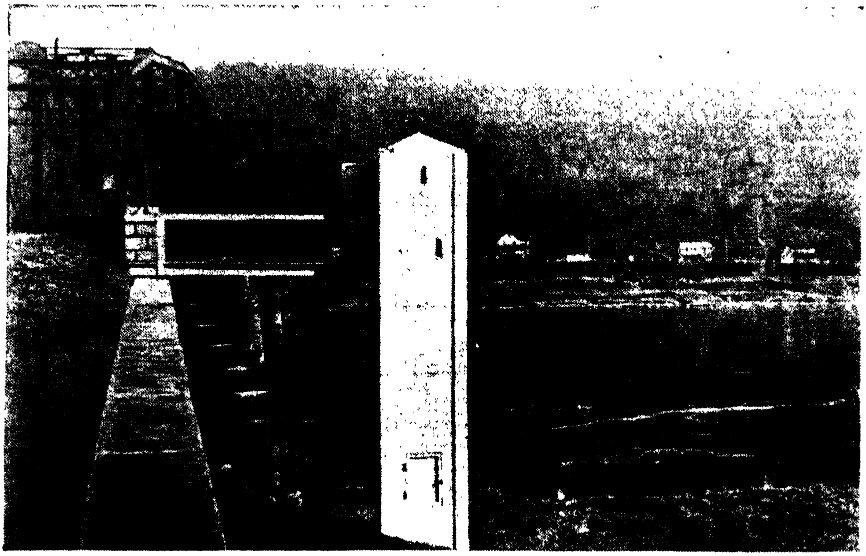


FIG. 4·3. Recording-gauge installation and Eighth St. Bridge used for gaugings at the U.S. Geological Survey, Franklin, Pa.

French Creek, and was constructed in 1932. Records began at this station in October, 1918. The drainage area is 5,982 sq miles.

The gauge house and well are of reinforced concrete from standard plans, 5 ft square, over-all height 41 ft, and cost about $2,500. Discharge measurements are made either by wading at very low stages or from the highway bridge where the waterway is about 650 ft wide.

In the New England district before the war, an average price for the ordinary reinforced-concrete gauging station on New England rivers was about $1,600 and ranged from $1,200 for a low installation like those on the Charles and Ipswich Rivers to $2,000 or more on rivers with higher houses. This includes the cost of the recording instrument, which was about $280.

The cost of cableways depended on the type of anchorage and supports as well as the width of the river at flood stage and ranged from about $100 when the cable was attached to trees, as they were in several installations, to about \$1,000 for long spans supported by steel A frames.

About 25 artificial controls are in use in this district. They are built of reinforced concrete and raise the water from 1 to 3 ft. They are particularly valuable in eliminating the effect of weeds if they are built with an upstream concrete apron whose width exceeds the length of the weeds. These aprons also serve as measuring sections for wading measurements. Concrete controls eliminate backwater effect from ice during the early part of the winter but not for the entire winter. Their cost averaged about \$25 per lin ft. Postwar costs are about 50 per cent greater.

Flood Formulas

Many formulas have been devised for the purpose of estimating flood flows. These are safely applicable in many cases only to the areas or regions where they were developed. They vary in scope from those involving only the size of the drainage area to those which attempt to include drainage-area characteristics and the factor of time or frequency. Some of the more important formulas will be considered.

Frequency Not Primarily Considered. *Formulas Involving Drainage Area Only.* These usually refer to the maximum expected flood and are of the type

$$Q = CA^n$$

where C is a coefficient, A the drainage area, and n a constant.

KUICHLING[1]

$$Q = \frac{44,000}{A + 170} + 20 \text{ (occasional floods)} \tag{4·1}$$

$$Q = \frac{127,000}{A + 370} + 7.4 \text{ (rare floods)} \tag{4·2}$$

JARVIS[2]

$$Q = C\sqrt{A} \tag{4·3}$$

where C varies between 10,000 as a maximum and 100 as a minimum. Jarvis compiled and plotted all available flood data (in 1924) with reference to this formula. This is shown in Fig. 4·4, page 46. As will be noted, limiting or 100 per cent floods are given by a value of C of 10,000.

Formulas Involving Drainage Area and Its Shape. DREDGE OR BURGE

$$Q = 1,300 \frac{A}{L^{2/3}} \quad \text{(based on Indian records)}$$

[1] *Report of New York State Engineer on Barge Canal*, 1901, p. 844.

[2] JARVIS, C. S.: Flood Flow Characteristics, *Trans. A.S.C.E.*, 1926, p. 985 *et seq.*

where L is the length of drainage area in miles. This can evidently be reduced to the form

$$Q = 1{,}300\ WL^{1/3}$$

where W is the average width of the drainage area in miles.

Formulas Involving Rainfall and Drainage-area Factors. PETTIS[1]

$$Q = CPW^{5/4} \tag{4·4}$$

where C is a coefficient relating to regional and drainage-basin characteristics, P is the storm rainfall in inches, and W is the average width of the drainage area in miles. The value of P is taken as the 100-year pluvial index of a 6-day storm from the Miami Conservancy reports.[2]

This gives for Northern United States, Ohio to Connecticut,

$$Q = 328\ PW^{5/4} \quad \text{(100-year 6-day rainfall)} \tag{4·5}$$

and

$$Q = 480\ PW^{5/4} \quad \text{(100-year 1-day rainfall)} \tag{4·6}$$

Pettis has also modified his formula[3] to read

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

where P is the probable 100-year maximum 1-day rainfall in inches.

This gives

$$Q = 310\ (PW)^{5/4} \quad \text{(in humid areas)} \tag{4·7}$$

and

$$Q = 40\ (PW)^{5/4} \quad \text{(in desert areas)} \tag{4·8}$$

The Pettis formula assumes a single storm covering the entire width of the drainage area, with high initial soil moisture content with fairly average channel slopes and stream velocities. It is generally limited in application to areas between 1,000 and 10,000 sq miles, with no storage effect and fairly uniform width. If the area is wider at its lower end, the formula gives too small results; and, conversely, if it is wider near its upper end, corrections of 10 to 13 per cent may be required for width variations.

Formulas Involving Total Runoff and Drainage Area. BOSTON SOCIETY OF CIVIL ENGINEERS METHODS. The Committee on Floods of the Boston Society of Civil Engineers made a study of flood runoff in their report upon the New England flood of November, 1927.[4]

[1] "A New Theory of River Flow," published privately, 1927.

[2] Storm Rainfall in Eastern U.S., *Miami Conservancy District Tech. Rept.*, 2d ed., Part V, Dayton, Ohio, 1934.

[3] *Eng. News-Record*, June 21, 1934, p. 804.

[4] *Jour. Boston Soc. Civil Eng.*, September, 1930, and December, 1932.

Assuming that the flood hydrograph is a triangle, they find

$$q = \frac{1{,}290\ R}{T} \tag{4·9}$$

where q = peak or crest flow, sec-ft per sq mile
R = flood runoff, in. on the drainage area
T = total flood period, hr (or the base of the flood hydrograph)

This formula was found to check well with the peak flows observed on numerous rivers. It was also found that T could usually be well established by a simple flood hydrograph and tends to remain approximately constant, irrespective of the size of flood. As previously noted, this is a fundamental concept of much significance.

Where no flood hydrographs are available, the committee suggested formulas derived from formula (4·9), as follows:

$$q = \frac{C_F R}{\sqrt{A}} \tag{4·10}$$

or

$$Q = C_F \sqrt{A}R \tag{4·11}$$

where C_F = the "flood characteristic" or coefficient of a stream—ordinarily varying between 100 and 500 but with values in mountainous regions of 500 to 1,000 and occasionally, with very flat streams, less than 100
A = drainage area, sq miles
Q = peak flow, cfs

For New England, values of R are as follows:

Occasional floods	R = 3 in.
Rare floods	R = 6 in.
Maximum flood	R not over 8 in.

Formulas Involving Rainfall Intensity and Drainage Area. RATIONAL METHOD. The so-called "rational method" used in the design of storm-water drains for comparatively small areas may be applied in principle to the determination of maximum flow of larger streams. This is expressed by the formula

$$Q = CiA \tag{4·12}$$

where C = a coefficient or ratio of runoff to rainfall
i = intensity of rainfall, in. per hr, usually taken as the maximum average rate during the concentration period T_c, hr
A = drainage area, acres

The runoff in inches is

$$R = CiT_c \quad \text{or} \quad Ci = \frac{R}{T_c}$$

Hence

$$Q = \frac{RA}{T_c}$$

For similar drainage areas, T_c will vary directly with the linear dimensions or as the square root of the area. Hence, by changing the coefficient and making A square miles instead of acres, formula (4·12) becomes the same as (4·11), *viz.*,

$$A = \frac{C_F AR}{\sqrt{A}} = C_F \sqrt{AR}$$

Frequency Considered as a Primary Factor. FULLER'S METHOD. Weston E. Fuller[1] after an elaborate study of flood data in the United States as available in 1913, suggested a method and formula that have been widely used, in which the factor of time of occurrence or probability was included. He divided conditions affecting floods into two classes:

1. Those relating to one stream tending to make all floods greater or less than on others—including hydrological and drainage-basin characteristics.
2. Those general in effect on different areas but variable in time—including rates of rainfall, special hydrological and basin conditions, etc.

The effect of the first class of conditions for any given area appears in the value of C, while the second class is made a fraction of time, or T in the formula.

The average yearly flood

$$Q_{\text{avg}} = CA^{0.8} \tag{4·13}$$

where C is a coefficient and A the drainage area in square miles, is the basis for his formula, obtained by logarithmic plots of actual flood flows, substantially all of which showed consistently common slopes ($0.8\pm$) but different values of C. He concluded that 5 or 6 years of stream-flow records would fix Q_{avg} fairly well, and hence a value for C.

To obtain the greatest daily or 24-hr flood in T years, the factor $(1 + 0.8 \log T)$ is applied to Q_{avg}. This relation was also obtained from logarithmic plots of actual flood ratios to Q_{avg} over different periods of time.

To obtain the crest flood or maximum during a portion of the day, the factor $(1 + 2A^{-0.3})$ is further applied, also based upon such actual relations as were available for this purpose. This factor is perhaps the most uncertain of any that Fuller developed, owing to rather meager data.

[1] *Trans. A.S.C.E.*, 1914, p. 564.

The complete formula is then

$$Q_{avg} = CA^{0.8} (1 + 0.8 \log T)(1 + 2A^{-0.3}) \qquad (4\cdot14)$$

Tables 4·1 and 4·2 are given to show the effect of T and the size of drainage area.

Table 4·1 Relation between Flood to Be Expected in a Series of Years and the Average Yearly Flood

$$Q = Q_{avg} (1 + 0.8 \log T)$$

Time, years (1)	Ratio of largest flood to average yearly flood (2)	Time, years (1)	Ratio of largest flood to average yearly flood (2)
1	1.00	50	2.36
5	1.56	100	2.60
10	1.80	500	3.16
25	2.12	1,000	3.40

Table 4·2 Relation between Maximum Flood and Average 24-hr Flood

$$Q_{max} = Q (1 + 2A^{-0.3})$$

Catchment area, sq miles (1)	Ratio of max. flood to average 24-hr flood (2)	Catchment area, sq miles (1)	Ratio of max. flood to average 24-hr flood (2)
0.1	5.0	500	1.31
1.0	3.0	1,000	1.25
5.0	2.23	5,000	1.15
10	2.0	10,000	1.12
50	1.62	50,000	1.08
100	1.5	100,000	1.06

The values of C for different sections of the country are given in Table 4·3. The use of the average yearly flood in his formula as a basic element is logical and gives for the drainage area in question a direct means of determining its particular flood tendencies. Since many stream-flow records of 15 years extent and more are available, this portion of the formula may now usually be well determined.

The time or probability factor $(1 + 0.8 \log T)$, increasing as it does with T and at a rate of 0.8 for each multiple of 10 increase in time, would indicate that it is only a question of time when any given flood amount will be ex-

Table 4·3 Values of C in Fuller's Flood Formula

Locality	Number of stream stations considered	Value of C		
		Maximum	Minimum	Average
New England	32	110	17.0	57
Hudson River Basin	14	132	37.0	85
Middle Atlantic states	31	140	30.0	76
South Atlantic states	49	113	25.0	66
Ohio River Basin	38	142	47.0	81
St. Lawrence River Basin	23	110	9.2	31
Hudson Bay Basin	9	57	1.3	20
Upper Mississippi Basin	16	55	7.4	22
Missouri Basin	66	49	1.8	11
Lower Mississippi Basin	8	52	1.7	17
Western Gulf of Mexico	4	31	9.5	17
Colorado River Basin	24	46	3.0	18
Great Basin	45	55	0.6	13
Southern Pacific Coast	29	193	15.8	68
Northern Pacific Coast	52	186	2.2	45
Total or average	440	98	14.0	42

ceeded. This is not fundamentally correct, because every drainage area has some limiting flood possibility imposed by its location and character.

The relation of crest or peak flood to the maximum daily flood as given by $(1 + 2/A^{0.3})$ was only approximate as determined by Fuller. A study by the author shows the results in Table 4·4.

Table 4·4 Values of x in Fuller's $(1 + 2/A^x)$

Section	No. of gauging stations	Exponent of A in $(1 + 2/A^x)$	
		Range	Mean
North Atlantic slope	7	0.30–0.44	0.35
South Atlantic slope	6	0.32–0.57	0.42
Ohio River	10	0.28–0.53	0.36
Hudson Bay and Upper Mississippi	8	0.32–0.56	0.44
Missouri River	7	0.25–0.42	0.35
Pacific slope—California	7	0.19–0.33	0.25
North Pacific slope	7	0.22–0.55	0.41
Total or average	52	0.27–0.49	0.37

It will be noted that, in general, values of x in $(1 + 2/A^x)$ range from about 0.2 to 0.6 and average nearly 0.4, whereas Fuller suggested an exponent of 0.3, indicating a tendency to overestimate peak flows as compared with maximum daily flows in his formula as given. These are averages, however, and individual stations show wide departures from these relations, particularly in the Pacific Coast region, where Fuller's formula is of doubtful applicability.

Fuller's method of estimating flood flows has proved of value and is in frequent use. The significance of T in his formula is that of a factor of safety or chance that may be taken in allowing for floods. If great damage and perhaps loss of life may be involved, a large value of T should be used. The added cost of using a large value of T is insurance against loss.

The use of flood-flow formulas as a basis for flood study as presented upon the preceding pages may be necessary where no better method is possible and for other general purposes. They should be regarded, however, as only approximations, and preference should be given to data based upon actual flood flows where they are available.

Maximum Runoff

In Fig. 4·4 is a logarithmic plot of maximum runoff rates in csm against drainage area for the Eastern United States, proposed and used by the Tennessee Valley Authority in connection with the Hiwassee project upon Hiwassee River and other adjacent projects. As will be noted, this is based upon the various great storms in this general district between 1887 and 1936, including such data as were available in the Tennessee Basin itself. Storms of importance in fixing the limiting line, aside from the Tennessee and Cumberland basins, were New York, 1935 for smaller areas, and Miami, 1913 for larger areas and March, 1936, for about 5,000 sq miles and more. The point marked + at about 700 sq miles and 200 csm, which was on White River in Vermont, in 1927, is also noteworthy.

Discharge of Rivers of United States

In Appendix A are given data of discharge of rivers of the United States compiled from recent *Water Supply Papers* of the U.S. Geological Survey and arranged according to drainage area (sections or groups) following divisions like those published, though slightly changed in order so that the numbering here is not consecutive.

This table includes rivers and stations, drainage areas and average, maximum, and minimum discharge expressed in second-feet per square mile (csm). Average discharge is, in general, where data are available, for a period of about 10 years or more; maximum discharge is that observed usually during the period of station record, but occasionally, where available, it is based upon earlier flood records; minimum discharge is for the lowest day during the period of record.

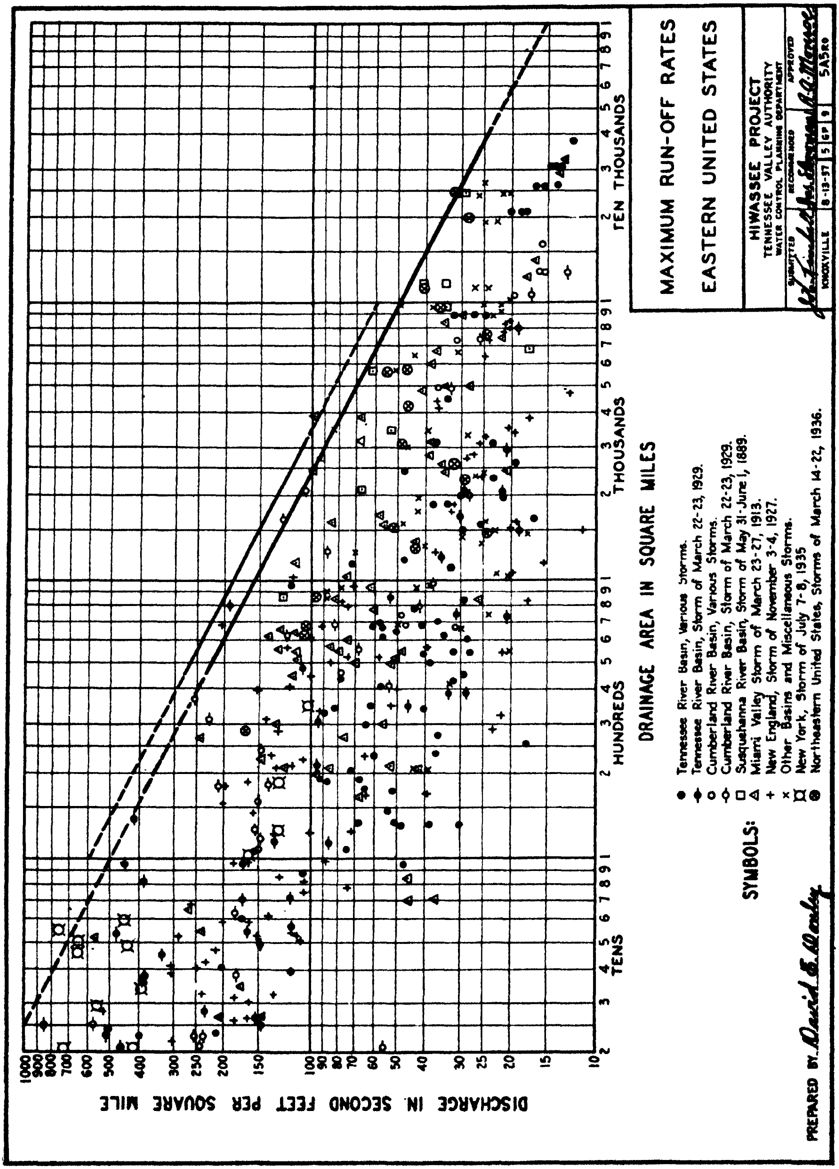


Fig. 4·4. Maximum runoff for Eastern United States. (TVA)

Note that average discharge may be transformed to inches depth of yearly runoff by multiplying csm by 12.0 1.13 = 13.56. In computing *average section discharges*, where more than one station is included upon a river, only data for the point of largest drainage area are included. Characteristics of discharge in these different subdivisions of the United States will now be discussed.

1. North Atlantic. *New England.* Dominant topography includes the White and Green Mountain ranges in New Hampshire and Vermont, which show a variation in yearly rainfall from about 80 in. at the summit of Mount Washington to 30 in. or more in southern New England. This results in greater unit runoff in the upper portions of most of these rivers. The average discharge, as given from about 60 per cent of the total area of New England, is about 1.6 csm or 22 in. yearly.

Floods are frequently due to melting snow. It must be kept in mind that unit flood runoff usually increases as drainage area decreases. Thus the flow of 29.4 csm on the Connecticut River is an outstanding flood for the 9,661 sq miles at Thompsonville; whereas, upon small tributaries of this river and elsewhere in New England, flood flows of 300 csm and more are of record (see Chap. 4, Fig. 4·4). Minimum flows are generally well sustained because of actual and artificial storage, as noted by the letter *s*.

Rivers in northern Vermont drain into Lake Champlain and are included in the St. Lawrence River drainage (Sec. 4).

Middle Atlantic. In New York, the Adirondacks and Catskill Mountains are important as affecting runoff, and show runoff-elevation gradients. The Hudson River in eastern New York is typical with a yearly average runoff of 22 in., and reaching 27 in. in its headwaters. Much of New York drains into the St. Lawrence River Basin. Farther south, the Delaware drains parts of Pennsylvania and New York, with 23 in. of yearly runoff. The Susquehanna in Pennsylvania and Maryland with 18 in. and the Potomac with 13 in. show a decreased yield in the southerly portion of this district.

Maximum flows are similar to those in New England and minimum flows somewhat lower. The average discharge is 1.28 csm, or a yearly run off of about 18 in.; for the entire North Atlantic district, including New England, runoff is about 19 in.

2. Southern Atlantic and Eastern Gulf of Mexico. This section includes the portion of Virginia south of and including the James River, North and South Carolina, Georgia, Alabama, and Mississippi. It includes an area of higher rainfall, reaching 70 in. yearly and over in the southern Appalachians and 50 to 60 in. near the Atlantic and Gulf Coast. Average unit flows vary from about 1.0 csm, or about 13 in., in and near Virginia to 1.5 csm, or about 20 in., in the south and along the Gulf.

For this section, average discharge is about 1.3 csm, or 19 in. yearly runoff. Maximum and minimum flows are similar to those farther north except in a few cases where minimum flow is affected by some storage regulation.

4. St. Lawrence River. This section includes the Great Lakes, Niagara and St. Lawrence River with portions of five central states, and upper New York and Vermont.

The Great Lakes, with a total drainage area of about 298,000 sq miles at the outlet of Lake Ontario, constitute one of the great natural storage systems of the world, with a total water area for the five lakes of about 95,000 sq. miles, or 32 per cent. The average flow at lake outlets is as follows:

Lake Erie (Niagara Falls at Buffalo)	190,000 cfs, or 0.72 csm
Lake Ontario	218,000 cfs, or 0.74 csm

These figures do not include diversion from Lake Michigan. Allowing for the unit flow of rivers in northern New York and Vermont, the flow of the St. Lawrence section averages about 0.75 csm, or a yearly runoff of about 10 in.

The regulating effect of the storage in the Great Lakes results in a maximum flow, at their outlet, of only 1.0 csm and a minimum of about 0.5 csm or a range from 1⅓ to ½ the mean flow between maximum and minimum—a remarkable degree of regulation for so large a river system, as may be seen by comparing with similar ratios in the St. Lawrence and other sections.

Rivers *tributary* to the Great Lakes in Wisconsin, Michigan, and Ohio show relatively low unit discharge—from 0.1 to 0.8 csm. In New York and Vermont, average flow is affected by altitude-rainfall gradients in the Adirondacks and Green Mountains. Flood flows in northern Vermont (in November, 1927) are high except for Otter Creek, which has unusually large valley storage above Middlebury. Minimum flows are generally low.

3. Ohio River. The Ohio is formed by the confluence of the Allegheny from the north in Pennsylvania and New York and the Monongahela from the south in West Virginia, where they meet just below Pittsburgh. At Sewickley, Pa., a little below the point of confluence, the drainage area is 19,500 sq. miles and average unit flow 1.6 csm.

At Huntington, W. Va. (55,900 sq miles), the average unit flow is 1.4 csm; at Louisville, Ky., 1.2 csm; and near the mouth, at Metropolis, Ill. (203,000 sq miles), again 1.4 csm because of the relatively high yield of the Tennessee and Cumberland Rivers (respectively, 1.75 and 1.45 csm) entering from the south. The northerly tributary streams—the Scioto, Miami, and Wabash—have lower average unit flows, from 0.7 to 0.9 csm.

Maximum unit flows are relatively high for the Ohio and its entering tributaries and result in a flow at Metropolis of nearly 9 csm, which for 203,000 sq miles of drainage area is high. Minimum flows are generally low.

5. Upper Mississippi. Average unit flow decreases going northward upon the upper Mississippi, varying between 0.5 csm at Keokuk, Iowa (119,000 sq miles), and 0.23 csm at Royalton, Minn. (11,600 sq miles). This is due to the progressive lessening of continental rainfall between the Gulf and Canada. It will be noted that the average unit discharge of the upper Mississippi is materially less than that of the Ohio. This is also generally true for maximum and minimum unit flows.

5. Hudson Bay. Some 30,000 sq miles of area in Minnesota and North Dakota drain toward Hudson Bay. The principal river—the Red River of the North—has a very low unit discharge, less than 0.01 csm, and also little flood tendency and low minimum flows.

6. Missouri. This is the largest tributary of the Mississippi, with about 530,000 sq miles, and drains a greatly diversified area, much of it with light rainfall, including Montana, the Dakotas, Iowa, Kansas, Nebraska, and Missouri, and a portion of Colorado and Wyoming. Average unit discharge is as follows:

Point	Drainage area, sq. miles	Discharge, csm		
		Avg.	Max.	Min.
Fort Benton, Mont.	24,600	0.33	5.70	0.025
Fort Peck Dam, Mont.	57,800	0.10	0.57	
Bismarck, N. D.	186,400	0.10	1.50	0.01
Omaha, Nebr.	322,800	0.082	0.62	0.0007
Hermann, Mo.	528,200	0.12	1.10	0.0080

Considering these three portions of the Mississippi drainage area together, flows are as follows:

Table 4·5 Combined Flows of Ohio, Upper Mississippi, and Missouri Rivers

River	Drainage Area, sq miles	Avg. flow		Max. flow		Min. flow	
		cfs	csm	cfs	csm	cfs	csm
Ohio	203,000	284,000	1.40	1,800,000	8.88	20,300	0.10
Upper Mississippi	119,000	61,000	0.51	315,000	2.64	5,000	0.042
Missouri	528,000	64,000	0.12	580,000	1.10	42,000	0.080
Total or mean	850,000	409,000	(0.48)	2,695,000	(3.15)	67,300	0.080

7. Lower Mississippi. The totals in Table 4·5 give the approximate drainage area and unit flows, as measured, of the three tributaries, which together make up the upper end of the lower Mississippi. They are also, with the exception of the upper Mississippi, the total tributary drainage areas. The point for gauging of the latter, however, is at Keokuk, with 119,000 sq miles of drainage area, whereas the entire upper Mississippi area is 172,000 sq miles.

Actual drainage areas and corresponding unit discharges are given in Appendix A in the lower Mississippi section. These include data for the main stem at St. Louis, Memphis, and Vicksburg, as well as the principal entering tributaries between Memphis and the Gulf, *viz.*, the St. Francis, White, Arkansas, and Red on the west and the Yazoo on the east, and include about 100,000 sq miles, making the center area of the lower Mississippi about 1,244,000 sq miles.

The *average* unit flow at Memphis, Vicksburg, and at the Gulf is 0.46 csm, which differs little from the 0.48 csm in Table 4·5.

The actual maximum unit flow at Memphis of 2.12 csm, or about 2,000,000 cfs, is, however, materially less than the sum of the maximums for the three tributaries in Table 4·5 of 3.15 csm, or 2,695,000 cfs. This is due of course to lack of synchronization of floods upon the three tributaries. Ohio River maximum floods have thus far not coincided in timing with those of the Missouri.

Hence the peak floods upon the lower Mississippi have thus far been a combination of floods upon the Ohio and on the westerly tributaries below Memphis, chiefly the White, Arkansas, and Red rivers with some 280,000 sq miles of rivers with high flood tendencies.

Minimum flow at Memphis and Vicksburg—of 0.085 csm, or, respectively, about 80,000 and 97,000 cfs—approximates the 0.080 csm, as shown in Table 4·5 for the sum of the three large tributaries, being increased a little, chiefly because of the higher minimum unit flow of White River.

8. Western Gulf of Mexico. This section includes chiefly Texas. The Rio Grande, Brazos, and Colorado are the larger streams. Average unit flows for this section are very low at 0.21 csm and minimum flows in many cases none. Maximum unit flows are in most cases low.

9. Colorado River. The river with its tributaries drains about 180,000 sq miles, mostly arid region, rising in Colorado and flowing through Utah, and between Arizona and New Mexico. Average unit flow decreases as the river flows southwesterly, varying from 1.0 csm in the headwaters to a low of about 0.1 csm at Parker Dam (178,000 sq miles). At Boulder Dam (170,000 sq miles), storage of about 3 in. depth over the drainage area practically regulates stream flow so that, below this point, maximum and minimum unit flows differ little from average flow.

10. Great Basin. This section lies in the arid region of Utah, Nevada, and southern California. The gauged area includes about 20,000 sq miles with the low average unit flow of 0.16 csm, or about 2.2 in. yearly, or a yield smaller than that of the Colorado. Maximum and minimum unit flows are very low.

13. Snake River. With a total drainage area of 109,000 sq miles, it is the principal tributary of the Columbia River, including chiefly the state of Idaho and a portion of eastern Oregon. The station at Clarkston, Wash.—103,000 sq miles—has an average flow of 0.46 csm, or 48,000 cfs: maximum 2.60 csm; minimum 0.10 csm—a materially smaller unit flow than the Columbia main stem, as further noted.

12. Columbia River. At Grand Coulee Dam, Wash.—74,100 sq miles—the Columbia has an average flow of 1.41 csm, or 104,000 cfs; a maximum of 6.65 csm and a minimum of 0.30 csm.

Combining these figures with those for the Snake River at Clarkston previously noted gives

Drainage area	177,100 sq miles
Average flow	152,500 cfs = 0.86 csm

At The Dalles—237,000 sq miles—the average flow is 0.82 csm, or 195,000 cfs. The 60,000 sq miles between Clarkston on the Snake River and The Dalles on the Columbia therefore has an average flow of 195,000 − 152,000 = 43,000 cfs or 0.72 csm, further showing the greater yield of the Columbia main stem as compared with that of the Snake River.

Nearer the headwaters, the Columbia at Birchbank, B.C., has a flow of 2.03 csm. Maximum flow upon the Columbia usually occurs in June because of melting snow at the higher elevations. The maximum observed, June 6, 1894, reached 1,170,000 cfs at The Dalles, or 4.95 csm. Minimum flows are affected by storage at Coulee Dam.

14. North Pacific. This includes rivers draining into Puget Sound, including Puyallup, Snoqualmie, and Skagit—all of which are rivers with high average flow—from 3 to 6 csm—and well-sustained minima.

11. California. The dominant river systems in California include the Sacramento from the north and the San Joaquin from the south, which together make an area of about 58,000 sq miles extending from about two-thirds the length of the state in its central point and draining through one pass to the Pacific at San Francisco Bay.

The southern part of the state includes a great coastal plain with streams of moderate size, with the highly populated centers of Los Angeles and San Diego. In the eastern portion lies the Sierra Nevada Range, with the Mohave Desert in the southeast separated from the coastal plain by lesser mountain ranges.

The records of 17 rivers, totaling 54,396 sq miles of drainage area in California, show an average flow of 0.98 csm with high maximum flows—particularly in the southern coastal plain. Some of the flood flows from smaller areas in the vicinity of Los Angeles in the flood of Mar. 2, 1908, show exceedingly high unit flows, some of which were as follows:[1]

Stream	Drainage area, sq miles	Maximum flow csm
Loop Canyon	1.06	1,717
Fall Creek	2.29	1,883
Millard Canyon	2.74	1,880
Rubio Canyon	0.88	2,273
Pasadena Glen	0.78	3,589
Devils Canyon	15.4	1,493

As noted elsewhere (page 318), the flood problem in and near Los Angeles has been unusually difficult to solve.

Average River Discharge in the United States. The data in Appendix A afford a basis for making an estimate of the average river discharge in the United States. In Table 4·6 are computations made as a basis for this estimate.

The weighted average discharge as determined in column (4) of Table 4·6 is 0.60 csm, or an average yearly runoff of 8.1 in. [column (7)]. This

Table 4·6 Average River Flow in United States

Section	Drainage area		Average discharge, csm	cfs Col. (2) × Col. (4)		Average yearly runoff, in.
	Sq miles	%		Amount	%	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
North Atlantic	85,420	3.6	1.43	130,000	9.0	19.5
South Atlantic and Gulf	122,193	5.0	1.28	156,000	10.9	17.5
St. Lawrence	303,080	12.5	0.75	22,800	16.0	10.1
Mississippi	1,244,000	51.0	0.46	575,000	40.0	6.2
Western Gulf	171,592	7.0	0.21	35,800	2.5	2.8
Colorado	180,000	7.4	0.10	18,000	1.3	1.3
Great Basin	20,502	0.9	0.16	3,260	0.2	2.2
Columbia	246,950	10.3	0.95	234,000	16.4	12.8
California	54,386	2.3	0.98	53,000	3.7	13.2
Total or weighted mean	2,428,123	100.0	(0.60)	1,433,060	100.0	(8.1)

[1] *Report upon Flood of Mar. 2, 1938*, Los Angeles County Flood Control District.

is an approximation, since the area of 2,428,123 sq miles for which discharge measurements have been made is about 80 per cent of the total area of the United States, which is about 3 million sq miles. Furthermore, the discharge measurements include some areas without the limits of the United States in the case of the St. Lawrence and Columbia River basins. However, the result appears reasonably accurate.

The various districts show wide variations. The Mississippi Basin, with about 41 per cent of the total area of the country and 51 per cent of the gauged area and about 6 in. of runoff, is important in effect. The Ohio, with about 16 per cent of the area of the Mississippi Basin, contributes just about one-half of its average flow. The Missouri, however, with 42 per cent of the basin area, yields only about 11 per cent of the average flow. Again, New England, with a gauged area of less than 3 per cent of the Mississippi Basin and about 7 per cent of the area of the Missouri, has an average yearly runoff equal to about 90 per cent of that of the Missouri.

The St. Lawrence and Columbia Rivers each contribute about one-sixth of the average country-wide flow, as shown in column (6) of Table 4·6. A comparison of columns (6) and (3) shows clearly the relative yields of the different sections of the country.

CHAPTER 5

THE FLOOD HYDROGRAPH—FLOOD FREQUENCY

An important means of studying flood flows is afforded by the *flood hydrograph*, with which, for a given point or station upon a river, a continuous record of stage is obtained for the entire period of the storm flow. By a discharge-rating curve applied to the record of gauge heights, a continuous curve of time discharge is obtained that has many interesting and useful characteristics.

This curve is influenced in form by all the various factors affecting the regimen of the river above the station, and the manner of its modification as greater floods occur is of great significance in the prediction of their characteristics and volume.

The flood hydrograph may be considered as the integration of all the upstream factors affecting flow—even to the confines of the drainage area, although often affected to a greater extent by conditions nearer the station. A series of flood hydrographs at different stations along a river furnishes a basis for a complete flood picture for the river and the most effective means for flood studies.

Examples of Flood Hydrographs

1. Susquehanna River at Harrisburg, Pa. In Fig. 5·1 the curve *ABCEF* shows a flood hydrograph for the Susquehanna River at Harrisburg, Pa., with 24,100 sq miles drainage area, based upon the records for the period Mar. 17–25, 1936. At the top of Fig. 5·1 is shown the occurrence of rainfall at Harrisburg to give a general idea of the rainfall distribution. Some runoff from melting snow also occurred.

The base line of the hydrograph *AFE*, varying from about 150 to 200 cfs, was arbitrarily placed to segregate approximately the runoff of this particular flood.

Beginning at *A*, the rate of flow for the *rising* limb of the curve increases gradually to *B*, with a maximum rate of increase in flow to about B^1, thence with a lessening rate of increase to the point of maximum flow at *C*.

The *descending* limb of the curve decreases gradually in flow from *C* to *O* and thence more regularly to the base curve at *E*.

The portion *DE* of the curve represents flow due to the draining off of the river valley as stage decreases. Beyond point *D*, as will be further noted, the flow is practically limited to this valley storage, as first suggested by Horton.[1]

A determination of valley inflow[2] is made in Fig. 5·1 shown by the curve *AGCF*, which was determined as follows, as shown in Table 5·1.

Column (1) Flow in thousand cfs at intervals of 50,000 cfs.

Descending Limb:

Column (2) Portion of day required for flow interval.

Column (3) Flow above base for flow interval in thousand cfs days.

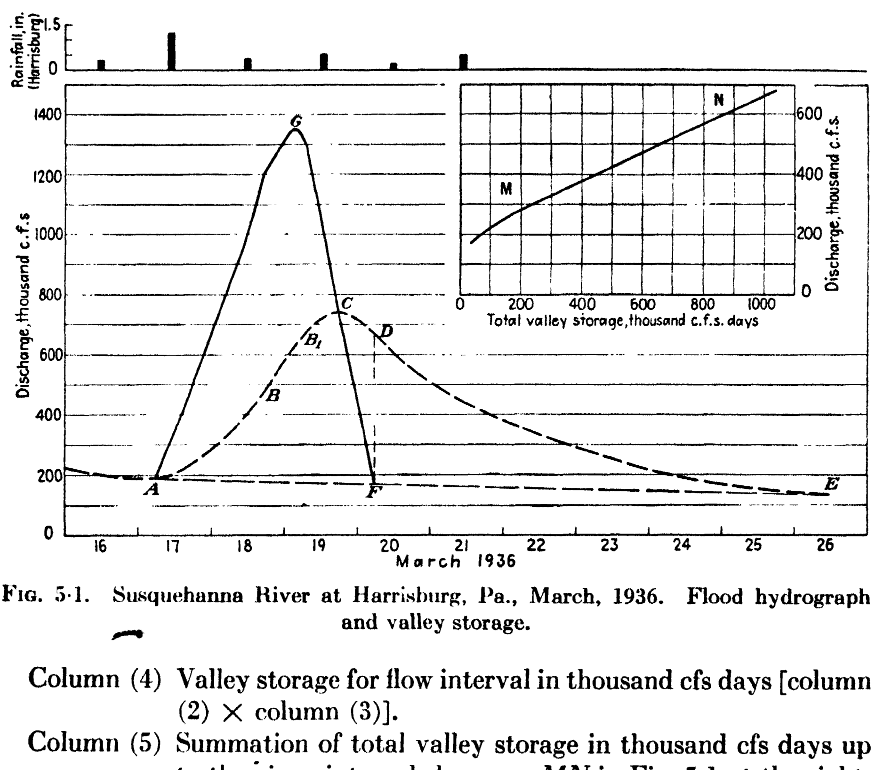


FIG. 5·1. Susquehanna River at Harrisburg, Pa., March, 1936. Flood hydrograph and valley storage.

Column (4) Valley storage for flow interval in thousand cfs days [column (2) × column (3)].

Column (5) Summation of total valley storage in thousand cfs days up to the given interval shown as *MN* in Fig. 5·1 at the right, a plot of column (5) against flows in column (1).

Rising Limb:

Column (6) Portion of day required for flow interval.

Column (7) Valley storage in cfs days for the interval taken from the curve, as noted under column (5).

[1] HORTON, ROBERT E.: Natural Stream Channel Storage, *Trans. Am. Geophys. Union*, 1936, Part II, pp. 406–415.

[2] BARROWS, H. K.: A Study of Valley Storage and Its Effect upon the Flood Hydrograph, *Trans. Am. Geophys. Union*, 1942, Part II, pp. 483–488.

Table 5·1 Susquehanna River at Harrisburg—Mar. 17–25, 1936

Computations of Valley Inflow

	Descending limb				Rising limb			
Flow, thousand cfs (1)	Days for interval (2)	Flow above base, thousand cfs (3)	Valley storage, Col. (2) × (3), thousand cfs days (4)	Σ valley storage, thousand cfs days (5)	Days for interval (6)	Valley storage for interval, thousand cfs days (7)	$\frac{\text{Col. (7)}}{\text{Col. (6)}}$ thousand cfs (8)	Valley inflow, Col. (8) + Col. (1) thousand cfs (9)
150								
200	1.37	30	41	41	......	60		
250	0.85	75	66	107	0.375	75	200	425
300	0.63	120	76	183	0.27	100	370	645
350	0.58	180	104	287	0.23	106	460	785
400	0.52	215	112	399	0.19	106	560	935
450	0.42	260	110	509	0.167	106	640	1,065
500	0.35	300	105	614	0.145	106	730	1,205
550	0.27	360	98	712	0.145	106	730	1,255
600	0.25	400	100	812	0.145	106	730	1,305
650	0.23	490	111	923	0.145	106	730	1,355
700	0.21	530	110	1,033	0.17	106	620	1,295
740					0.33	84	304	1,040
					0.375	84	−224	496
700								
650					0.21	106	−500	175

Column (8) The "build-up" in valley storage in cfs corresponding to amount in column (7) during the time noted in column (6), or column (7)/Column (6).

Column (9) The valley inflow for the various intervals of flow in column (1), which is column (8) plus the mean initial flow in column (1).

This computation, as will be noted, is based upon the fact that (referring to Table 5·1), as river stage rises, the actual flow or runoff, for illustration, for the interval 300,000 to 350,000 cfs has been diminished by the amount of valley storage existing in this interval of 106,000 cfs days for a time of 0.23 day or 460,000 cfs. This amount, added to the interval mean flow of 325,000 cfs, gives 785,000 cfs as the valley inflow, unaffected by storage.

The *valley-inflow* curve $AGCF$ is approximately triangular in shape, with point G vertically over point B, running through the point of maximum runoff C and meeting the flow base line at F directly under the point of inflection D in the runoff curve, beyond which (toward E) the flow is substantially all from valley storage.

It will also be seen that the curve of valley inflow $AGCF$ encloses an area equal to the area under the runoff curve $ACDEF$. Also, total valley storage corresponds to the area $AGCB_1B$ and again to the area $CDEF$. It also defines the upper usable limit of the curve MN in Fig. 5·1.

For Fig. 5·1, these values are

Item	Million cfs days	Million acre ft	in. depth
Runoff and valley inflow	1.81	3.62	2.8
Valley storage	1.06	2.12	1.63

The ratio of peak flows of valley inflow and runoff, respectively, is $1{,}350{,}000/740{,}000 = 1.81$, as compared with an approximate storage effect of $(2.8 + 1.63)/2.8 = 1.58$.

2. Ohio River at Metropolis, Ill. In Fig. 5·2 are shown a flood hydrograph, valley-inflow curve, valley storage, and rainfall chart for the Ohio River at Metropolis, Ill., with the large drainage area of 203,000 sq miles—practically at the mouth of the Ohio River. This covers the period of Dec. 25, 1936, to about Mar. 15, 1937, some 80 days, during which occurred the greatest flood of record upon the lower Ohio River. Upon such a large river, it takes a series of storms, as will be noted by the rainfall chart, to produce such a flood.

Nevertheless, the curve of runoff is, on a much larger scale, similar in general form to that in Fig. 5·1 for the Susquehanna River with approximately one-tenth the drainage area. Comparative peak flows were as follows:

River	Peak flow, cfs	Unit peak flow, csm
Ohio	1,780,000	8.8
Susquehanna	740,000	30.8

The much larger *unit flow* of the smaller river is typical.

The valley-inflow curve shows some irregularity in its rising limb because of the manner of rainfall and is approximately trapezoidal in shape, rather

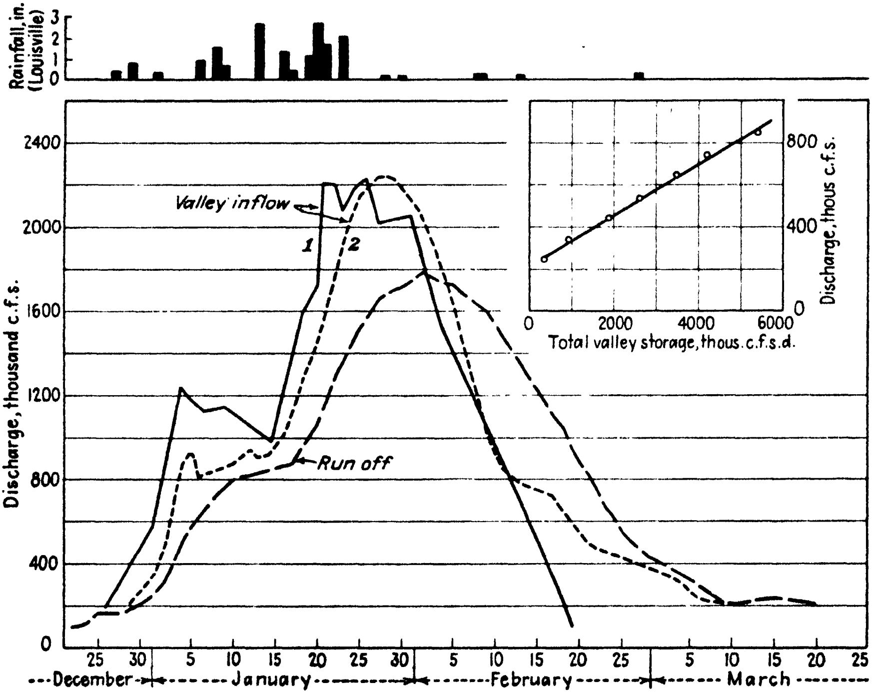


FIG. 5·2. Ohio River at Metropolis. Flood hydrograph and valley storage, 1936–1937.

than triangular. In Table 5·2 are given further details taken from Fig. 5·2 and its supporting computations. Note that, in Fig. 5·2, two determinations of valley inflow are made, the second (2, dotted line) with smaller increments of cfs than (1).

3. White River at West Hartford, Vt. Similar curves of runoff, valley inflow, and storage are shown in Fig. 5·3 for the White River at West

Hartford, Vt., for Sept. 19–25, 1938 (the so-called "hurricane flood"), this being a relatively small river, with drainage area of 690 sq miles.

4. Lead Mine Brook near Thomaston, Conn. In order to include a very small drainage area, curves of runoff, etc., are shown in Fig. 5·4 for Lead Mine Brook near Thomaston, Conn., with an area of only 24 sq miles.

In comparing Figs. 5·1 to 5·4, it must be kept in mind that there is necessarily a great variation in scale in showing adequately the flow for

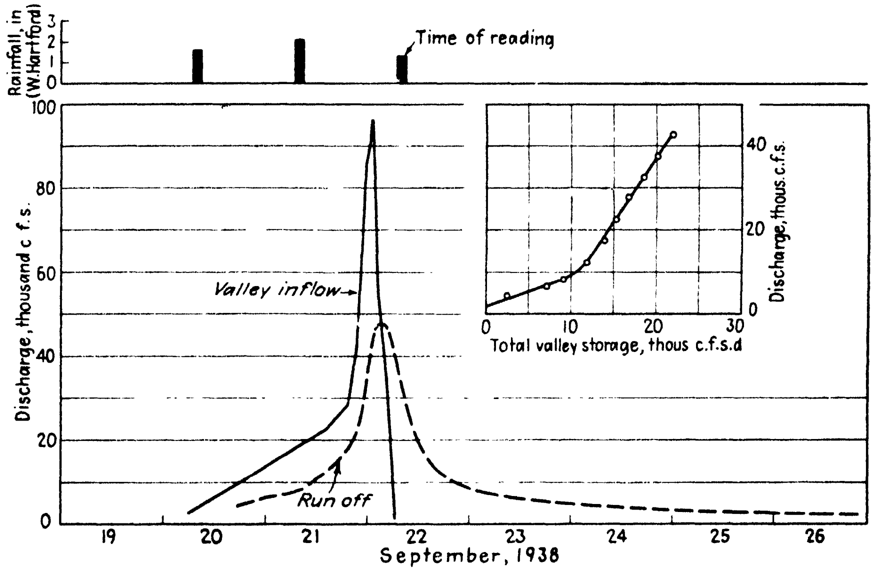


FIG. 5·3. White River at West Hartford, Vt., Sept. 19–26, 1938. Flood hydrograph and valley storage.

drainage areas of such widely different sizes. If these were, for instance, all plotted on one sheet with the scales of Fig. 5·2, for the Ohio River, the graph for Lead Mine Brook would hardly be large enough to see and that for White River, with 690 sq miles area, would also be very small in size. They would then be more nearly similar in form but would differ greatly in size.

5. Summarized Comparison of Runoff and Valley-storage Characteristics. In addition to the four foregoing rivers, studies have been made of flood hydrographs and valley storage for numerous other rivers, as shown summarized in Table 5·2. Those for the Ohio River stations are of special interest.

A study of Table 5·2 shows a range in total valley storage varying in amount from 33 million acre-ft on the Ohio River to 2,160 acre-ft on the small Lead Mine Brook in Connecticut of 24 sq miles drainage. On the other hand, the *unit* or square-mile amounts for these areas, so widely divergent in size, are, respectively, 162 and 90 acre-ft, or not widely different. Still more significant, adjusting to a 1-in. runoff basis, valley storage expressed in inches depth per inch of runoff is 0.28 for the Ohio at Metropolis and 0.45 for the small Lead Mine Brook, showing comparatively a greater unit area amount for the *small brook*.

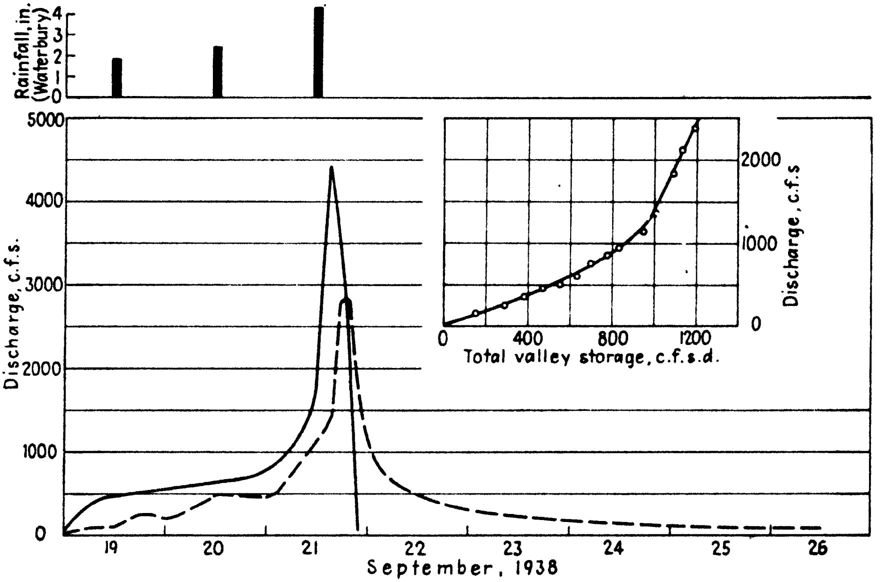


FIG. 5·4. Lead Mine Brook near Thomaston, Conn. Flood hydrograph and valley storage, September, 1938.

This same tendency also appears upon the Merrimack and Delaware Rivers, where data for three stations upon each river basin are given, *viz.*, for valley storage, expressed as inches depth per inch of runoff, which tends to *increase* in amount as drainage area *decreases*.

The effect of valley storage in reducing runoff peaks is shown in column (8) of Table 5·2, where the ratio of valley inflow and runoff peaks is given. This ratio varies from about 1.5 to 2.0 or more on many of these rivers but averages only about 1.20 on the Ohio River stations.

The ratio of the peaks of valley inflow and runoff would be approximately in accord with the relation of valley storage and inflow (or runoff) (see Figs. 5·1 to 5·3) or the "storage effect" as a ratio would be

$$\frac{\text{Runoff} + \text{valley storage}}{\text{Runoff}} \quad \text{or} \quad \frac{\text{Col. (3)} + \text{(5)}}{\text{Col. (3)}} \text{ in Table 5·2}$$

Table 5·2 Various Rivers—Summary of Runoff and Valley-storage Characteristics

River and station	Drainage area, sq miles	Runoff, in.	Valley storage				Peak ratio, valley inflow to runoff	$\frac{\text{Col. (3)} + \text{(5)}}{\text{Col. (3)}}$
			Acre-ft	In.	Acre-ft per sq mile	In. per in. of runoff		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Ohio:								
Metropolis, Ill.	203,000	10.5	33 million	3.0	162	0.28	1.25	1.28
Evansville, Ind.	107,000	9.8	13 million	2.3	121	0.23	1 38	1.23
Louisville, Ky.	91,200	7.7	9.8 million	2.0	107	0.26	1 24	1.26
Cincinnati, Ohio	76,000	8.0	6 2 million	1.5	82	0.19	1.20	1.18
Huntington, W. Va.	55,200	6.8	3.8 million	1.3	68	0.19	1.08	1.19
Pomeroy, Ohio	39,950	7.8	2.8 million	1.3	70	0.17	1.16	1.17
Wheeling, W. Va.	24,800	6.0	1.8 million	1.4	73	0.23	1.14	1.23
Susquehanna, Harrisburg, Pa.	24,100	2.8	2.2 million	1.70	92	0.60	1.80	1.60
Delaware:								
Riegelsville, N. J.	6,344	1.90	300,000	0.88	71	0.47	2.32	1.47
Belvidere, N. J.	4,542	2.04	272,000	1.12	60	0.55	2.12	1.53
Port Jervis, N. Y.	3,076	2.14	252,000	1.54	82	0.72	2.43	1.71
White:								
West Hartford, Vt.	690	2.43	41,200	1.17	63	0.48	2.00	1.50
Bethel, Vt.	241	3.20	16,700	1.30	69	0.40	1.83	1.41
Lead Mine Brook, Thomaston, Conn.	24	3.76	2,160	1.68	90	0.45	1.83	1.43
Winooski, Essex Junction, Vt.	1,079	1.53	28,600	0.50	26	0.33	1.18	1.33
Quinnebaug, Jewett City, Conn.	711	3.28	58,000	1.30	81	0.40	1.78	1.40
West, Newfane, Vt.	308	4.55	32,800	2.00	106	0.44	2.10	1.41
Merrimack:								
Lowell, Mass.	4,635	2.75	268,000	1.08	58	0.39	1.60	1.40
Franklin, N. H.	1,000	4.10	92,000	1.70	92	0.41	1.50	1.42
Plymouth, N. H.	622	4.70	72,000	2.20	116	0.47	1.92	1.46

Thus, for the Ohio River at Metropolis (line 1, Table 5·2), in column (9), the storage effect would be $(10.5 + 3.0)/10.5 = 1.28$ as compared with a peak ratio of 1.25. Note that the average of the seven Ohio stations shows these ratios as 1.20 and 1.22, respectively. In general, however, for the other stations, the peak-flow ratios are 20 to 40 per cent in excess of the storage ratios (about 1.9 and 1.4, respectively), as might be expected from the generally irregular form of the valley-inflow curves.

Results for the Ohio River stations can be checked by reference to *Water Supply Paper* 838 of the U.S. Geological Survey,[1] page 604, where valley or channel storage is estimated for each of the different reaches of the Ohio River in the flood of 1937. This comparison is shown in Table 5·3.

The method of determining valley storage in *Water Supply Paper* 838 was essentially as follows: For a given reach, tabulations were prepared of cumulative differences by 5-day periods from Dec. 25, 1936, to Feb. 28, 1937, between the sums of the measured or estimated daily flow into and outflow from the reach. These cumulative differences are equal to the water stored or released in the given reach for the period considered. Plotting total storage volumes against mean river stage, based upon gauges in the middle and both ends of each reach, fairly definite stage-storage relations were derived for seven records of the main river and the lower reaches of principal tributaries. From these relations, the daily volume of water or storage was determined in each reach in excess of the amount in the reach on Dec. 25, 1936, taken as a base. The results are believed to be of comparative accuracy.

In Table 5·3 are shown the successive reach valley-storage amounts—in column (3) based upon the results in Table 5·2, in column (5) by those given by the Geological Survey, and in columns (4) and (6) summation values for each method, for progressive comparison.

Considering the entirely different methods of determining valley storage, the results as shown by Table 5·3 are in fairly good agreement. Although the results for individual reaches differ somewhat, the total valley storage for the six reaches differs by only about 5 per cent. It is interesting to note that, from the various reach flood hydrographs, valley storage can be determined with fair accuracy and in a short time by the method outlined in Fig. 5·1 and Table 5·1, whereas the method used in *Water Supply Paper* 838 involves extended computations and diagrams, as well as additional field observations on tributary streams.

A further check of the Ohio River total valley storage is afforded by the total amount on Jan. 28, 1937, when the flood was a maximum, stated as 27.3 million acre-ft on page 605 of *Water Supply Paper* 838. The corre-

[1] Floods of Ohio and Mississippi Rivers, January–February, 1937.

Table 5·3 Valley Storage on Ohio River—1937 Flood

Station	Valley storage, thousand acre-ft				
	From Table 5·2, Col. (4)			From *Water Supply Paper* 838, p. 604	
	Total	For interval	Σ Col. (3)	For interval	Σ Col. (5)
(1)	(2)	(3)	(4)	(5)	(6)
Metropolis, Ill.	33,000				
		20,000	20,000	16,000	16,000
Evansville, Ind.	13,000				
		3,200	23,200	6,600	22,600
Louisville, Ky.	9,800				
		3,600	26,800	2,500	25,100
Cincinnati, Ohio	6,200				
		2,400	29,200	2,800	27,900
Huntington, W. Va. . . .	3,800				
		1,000	30,200	890	28,790
Pomeroy, Ohio	2,800				
		1,000	31,200	850	29,640
Wheeling, W. Va.	1,800				

sponding figure by the first method is 28.2 million acre-ft, or a difference of only about 3 per cent, again confirming its substantial accuracy.

The significance of valley storage in lessening peaks of floods is shown by this total of some 27 million acre-ft, or about 130 acre-ft per sq mile for the total Ohio River drainage of 203,000 sq miles. This is 1.8 in. depth and 22 per cent of the average runoff for the greatest flood of record.

Form of Valley-storage Curves. In Fig. 5·5 are shown curve relations for valley storage and discharge for the seven Ohio River stations—all to the same peak. These are straight lines, as is usually the case, with varying slopes, showing lesser total amounts and hence steeper curve slopes proceeding upstream. Note, however, by reference to Table 5·2, that, when adjusted to a runoff of 1 in., these all depart but little from their average of 0.22 in.

Susquehanna River at Harrisburg (Fig. 5·1) shows a valley-storage–flow relation slightly curved near its lower end. The straight line shown, however, could have been used through the entire range of discharge without material error.

White River at West Hartford (Fig. 5·2) (690 sq. miles) shows a distinct two-line curve of valley storage, with an angle at about 10,000 cfs discharge (or about 14 csm).

Lead Mine Brook (24 sq miles) is definitely curved.

Horton,[1] in his initial discussion of discharge–valley-storage curves, expressed this in the form $Q_s = K_s S_r^M$, which plots as a straight line on

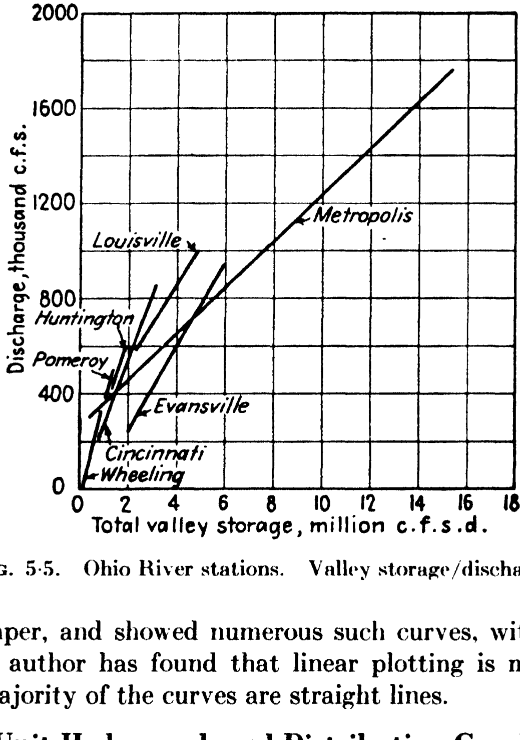


FIG. 5·5. Ohio River stations. Valley storage/discharge.

logarithmic paper, and showed numerous such curves, with values of K_s and M. The author has found that linear plotting is more adaptable, because the majority of the curves are straight lines.

Unit Hydrograph and Distribution Graph

The basis for the unit hydrograph consists in the assumption that, at a point in a given drainage-basin surface, runoff from rainfall occurring in a unit of time, as a day, will produce hydrographs of approximately equal bases and the ordinates will vary with the magnitude of runoff. This appears to be generally the case except for a large storm that exceeds the concentration period T_c of the area in question, the latter being defined as the time required for water to arrive from the farther limits of the basin.

A *distribution graph* is first made by selecting an isolated storm and determining, for the successive days of runoff due to that storm, the percentages of the total storm runoff. The procedure is illustrated in Fig. 5·6

[1] *Trans. Am. Geophys. Union*, 1936, Part II, pp. 406–415.

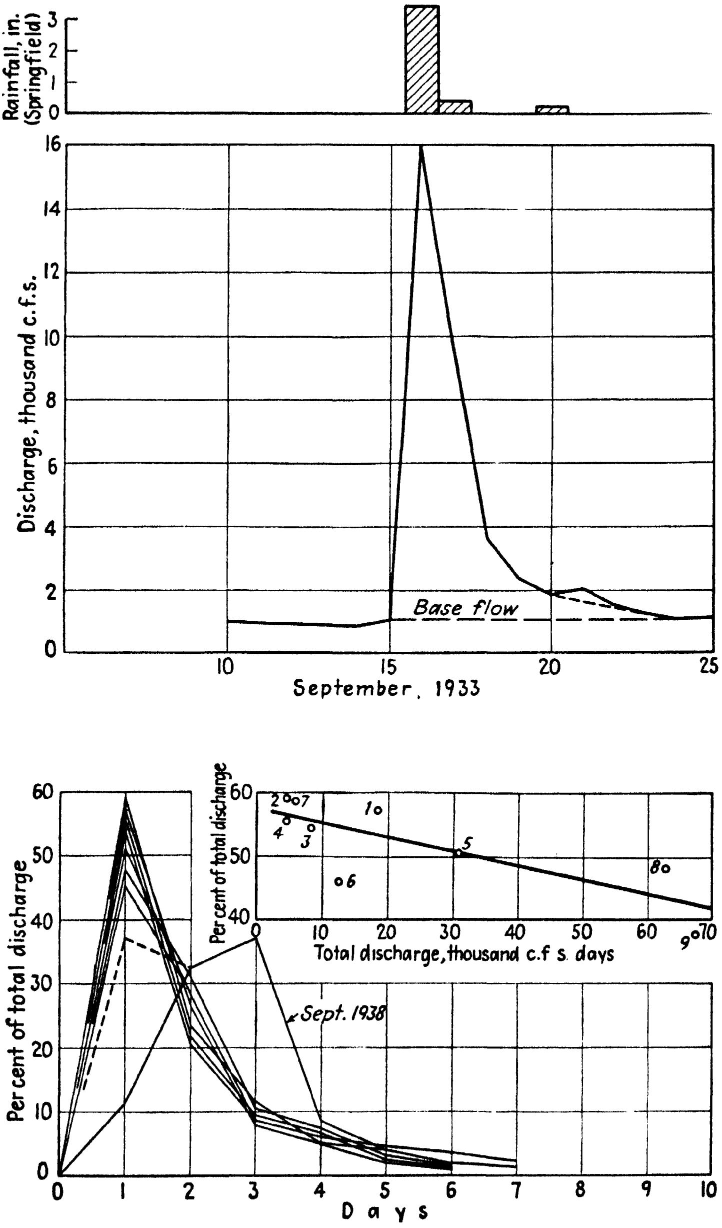


FIG. 5·6. Westfield River at Westfield, Mass. Flow-distribution curves.

and Table 5·4, for the Westfield River at Westfield, Mass., drainage area 497 sq miles.

In the upper portion of Fig. 5·6 is a hydrograph and accompanying rainfall chart for a storm in September, 1933. The *base flow*, or that just prior to and after the storm effect, is also indicated. In Table 5·4, line 5, are the percentages of total storm runoff for each day, computed from the hydrograph, with relation to the total runoff shown in the last column.

Data for eight other storms are also given in Table 5·4.

Table 5·4 Computations for Flow-distribution Curve, Westfield River, Westfield, Mass.

Storm (peak day)	Per cent of total runoff above base flow							Total runoff above base flow, cfs days
	Days							
	1	2	3	4	5	6	7	
1. Oct. 24, 1923	57.2	26.7	8.0	4.8	2.1	1.1	....	18,700
2. Oct. 1, 1924	59.1	21.5	9.5	6.4	2.4	1.1	...	4,700
3. Jan. 19, 1926	54.2	20.5	8.4	6.0	4.8	3.6	2.4	8,300
4. Oct. 23, 1929	55.7	23.2	11.4	4.6	3.5	1.6	...	4,300
5. Sept. 16, 1933	50.5	27.9	8.6	5.3	3.3	2.7	1.7	30,100
6. Sept. 17, 1934	45.2	27.8	10.3	5.6	4.8	4.0	2.4	12,600
7. Sept. 9, 1934	58.7	23.8	11.0	3.7	1.8	0.9	.	5,500
8. Mar. 18, 1936	48.0	31.0	10.7	7.6	3.1	....	..	63,300
9. Sept. 22, 1938	11.5	32.4	37.2	8.4	4.3	1.9	1.2	67,600

In the lower part of Fig. 5·6 is plotted the group of flow-distribution curves from Table 5·4. These all have approximately the same base length or time of duration and are similar in form, with the exception of storm No. 9, for which the third day is that of peak, instead of the first day, as with all the other curves. Plotted in this form, a basis is available for defining an approximate flood hydrograph for a higher storm and the corresponding flow for the peak day. The method is useful in the many cases for which only mean daily flows are available.

At the lower right in Fig. 5·6, a graph is shown relating percentage of peak flow to total discharge for each storm. This relation is approximately determined and shows a definite tendency toward a lessening in peak percentage as total flow increases. It is evident that, in the use of the flow-distribution curve for the estimation of highest daily flow, beyond the range of existing data, it is essential to include as an auxiliary such a relation, rather than to average the peak percentages without reference to the various magnitudes of total storm flow.

The peak 24-hr period may not coincide with the peak calendar day. Thus, referring to Fig. 5·1, page 55, for the Susquehanna River at Harrisburg, the peak calendar day (at noon, Mar. 19) shows an average flow of 690,000 cfs. but the peak for the maximum 24-hr period, at 6 P.M. of Mar. 19, was about 710,000 cfs.

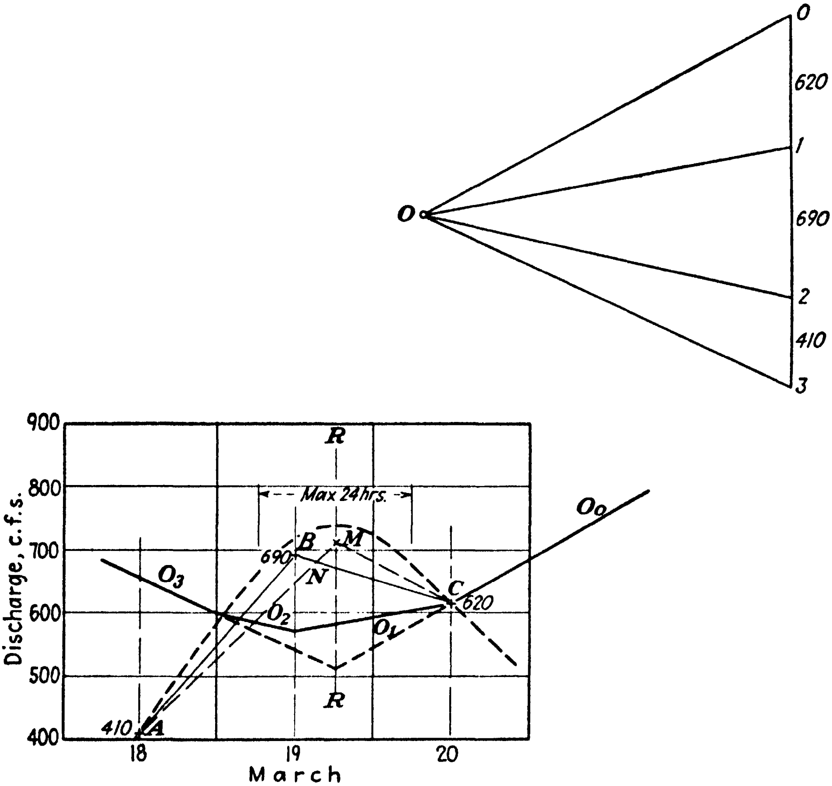


FIG. 5·7. Location of highest flow on daily hydrograph.

This may be determined as shown in Fig. 5·7, where the upper part of the flood hydrograph is shown to the same scale as in Fig. 5·1. Here *A*, *B*, and *C* are respective day average flows for Mar. 18 to 20. The time location *R-R* for the highest 24-hr period is determined graphically, as will be noted, by the use of a "resultant force polygon," and the largest average 24-hr flow is at *M*, making the areas *MNC* and *ABN* alike by trial. The flow value at *M* is 710,000 cfs, or somewhat greater than at *B*, which is 690,000 cfs. It will be found that, as storm flow increases, the time to the peak day, T_c, will increase and frequently will result in a maximum 24-hr period other than that of the calendar day.

Note that the short-time actual peak flow in Fig. 5·7 was 740,000 cfs, in the ratio of 1.04 to the average 24-hr flow. The relation between these two quantities is given by Weston E. Fuller's formula (see page 43).

$$Q_{\text{max}} = Q_{24\text{ hr avg}} \times [1 + (2/A^{0.3})]$$

where A is the drainage area in square miles. For the Susquehanna in Fig. 5·7, with 24,100 sq miles drainage area, this formula would give 1.10, differing somewhat from the actual ratio. This method of obtaining peak flow from a daily average is only an approximation, as noted in the discussion of Fuller's formula (page 44).

Referring again to Fig. 5·6 for the Westfield River and the form of the distribution curves, the rainfall occurrence was in all cases, except that for the large storm of September, 1938, practically confined to one or two days prior to the peak day of flow. In this large storm, rainfall and runoff occurrence was as follows:

Date	Rainfall, in.		Runoff			Gauge height, ft
	Day	Total	cfs above base	In. day	In. total	
Sept. 17	0.45	0.45	455	0.03	0.03	4.1
18	1.10	1.55	413	0.03	0.06	4.0
19	2.40	3.95	700	0.05	0.11	6.0
20	2.95	6.90	7,790	0.59	0.70	11.5
21	2.00	8.90	21,900	1.65	2.35	18.4
22	....	....	25,100	1.90	4.25	19.7
23	....	....	5,680	0.45	4.70	10.1
24	....	....	2,890	0.22	4.92	
25	....	....	1,290	0.10	5.02	
26	....	....	810	0.06	5.08	

The Westfield River Basin is about 40 miles long and 12 miles in average width, with a mean elevation of about 1,130 ft above sea level and an elevation of about 100 ft at the Westfield gauging station. Mean land slope is about 11.0 per cent; mean stream slope 2.5 per cent. Aside from perhaps 5 per cent of urban area, the basin is about equally divided between second-growth forests and grassland. It is obviously a stream of quick runoff.

The time of concentration T_c was probably not over 15 to 20 hr. Thus, in the long storm of September, 1938, with four successive days of substantial rainfall, the unit graph on Fig. 5·6 varied materially from the usual curves, as will be noted. Even in this case, however, the base flow length of the hydrograph was substantially the normal amount.

Flow-characteristic Curves[1]

These are flood hydrographs modified in form by dividing both discharge and time by the square root of the drainage area and thus obtaining discharge and time upon a characteristically similar area of 1 sq mile.

A further modification may also be made by dividing the discharge by the total number of inches of flood runoff, thus obtaining flow per square mile for a 1-in. flood.

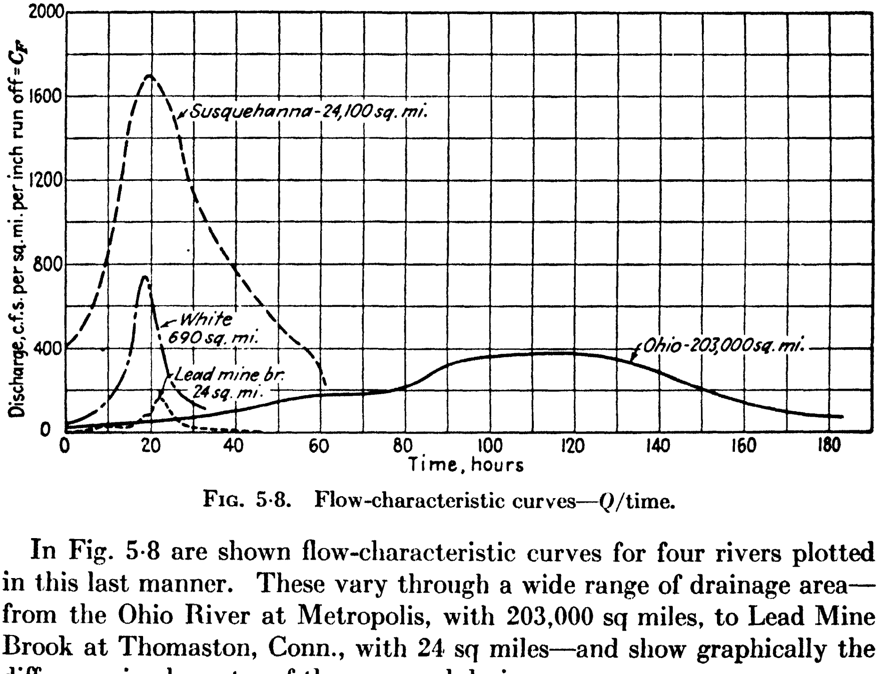


FIG. 5·8. Flow-characteristic curves—Q/time.

In Fig. 5·8 are shown flow-characteristic curves for four rivers plotted in this last manner. These vary through a wide range of drainage area—from the Ohio River at Metropolis, with 203,000 sq miles, to Lead Mine Brook at Thomaston, Conn., with 24 sq miles—and show graphically the difference in character of these several drainage areas.

This flow-characteristic curve is based upon the Boston Society of Civil Engineers flood formula[1] $Q = C_F\sqrt{A}\ R$ (see page 40); and, with A and R both equal to unity, the value of maximum Q in Fig. 5·8 for each curve gives the value of C_F in this formula. These values for the four rivers are as follows:

River	C_F
Susquehanna	1,700
White	680
Ohio	376
Lead Mine Brook	153

[1] Report of Committee on Floods, *Jour. Boston Soc. Civil Eng.*, September, 1930, pp. 392–395; also January, 1942, pp. 74–75.

The committee[1] suggested a range in values of C_F from about 1,000 to 100 or less. Evidently the Susquehanna, with 1,700, is unusually high.

In Fig. 5·9 a further transformation is made in the plotting by using percentage of time instead of time in hours. This is similar to the flow-distribution curve but upon the basis of 1 sq mile of drainage rather than

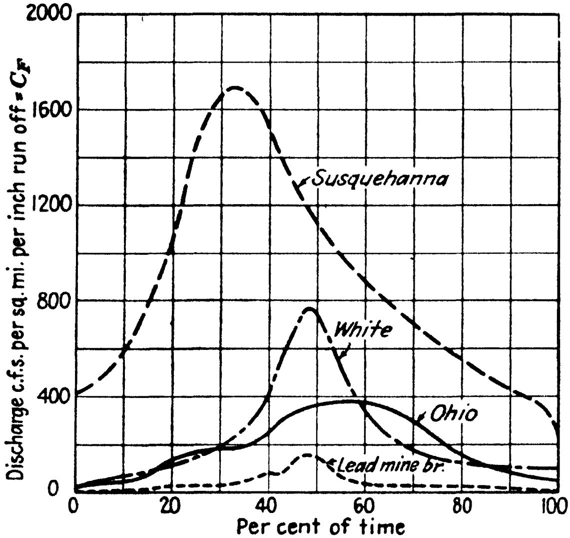


FIG. 5·9. Flow-characteristic curves—Q/per cent of time.

the entire area, as usually plotted. It gives a somewhat clearer picture of relative flow and peak characteristics.

Actually, as pointed out by Sherman,[2] all the various flow curves that have been discussed are simply modifications of the flood hydrograph, and the different quantities can also be shown by auxiliary scales placed at the sides and below the parent curve.

Comparison of Successive Flood Hydrographs

The comparison of flood hydrographs at the upper and lower ends of a stretch of river or reach gives another means of determining the extent of valley storage in the reach. At a given time, inflow at the upper station plus inflow from tributary streams in the reach would equal the outflow at the lower station if no valley storage were occurring or, in other words, if water elevation in the reach remained constant. Frequently, however, river storage is changing, so that outflow at the lower station will equal inflow to the reach plus or minus valley storage. Hence a comparison of

[1] *Ibid.*

[2] *Trans. A.S.C.E.*, Vol. 61, p. 365, 1935.

inflow and outflow at any time will determine the amount of valley storage in the reach.

In Table 5·5 are shown extracts from computations comparing inflow and outflow for the reach of the Ohio River between Evansville and Metropolis—a distance of 152 miles. All data of flow are taken from *Water Supply Paper* 838.[1] In line 1 is main-river inflow at Evansville.

Between Evansville and Metropolis, the Wabash River enters the Ohio from the north and the Cumberland and Tennessee Rivers from the south, with areas as follows:

River	Gauged area		Approximate ungauged area, sq miles	Total area, sq miles	Ratio total area to gauged area
	Station	Drainage area, sq miles			
Wabash	Mt. Carmel	28,600	7,200	35,800	1.25
Cumberland	Clarksville	16,000	3,000	19,000	1.20
Tennessee	Buchanan	39,700	1,500	41,200	1.05
Totals	....	84,300	11,700	96,000	

To take into account flow from ungauged areas, the above ratios have been applied to daily flow records at each of the three gauging stations, to get the approximate inflow from the drainage area of 96,000 sq miles between Evansville and Metropolis, in lines 2 to 4, Table 5·5.

Total inflow, lines 1 to 4, is given in line 5.

Outflow at Metropolis is given in line 6; line 5 minus line 6, or valley storage, in line 7 and the summation of valley storage in line 8.

Computations were made for each day from Dec. 25, 1936, to Mar. 9, 1937, with results for seven of these days given in Table 5·4.

December 26 is approximately the beginning of rising stage and accumulation of valley storage.

January 23–25 are successive days during the rising stage.

February 4 is the crest day for valley storage, with a total amount of 9,637,000 cfs days stored since Dec. 26.

February 9, 10, and 18 are typical days during the falling stage, with large withdrawals of valley storage.

March 9 is near the end of the falling stage, with some valley storage still on hand, however, as compared with Dec. 26.

In Fig. 5·10 is a plot of daily total valley storage against discharge at Metropolis, the outflow point. This should be studied and compared with

[1] Floods of Ohio and Mississippi Rivers of January–February, 1937, *Water Supply Paper* 838, U.S. Geological Survey.

Table 5·5 Comparison of Flood Hydrographs on the Ohio River at Evansville and Metropolis, 1936-1937

Station or item	Discharge, thousand cfs, or valley storage, thousand cfs days								
	Dec. 26	Jan. 23	Jan. 24	Jan. 25	Feb. 4	Feb. 9	Feb. 10	Feb. 18	Mar. 9
1. Evansville	111	926	1,030	1,170	1,180	716	595	368	94
2. Wabash	8	350	344	328	174	110	100	85	34
3. Cumberland	20	324	337	318	183	55	68	54	19
4. Tennessee	36	335	344	332	212	166	169	195	73
5. Total inflow, cfs	175	1,935	2,055	2,148	1,749	1,047	932	730	220
6. Outflow, Metropolis	168	1,360	1,440	1,520	1,740	1,600	1,540	1,090	227
7. Valley storage, cfs days	+7	+575	+615	+628	+9	−553	−608	−360	−7
8. Σ valley storage	7	5,079	5,694	6,322	9,637	7,935	7,327	3,891	1,252

Fig. 5·2 (page 58), which shows the runoff hydrograph and valley-inflow curve at Metropolis for this same period. It should be kept in mind that the valley-inflow curve in Fig. 5·2 includes the effect of valley storage both in and above the Evansville-Metropolis reach.

The valley storage on hand Feb. 4, 1937, at the time of approximate peak flow as determined in the computations for Table 5·5, is for the reach between Evansville and Metropolis and totals about 9.64 million cfs days or about 19.3 million acre-ft, which is in fairly good agreement with that

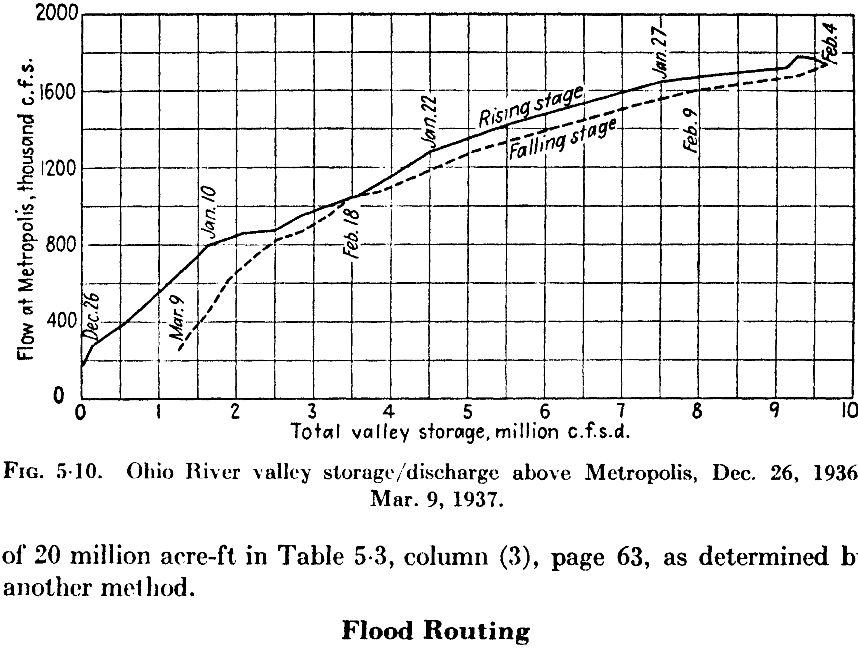


FIG. 5·10. Ohio River valley storage/discharge above Metropolis, Dec. 26, 1936–Mar. 9, 1937.

of 20 million acre-ft in Table 5·3, column (3), page 63, as determined by another method.

Flood Routing

An important problem in flood-control studies consists in the study of the effect of proposed reservoir storage systems upon the form or shape of flood hydrographs at different points on the river below the reservoir systems and the corresponding effect of these systems in reducing flood peaks. Where such flood hydrographs are available at several points upon the river and upon tributaries where reservoirs are planned for one or more major floods, the effect along the river of reservoir storage in such floods can be determined from the successive graphs; and, by means of distribution graphs determined from the major flood hydrographs, other larger assumed floods may be studied in a similar manner. The essential basis for such studies is therefore a sufficient number of flood hydrographs upon the main river and its tributaries.

As previously noted, the flood hydrograph at a given point reflects and automatically takes into account any valley storage in the reaches of the river lying upstream from the point where the hydrograph is taken.

The amount of valley storage may be determined by the two methods previously explained. It may also be estimated from profiles of water surface for the reach at different stages together with a sufficient number of valley cross sections at intervals in the length of the reach.

Flood Routing as Used in Muskingum Flood-control Project. Where hydrographs are not fully available, it may be desirable to interpolate at some point on the river, by computation, a flood hydrograph for use in routing floods, similar either to those of the past or to some possible assumed future flood.

A detailed method of analysis for this problem involving the estimation of valley storage was suggested and used in determining the effect of a reservoir system upon flood flows in connection with the Muskingum River Flood Control Project in Ohio.[1]

The basis for this was the following equation:

$$K = \frac{T\left(\frac{i_2 + i_1}{2} - \frac{d_2 + d_1}{2}\right)}{X[(i_2 - i_1) + (1 - X)(d_2 - d_1)]} \qquad (5\cdot1)$$

where K = ratio of storage increment in reach, cfs days, to the corresponding flow increment, cfs

T = time unit, days or fractions of a day

X = fraction of flow increment derived from inflow increment

i_1, i_2, etc. = total inflow to reach, cfs at beginning of successive time units T

d_1, d_2, etc. = total outflow of reach, cfs at beginning of successive time units T

It will be noted that this equation corresponds to the form $K = \triangle\text{storage}/\triangle Q$; or, more simply, the ratio of incremental valley storage to incremental Q is a constant K. This is probably not correct for the entire range of floods but is reasonably so for the range of larger floods.

The determination of X is made from available data from previous floods. The numerator of equation (5·1) (or incremental valley storage) is plotted against corresponding values of the denominator (or the weighted incremental discharge, usually the mean of inflow and outflow discharges) for different values of X, and the value of X is used for which the resulting plot is most nearly a straight line. This does not appear to give very definite results, and it seems questionable whether such a weighted dis-

[1] McCarthy, Gerald T.: "The Unit Hydrograph and Flood Routing," U.S. Engineer Office, Providence, R. I., June 24, 1938. (Unpublished.)

charge is essential. Usually X is of the order of 0.3, so that the denominator of equation (5·1) will be

$$0.3(i_2 - i_1) + 0.7(d_2 - d_1)$$

With equal weighting of inflow and outflow discharge increments, this would be

$$0.5(i_2 - i_1) + 0.5(d_2 - d_1)$$

or, in other words, the denominator would become the average of inflow and outflow incremental discharge.

It is also to be noted in support of the use of average incremental discharge that a plot of valley storage against such discharge conforms closely to a straight-line curve.

Flood-routing Procedure. In flood routing by the method used at Muskingum, i_1, i_2, and d_1 are known, and it is necessary to solve for d_2. From equation (5·1)

$$d_2 = -\frac{KX - 0.5T}{K - KX + 0.5T} i^2 + \frac{KX + 0.5T}{K - KX + 0.5T} i_1 + \frac{K - KX - 0.5T}{K - KX + 0.5T} d_1$$

which is in the form

$$d_2 = C_0 i_2 + C_1 i_1 + C_2 d_1 \tag{5·2}$$

where

$$C_0 = \frac{-KX - 0.5T}{K - KX + 0.5T} \qquad C_1 = \frac{KX + 0.5T}{K - KX + 0.5T}$$

$$C_2 = \frac{K - KX - 0.5T}{K - KX + 0.5T}$$

Graphs of C_0, C_1, and C_2 (which are called "routing coefficients") as abscissas, with three series of K as ordinates, *viz.*, for $T = \frac{1}{4}, \frac{1}{2}$, and 1 day, with curves for different values of X, are proposed for convenient use. Using this diagram, after K and X were determined, the routing coefficients C_0, C_1, and C_2 were interpolated, and flood routing was carried on by tabular solution of equation (5·2) for successive d_2's, beginning with the time when inflow to and outflow from the reach are approximately equal at the beginning of the flood hydrograph.

The suggested modification of equation (5·1), avoiding use of the factor X, is as follows:

$$K = \frac{T\left(\frac{i_1 + i_2}{2}\right) - \frac{d_2 + d_1}{2}}{\frac{i_2 - i_1}{2} + \frac{d_2 - d_1}{2}} \tag{5·1a}$$

Solving for d_2,

$$d_2 = i_1 - (i_2 - d_1)\frac{K - T}{K + T} \tag{5·2a}$$

This materially simplifies the successive determinations of d_2 without, it is believed, sacrificing essential accuracy in results.

The following table illustrates this modified method, using the period Jan. 10–29 (Table 5·5) to determine K.

	Discharge, cfs		Σ and Δ valley storage
	Jan. 10	Jan. 29	
Inflow	876,000	2,239,000	1,629,000
Outflow	795,000	1,700,000	8,640,000
Mean	835,000	1,964,000	7,011,000

$$K = \frac{7{,}011{,}000}{1{,}129{,}000} = 6.2$$

Note that ΔQ is taken with reference to the means of inflow and outflow discharge.

Assuming that inflow and outflow are available for Jan. 23 and that is it desired to compute outflow, d_2, for Jan. 24,

$$\begin{aligned} i_1 &= 1{,}935{,}000 \\ i_2 &= 2{,}055{,}000 \\ d_1 &= 1{,}360{,}000 \\ T^1 &= 1 \text{ day} \\ K &= 6.1 \end{aligned}$$

Then, using formula (5·2a),

$$\begin{aligned} d_2 &= 1{,}935{,}000 - (2{,}055{,}000 - 1{,}360{,}000) \times \frac{5.2}{7.2} \\ &= 1{,}935{,}000 - 505{,}000 = 1{,}430{,}000 \end{aligned}$$

The actual value of d_2 for Jan. 24 is 1,440,000.

By this method, values of d_2 are computed for successive days or other time periods, and in this manner an outflow hydrograph is developed. It may be desirable upon smaller rivers to use a time interval T of ½ day instead of 1 day.

Effect of Reservoirs upon Flood Peaks

Where a system of flood-control reservoirs is being planned in a river basin, the determination of the effect of storage in the reservoir system upon floods at points downstream in the river basin is a complicated problem often lacking definite information to permit an accurate solution. It involves the routing of the assumed flood through each reservoir, and thence downstream, and the gradual combination of flows from different reservoirs and intervening stretches of river into a flood wave progressing downstream to the point under consideration. To do this in detail usually involves numerous approximations and extended computations from which the results are likely to be of doubtful accuracy when completed.

One of the early attempts to solve this problem was made by the Pittsburgh Flood Commission in their 1912 report.[1] Approximate hydrographs of tributary floods were constructed based upon rainfall observations and an arbitrary runoff factor, and other hydrographs were plotted in a manner to permit time comparison of crests as well as volumes to obtain thus a rough idea of how tributary flood waves would combine. Obviously the method was subject to large or uncertain errors.

A method of flood routing to fill in gaps where flood hydrographs are not available has been described on page 74. Lacking a sufficient number of actual flood hydrographs as well as more accurate information as to channel velocities, however, detailed computations of reservoir effect, especially upon a large river system, are likely to be uncertain in accuracy as well as long and involved in character.

Hence it will often be sufficiently accurate to group the reservoirs in all or part of the river basin, consider their combined capacity, and determine their effect upon some actual flood hydrographs at the point below in the river where the flood peak is to be determined. An example of such a computation is given on page 150 for the Connecticut River Basin where the reduction in flood peak flow at Thompsonville in the March, 1936, flood is found to be 71 per cent, owing to storage on the river above.

A simpler method of treatment is based upon the principle formulated by the Flood Committee of the Boston Society of Civil Engineers,[2] which shows that the peak flow will vary directly with the total runoff, which is consistent with their formula for peak discharge,

$$q = \frac{1{,}290R}{T}$$

where R = flood runoff, in.
q = peak flow, csm
T = total flood period—a constant for a given river point

Example. As an example of this method, consider the Ohio River at Sewickley, Pa.,[3] somewhat below Pittsburgh, during the flood of 1936, with a drainage area of 19,500 sq miles.

[1] *Report of Pittsburgh Flood Commission*, 1912, Chap. VII.
[2] *Jour. Boston Soc. Civil Eng.*, September, 1930, pp. 378–381.
[3] *Water Supply Paper* 803, p. 13, *U.S. Geological Survey*.

Flood discharge Feb. 26–Apr. 20, 1936	6,706,000 cfs days
Base flow	1,460,000 cfs days
Total above base flow	5,246,000 cfs days

$$\frac{10,500,000}{19,500} = 540 \text{ acre-ft per sq mile} = 10.0 \text{ in.}$$

Flood peak discharge at Sewickley = 574,000 cfs on Mar. 18, 1936.

Assume storage capacity of 3,300,000 acre-ft in 11 reservoirs upon tributary streams above Pittsburgh, which is

$$\frac{3,300,000 \times 12}{19,500 \times 640} = 3.15 \text{ in.}$$

$$\text{Reduction in peak flow at Sewickley due to storage} = \frac{3.15}{10.0} = 0.32$$

$$\begin{aligned}
&& 574,000 &= \text{gauge height } 34.75 \\
574,000 \times 0.32 &= & 185,000 & \\
\text{Peak flow in cfs with storage} &= & 389,000 &= \text{gauge height } 26.8 \\
\text{Reduction in stage due to storage} &= & & \qquad\qquad 8.0
\end{aligned}$$

This result is consistent with crest reductions for proposed storage above Pittsburgh as determined by the U.S. Engineer's office[1] which were for the flood of March, 1936,

At Pittsburgh	7.7 ft
At Wheeling	8.5 ft

This method will be found useful for quick and reasonably good determinations of storage effect upon flood peaks.

Conclusion

In closing the subject of flood hydrographs, it is pertinent to accentuate again the importance of their availability at a sufficient number of stations and covering a range of higher stages, so as to include one or more floods of important magnitude. With such information adequately available upon a river system, the means will be available to study and solve the usual problems involved in flood control and relief, in a more detailed and accurate manner.

As has been noted, flow-distribution graphs based upon ordinary river stages and moderate floods are not well applicable in predicting volumes and peaks for high and rare floods, since the latter are likely to involve long storms, in excess of the concentration period, with flow characteristics modified from those prevailing for the common storm.

The present general equipment of the river-gauging stations of the U.S.

[1] Com. Doc. 1, H.R., 75th Cong., 1st Sess., p. 12.

Geological Survey with automatic recording gauges is now continually adding to our accumulation of flood hydrographs for higher river stages. As time goes on, less difficulty should be found in procuring adequate data of this kind, and less recourse will have to be made to supplying estimates by computation instead of using more dependable observed values as a basis for the study of important flood-relief projects.

Flood Frequency.[1] The consideration of flood frequency occurs in the study of the economics of flood control, as discussed later. The most effective method for determining flood frequency consists in plotting frequency in years or percentage of time against peak discharge on logarithmic paper, drawing a smooth curve through the plotted points, and extending this in the higher brackets by eye.

Statistical Methods. Various statistical methods are used in the determination of frequencies based upon the probability equations of Foster or Hazen and the Slade method, which consists of the statistical method between upper and lower limits fixed by nonstatistical data. These are merely methods of extending the use of data beyond the length of record; and, since the accuracy of results depends basically upon this length of record, it does not appear that the use of statistical methods adds to the dependability of the results over and beyond that obtained by the use of logarithmic plotting and extension of curves by eye. A full discussion of these statistical methods is given in *Water Supply Paper* 771 of the U.S. Geological Survey, Floods in the United States.

Connecticut River—Flood Frequencies. The greater portion of available stream-flow records in the United States are limited in time to 30 or 40 years, which is usually not a sufficiently long period to use as a basis for curve extension and the determination of rare flood frequencies. The longest available record of stage in this country is upon the Connecticut River at Hartford or Thompsonville, continuously available since 1843, or about 100 years. Information upon floods is available back to 1801.

Even in this case, the problem is complicated by the occurrence of two major floods during the past 8 years, *viz.*, in March, 1936, and September, 1938. These were both rare floods, and their inclusion in a frequency plot is questionable, as may be seen by reference to Fig. 5·11 showing the probable frequency of peak discharge on the Connecticut River at Hartford on three different bases.

In Fig. 5·11*A*, the curve is based on the period of 98 years through 1935, omitting the two major later floods, which upon this plot would show a frequency higher than 1,000 years, and the 1936 flood would probably never have occurred. In Fig. 5·11*B*, the 1936 flood is included as one of

[1] Report of the Boston Society of Civil Engineers Committee on Floods, *Jour. Boston Soc. Civil Eng.*, January, 1942, pp. 105–113.

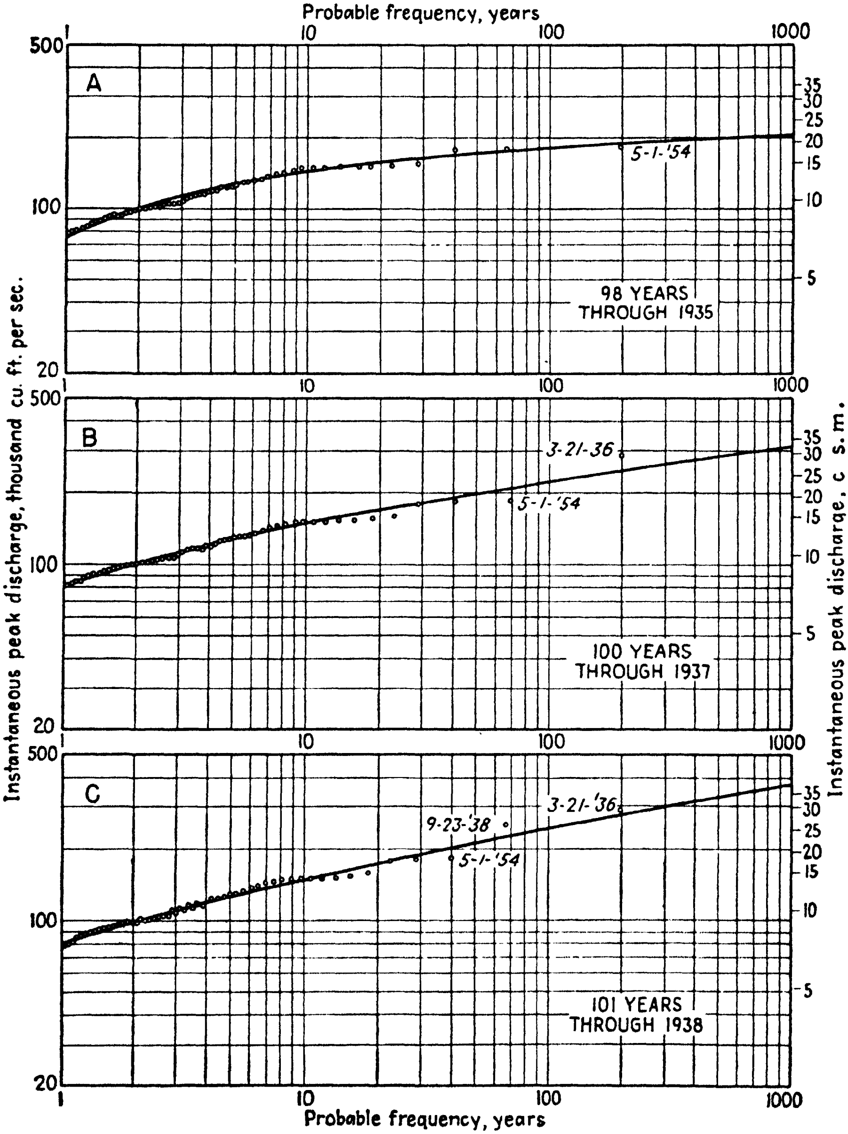


Fig. 5·11. Connecticut River at Hartford. Peak-discharge frequency.

the points in the plot; and, in Fig. 5·11*C*, both the 1936 and 1938 floods are included.

Frequency of the 1936 flood by these three plots would be

Fig. 5·11*A*, 98-year record	Probably never
Fig. 5·11*B*, 100-year record	Once in 500 years
Fig. 5·11*C*, 101-year record	Once in 265 years

Moreover, dividing the record into two 48-year periods, 1843–1890 and 1890–1938, as in Fig. 5·12 for Thompsonville, would show for the earlier

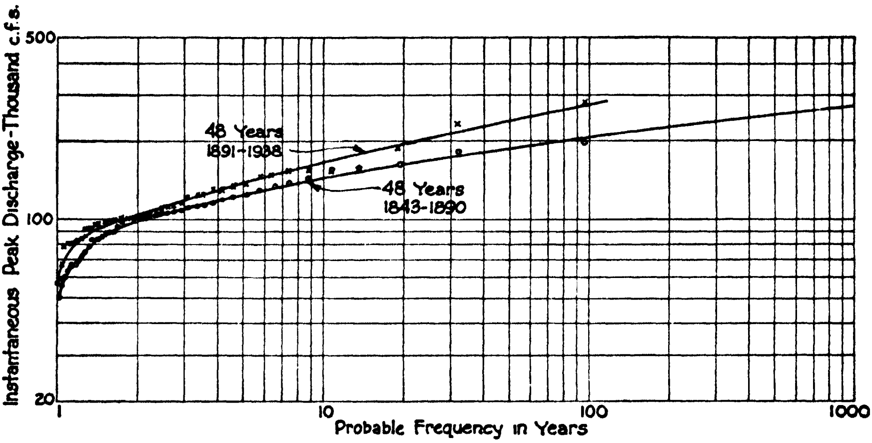


Fig. 5·12. Connecticut River at Thompsonville. Frequency.

half a frequency for a flood like 1936 of once in 1,300 years and for the later half, once in 108 years. This shows that a 48-year record, or longer than exists on most streams, is not nearly long enough to give valid information on the frequency of a very large flood.

With reference to the Connecticut River and the 1936 flood, the committee[1] made no attempt to fix closely the frequency of the 1936 and 1938 floods but stated that the 1936 flood was undoubtedly a major flood with a "frequency" of several hundred up to perhaps a thousand years.

Conclusions. The use of frequency figures is practically limited to the matter of economics of flood-control projects and particularly to the economics of average yearly flood losses, as noted later. Practically, these are largest for lower or intermediate frequency values; since, although a thousand-year flood, for example, might bulk large in total losses, its *average yearly* losses would be small because the time period is so long. Hence a 40- or 50-year flood record, with frequencies determined as well as may be by logarithmic plot, will usually be adequate in economic studies of flood benefits.

[1] *Ibid.*

CHAPTER 6

SPILLWAYS AND SPILLWAY GATES—COSTS

Spillways

An essential part of any dam is a spillway over which floodwaters may be passed and carried to the level below the dam. In the typical form, the ordinary concrete spillway has an upstream face vertical or with slight batter, a rounded crest, but with space for access purposes and for flashboards, where these are installed. The downstream face is approximately parabolic with a vertical axis and designed to have the flowing floodwater adhere to the face; with a downward curve, or "bucket," near the foot of the slope, which is about 50 deg with the horizontal. The "bucket" usually curves to the horizontal, at which level it is about the elevation of ordinary backwater level below the dam. This arrangement is intended to carry the water smoothly through the change of level and on its downstream path to the river below.

The spillway is usually normal or nearly so to the direction of river flow and designed to carry adequately the maximum expected flood flow. Side-channel spillways below the dam are also occasionally used in steep sloped valleys where room is lacking for the ordinary type.

Another type of spillway in limited use is the tunnel type. A circular spillway just above the dam discharges through a vertical shaft into a tunnel in rock running under the dam usually near the riverbank. This is sometimes called the "morning glory" type, from the general shape of the spillway and shaft entrance. It requires sufficient head to permit a tunnel section of reasonable cost and is best adapted for use at a dam with large reservoir area to reduce peak flows and for limited drainage areas. It is also well adapted for use with an arched dam.

Spillway Coefficients. The ordinary type of spillway, often called the "ogee type," from the shape of the downstream face, has coefficients of discharge in the usual Weir formula, $Q = CLH^{\frac{3}{2}}$, approximately, as shown on page 83, for different crest heads.[1]

Flashboards. The purpose of using flashboards on the spillway of a dam is to permit holding water level at the dam above the crest level for pondage purposes during seasons of ordinary and low-water flow. They

[1] For experimental values of C with different types of spillways, see *Water Supply Paper* 200, U.S. Geological Survey; also King's "Handbook of Hydraulics."

are arranged so that the water will go out or be removed at times of flood so that full spillway discharge capacity may then be maintained.

In the simplest form, they consist of boards or planks supplemented by steel or wrought-iron pins or pipes, to which they are usually wired so as to remain in place as water level drops and varies. The flashboard pins are usually proportioned so that, if water flows over the top of the boards to some predetermined amount, the pins will bend over and release the boards, which have to be renewed later when the river stage permits.

Head on Crest, Ft	C
1	3.3
2	3.4
3	3.4
4	3.5
5	3.7
10	3.9
15	4.0

Collapsible Flashboards. Collapsible hinged flashboards are also in use, built of steel in sections, hinged at the base so that they can be lowered to the crest of the spillway.

Stanchions. The stanchion or deck type of flashboard requires piers with bridge spans, the boards being held by vertical stanchions or piers, pivoted near the top and so arranged that they may be swung in a horizontal position, thus clearing the channel between the piers and allowing the boards to go out.

Pin and Board Type. The cost of pin and board type flashboards at two installations was as follows:

Plant	Location	Height, ft	Length, ft	Area, sq ft	Cost per sq ft
McIndoes 1930	Conn. River, Vt., N. H.	6	300	1,800	$2.05 5.75 with cableway
15 Mile Falls . . . 1928	Conn. River, Vt., N. H.	6	306	1,836	$3.10 5.45 with cableway

The cost of stanchion-type flashboards 18 ft high by 100 ft long was $3.50 per square foot at the McIndoes plant; it was $6.50 per square foot including cableway and handling equipment and $11.40 per square foot including spillway bridge, as well.

Spillway Gates

As an adjunct to spillway discharge, it is often desirable to install some type of spillway or crest gate. These, located between piers, give an oppor-

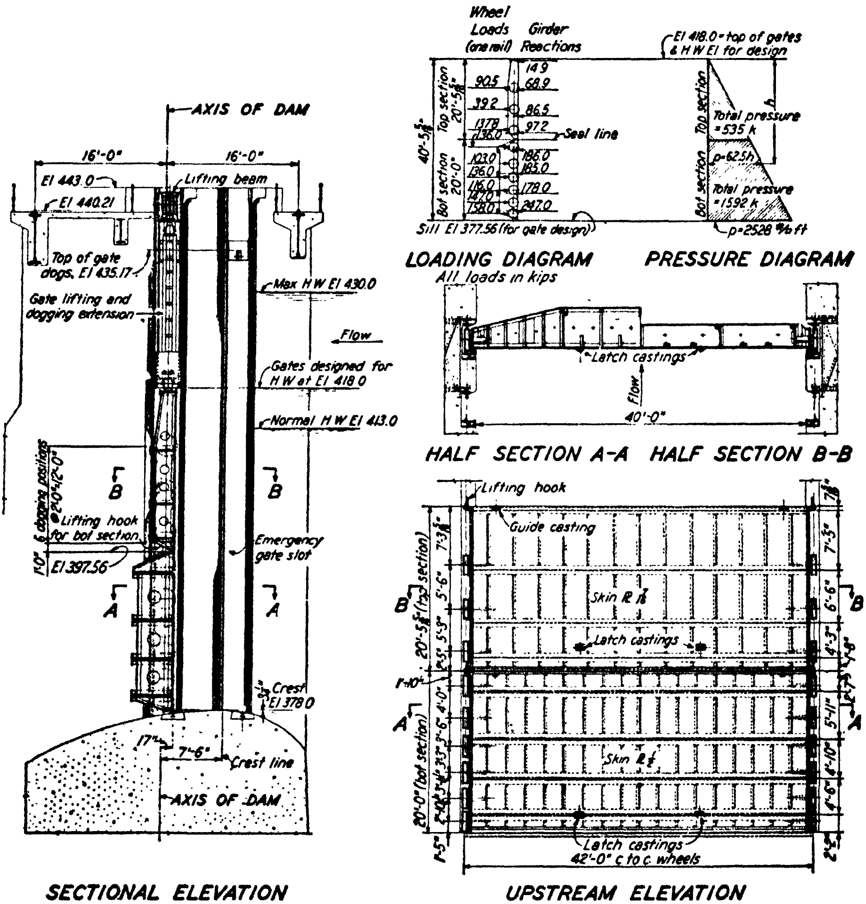


FIG. 6·1. Spillway gate, Pickwick Landing Dam.

tunity to utilize more discharge head than that afforded by the spillway. Such gates may be of the *vertical-lift type*, of timber, steel, or steel and concrete, with gate control by individual hoists or a traveling crane.

Where these gates are of large size, they may be built in sections, as shown in Fig. 6·1, which gives details of the vertical-lift gates at the Pickwick Landing development of the TVA, which are the same kind as those used at Guntersville and Chickamauga projects. These gates have a water area of 1,600 sq ft, 40 ft high by 40 ft wide. In addition to sections and an upstream elevation, pressure and loading diagrams are also shown. The cost of these gates in place was as follows:

Plant	No. of gates	Total area, sq ft	Frame and arch	Gates and seats	Cranes, bridge, etc.	Total cost	Cost per sq ft
Pickwick, 1935–38	22	35,200	$240,942	$573,951	$388,475	$1,203,368	$34.00
Guntersville, 1935–40	18	28,800	230,615	485,206	336,874	1,052,695	36.80
Chickamauga, 1936–41	18	28,800	268,379	425,021	263,220	956,620	33.20
						Mean	$34.70

Tainter Gates. Tainter, or radial, gates are often used for crest gates. In this type of gate, the pressure of the water causes no resistance to the

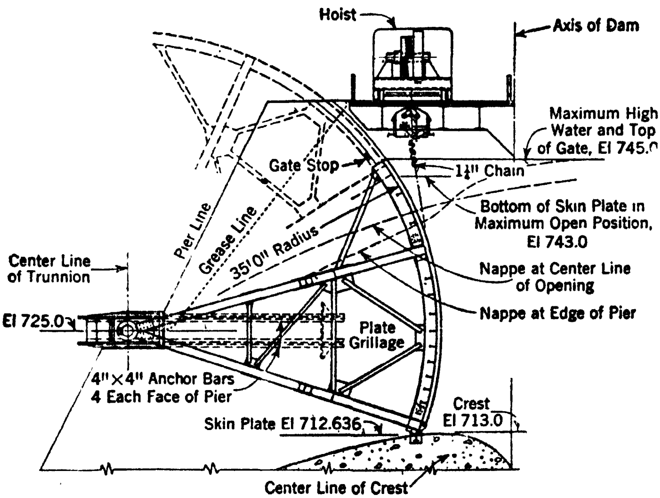


FIG. 6·2. Radial gate. Watts Bar Dam.

opening or closing, except the slight friction of the trunnion shaft bearings and of the sealing strips at the sides. Hence only the weight of the gates has to be controlled by the operating mechanism; and, in closing, the weight is ordinarily sufficient to overcome the slight friction, and little or no mechanical effort is required. Ice difficulties may be overcome by the use of warmed compressed air piped along the base of the gate and released at suitable intervals. In Fig. 6·2 is a sectional view of the 32-ft-high by 40-ft-wide radial gates at the Watts Bar project of the TVA as used at this and four other projects.

Data upon the size and cost of radial gates at eight of the projects of the

TVA, furnished through their courtesy, are given in Table 6·1. These show some variation in cost index [column (1)] and also in weight per square foot of gate area [column (4)/column (3)]. The Wheeler and Ocoee No. 3 gates are relatively light and the Hiwassee heavy. Apparently a normal weight is about 160 psf consistent with numbers 3, 4, 5, and 6. In the case of Wheeler, the large number of gates probably lowers the cost somewhat.

The cost data in Table 6·1 include first cost of materials delivered to the various projects and costs of assembling, erecting, and painting. All costs include a pro rata share of general construction facility expense but do not include design, management, and other overheads.

The Tainter-gate installation at the spillway of the McIndoes plant of the New England Power Association on the Connecticut River near Barnet, Vt., included three steel gates each 24 ft wide by 25 ft high and each equipped with a 30-ton motor-operated hoist. The cost (1930) was $36,671, or about $20 per square foot.

Broome Gates. Tractor, or Broome, gates are used for both spillway crest and power intake gates. They are cable-supported and motor-operated, set upon a slight batter and wedged into final position.

At the turbine intakes of the Norris Dam (1933–1937) of the TVA are installed two gates of this type for the two penstocks (see Fig. 6·3). They cover an opening 16.5 ft wide by 28.5 ft high and have tractor-type continuous roller trains with floating wedges for seating and unseating wedge movement, vertical 18 in., horizontal ⅜ in., gate clearance or movement to seat $\frac{3}{16}$ in. The invert of the penstock is at El. 853.45 and the base of the hoist at El. 1,045.90. The hoist is double drum 64 in. in diameter, with a continuous cable over an equalizing sheave. The gate and hoist weigh 162 tons, the lifted parts 121 tons. The two gates with operating equipment cost about $212,000 or $225 per square foot of water area.

These gates are designed to operate normally under a maximum hydrostatic pressure of 167 ft but are capable of prompt and safe closure under an emergency condition with an unbalanced pressure of 80 psi against the leaves and a flow of 8,500 cfs through each intake.

The Broome gate is a common form of tractor gate in general use for head gates. Some installations and costs are as follows:

Location	Installation	Dimensions	Cost per sq ft
Bristol, N. H.	Utilities Power Co.	3 ft 10 in. by 10 ft.	$ 43
Sherman Island, N. Y.	Hudson River	13 ft 10 in. by 13 ft	18
Remmel, Ark.	Arkansas Power and Light Co.	6 ft 11 in. by 13 ft	44
Denison Dam, Tex.	Red River (1940)	16 ft 9 in. by 19 ft	174

Table 6·1 Radial Gates—TVA Projects—Cost

Project, date, and ENR construction cost index*	Gates: No.	Gates: Size	Gates: Area, sq ft	Notes	Total weight, tons	Cost per ton	Cost per sq ft	Operating equipment
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1. Wheeler, June, 1935, 197	60	15 by 40	36,000	A	1,487	$ 197	$ 8.13	Individual motors and chain hoist
				B	736	285	5.81	
				C	2,223	$ 226	$13.94	
2. Hiwassee, August, 1939, 235	7	23 by 32	5,152	A	279	$ 326	$17.68	Gasoline power hoist and 120-ton multi-purpose crane
				B	340	396	26.15	
				C	619	$ 371	$43.83	
3. Watts Bar, July, 1940, 242	20	32 by 40	25,600	A	1,666	$ 222	$14.42	Two 92-ton deck-type traveling hoists
				B	236	646	5.96	
				C	1,902	$ 274	$20.38	
4. Cherokee, December, 1940, 249	9	32 by 40	11,520	A	868	$ 270	$20.34	Two deck-type traveling hoists
				B	94	882	7.20	
				C	962	$ 330	$27.54	
5. Douglas, February, 1942, 269	11	32 by 40	14,080	A	1,061	$ 356	$26.86	Two deck-type traveling hoists
				B	94	1,309	8.76	
				C	1,155	$ 434	$35.62	
6. Apalachia, October, 1941, 264	10	23 by 32	7,360	A	355	$ 486	$23.42	Individual motors and chain hoists
				B	81	1,266	13.88	
				C	436	$ 630	$37.30	
7. Ocoee No. 3, October, 1941, 264	7	23 by 32	5,152	A	254	$ 428	$21.13	Individual motors and chain hoists
				B	56	1,219	13.26	
				C	310	$ 571	$34.39	
8. Fort Loudoun, July, 1941, 258	14	32 by 40	17,920	A	1,415	$ 275	$21.69	Two deck-type traveling hoists
				B	160	1,005	9.00	
				C	1,575	$ 349	$30.69	

* See p. 96.

NOTES: A—Gates.
B—Operating equipment.
C—Total of gates and operating equipment.

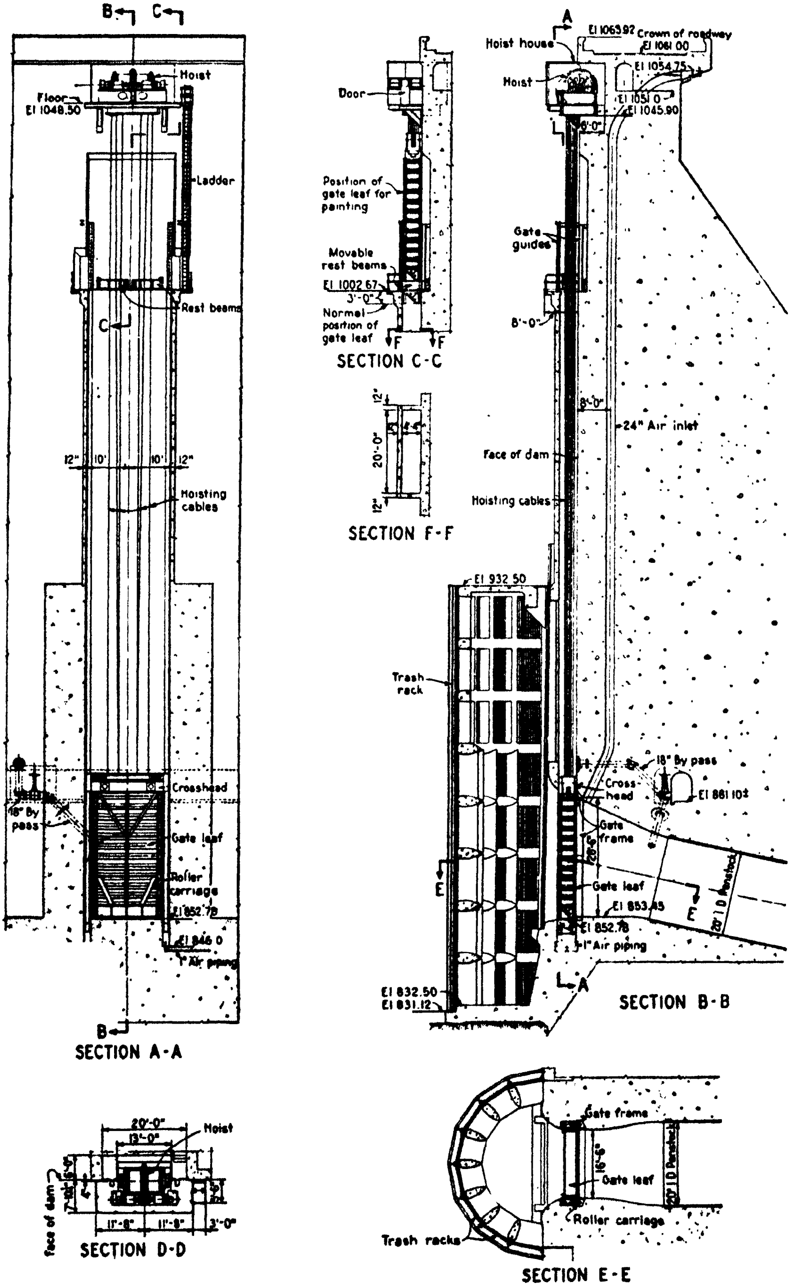


FIG. 6·3. Tractor gate, Norris Dam.

These are all at low-pressure power intakes except at Denison, which is under a head of about 80 to 100 ft.

Drum Gates. The drum gate is an automatic crest gate operated by water pressure in chambers. Gates of this type were installed at the Norris Dam of the TVA[1] where they are the principal control of reservoir level at flood stages. They also provide 517,000 acre-ft of flood storage between spillway crest level (El. 1,020) and the top of the gates in a raised position (see Fig. 6·4).

The drum-gate installation includes three units, each 100 ft long and 14 ft high when raised to a closed position. Each unit consists of a structural-steel buoyant drum in the shape of a modified section of a cylinder of about 15 ft radius, hinged along the upstream side of the dam; a chamber for the drum, recessed in the concrete spillway crest; and a piping system with valves, controls, and indicators for regulating the water level within the chamber and thereby the position of the gate. In the open or down position, the drum forms a part of the spillway crest. Forty-three cast-steel hinges support the rolled-bronze hinge pins, each casting anchored by bolts to the concrete. An end hinge pin, keyed to the drum and extended through the wall, connects with the gate-control mechanism.

The arrangement and details of this gate are shown in Fig. 6·3. It is operated by water in the chambers. Buoyancy of the metal drum causes it to rise or lower with changes of water level in the well, and the gate is designed to provide regulation of gate crest at any desired position between El. 1,020 and 1,034. Drum gates are also installed on the spillways at Boulder Dam (see page 328).

The cost of these gates (1935) was as follows:

3 gates	$132,356
Operating equipment	32,142
	$164,498

The cost per square foot (4,200 sq ft) was $39.

Cylinder Roller Gates. Cylinder roller gates are in use at the Bellows Falls, Vt., plant of the New England Power Association. These are two in number made of steel cylinders each 13½ ft in diameter and 121 ft long over all. To the cylinders are attached steel aprons provided with oak sealing strips, which when the gates are closed bear upon steel sills, making a height of gate of 18 ft. They are operated by a motor-driven hoist of 192 tons capacity which rolls the gate up an inclined plane, by a chain connection at one end. The cost of these gates (1927–1928) was about $42 per square foot.

In Fig. 6·5 is shown a downstream view of the dam at the Clarks Falls

[1] "The Norris Project," TVA, 1937, pp. 88–92.

power development on the Lamoille River at Milton, Vt. (drainage area about 700 sq miles). Flashboards are out and Tainter gates raised to clear pond-water level above the dam.

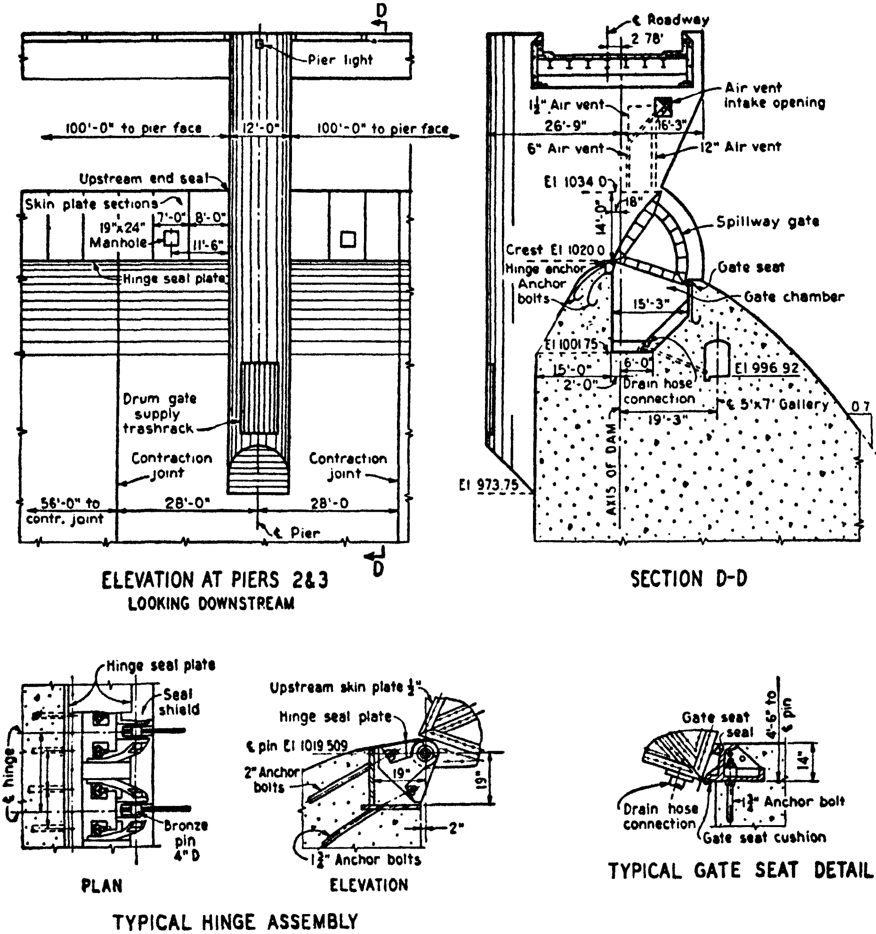


FIG. 6·4. Drum gate, Norris Dam.

Beginning at the left bank, the dam includes:

1. A low earth dike 170 ft long, top at El. 298.
2. An open 100-ft spillway at El. 288.
3. A bay of stanchion flashboards 42.5 ft long, top at El. 288, sill at El. 267.
4. Three Tainter gates with 6-ft piers, each gate 24 ft wide by 23.5 ft high with sill at El. 267.

5. Three bays of stanchion flashboards each 42.5 ft long, top at El. 288, sills at El. 269.5.
6. A 23-ft intake section with racks and intake for a 12-ft penstock, running along the right bank of the river about 400 ft to a powerhouse, with a single 4,000-hp unit under 41 ft head.

At El. 288, "full pond," the water area is about 350 acres and total storage about 4,600 acre-ft. This is an effective combination of crest gates and flashboards for this site.

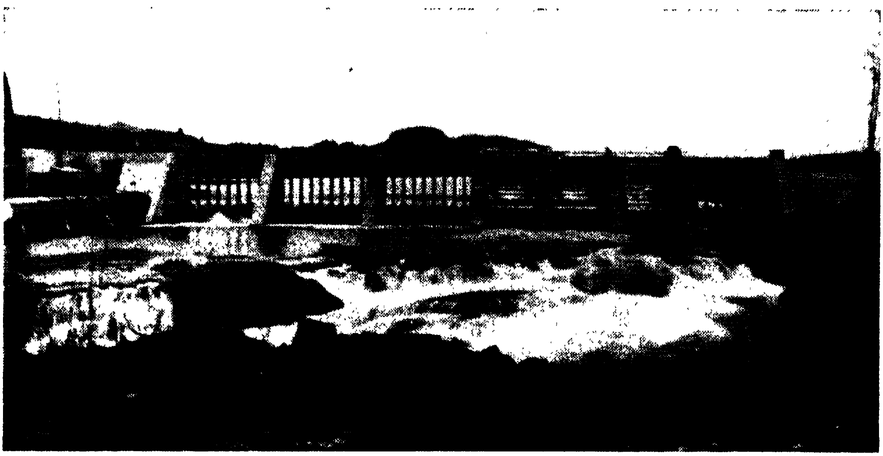


FIG. 6·5. Tainter gates and stanchion flashboards, Clarks Falls Dam.

Spillway Gates and Equipment on TVA Projects.[1] The system of dams of the TVA (see Chap. 13) has provided an opportunity for a review of designs and a standardization of spillway gates and equipment, with resulting economies. These have now been in operation for some time and afford some practical observations as to operation.

Conditions at these large reservoirs are generally favorable for the use of such equipment, since they are all cleared and ice conditions are of little importance. Because the necessary degree of regulation can be obtained with slow-speed equipment, the primary objectives have been strength, simplicity, and economy.

The last objective has been further obtained by duplicating equipment wherever possible. Thus the spillway gates at Pickwick, Guntersville, and Chickamauga Dams are of the same basic design as those at Hiwassee, Apalachia, and Ocoee No. 3. In other groups, intake gates and sluice gates are duplications that have been useful in emergencies and in transfer of equipment in cases of necessity.

[1] ROBERTS, KENNETH C.: *Civil Eng.*, January, 1946, pp. 13–16.

The elevation of the gate sill and the over-all length of the spillway are based upon the maximum design flood, headwater, and tailwater relations at different flows, stream-bed conditions, and space requirements of lock, powerhouse, and other elements. The top of the gate is fixed at maximum reservoir level. These considerations have led to the use of gates from 32 to 50 ft in height.

After a study of various types of gates, a decision was made to use either vertical-lift gates or Tainter gates. The latter were found to have an advantage in economy and simplicity of construction where conditions permitted a normal position of gate trunnions. A cost comparison also indicated that, for gates of the required heights, maximum economy is realized where the gate width approximately equals the gate height. Also, with these proportions, loads upon the operating elements and equipment are not excessive and permit design to be effected with readily available commercial parts. A riveted gate frame with skin plates attached by welding was used; and, with present improvements, welding will no doubt be more generally used.

Vertical-lift Gates. These have been installed as follows at various river projects (see Fig. 6·1):

Kentucky	50 ft high by 40 ft wide
Pickwick Guntersville Chickamauga	40 ft high by 40 ft wide

A gate of these proportions creates problems of erection and alignment at the job site, since it is impossible to obtain shipment of the completely assembled unit by ordinary means. For such a gate, good practice requires shop assembly of the complete gate after fabrication, dismantling for shipment, and erection and alignment of wheels in the field. With the number of gates required for each dam varying from 18 at Chickamauga to 24 at Kentucky, it may be readily understood that this field erection could be an operation of considerable magnitude. An additional and important consideration is the relatively high capacity and physical proportions of the gate-handling equipment.

To meet these objections, the gates are subdivided into sections not exceeding 20 ft in height. Each section is a self-contained unit complete with accessories, without couplings of any kind to its matching section, and is handled as a separate unit at all times. These sections were completely assembled in the shop and were shipped, by barge, ready for immediate installation in the gate slots. The only field work on the gates was the painting.

Discharge of water at moderate flows occurs through openings between the top and bottom sections of the gates. An important consideration with this type of discharge is the provision of adequate means of venting the space below the nappe at the downstream face of the bottom gate section in order to prevent subnormal pressures and the consequent overloading of the gate. For discharges greater than can be accommodated by the removal of the top sections, bottom sections in sufficient number are raised above the nappe.

The gates are supported and run on tracks located in slots in the piers. Two sets of identical slots are provided—the downstream, or service, slots in which the gates normally function, and the upstream, or emergency, slots in which a spare gate may be operated to effect a closure for maintenance purposes or for the repair of damage in case of an accident to the service gate in the downstream slot. Gates are stored, with clearance over the discharge nappe, in the slots at deck level when not in use. All gates are interchangeable and may be used in any slot, either upstream or downstream.

No single element of the gate is of more importance to successful operation than the wheels. Of the two common types, the one bronze-bushed and the other mounted on antifriction bearings, the former is somewhat less expensive, and considerations of economy dictate its use where its relatively high frictional resistance permits. Generally the wheels are mounted on axles cantilevered from the gate body. This mounting eliminates all machining on the gate body except the boring of the axle holes and permits the use of relatively small and shallow gate slots, avoiding excessive weakening of the piers. The wheels also serve as the guiding element in the slots and as support brackets when the gate is dogged in any of its raised positions.

Radial Gates. This type of gate has been installed at a group of projects including Hiwassee, Apalachia, and Ocoee No. 3 Dams, and at a second group comprising Watts Bar, Fort Loudoun, Cherokee, and Douglas Dams. Those in the first group are 23 ft high by 32 ft wide (see Fig. 6·2); those in the second group are 32 ft high and 40 ft wide. Within these groups, the gates for all projects are identical. A third design, with provisions to accommodate discharge over the top of the gate, is planned for Fontana Dam.

The practice of shipping gates fully assembled in the shop, previously followed and found advantageous at other main-river projects, was substantially repeated at Watts Bar Dam. The blade of the gate (Fig. 6·2), including skin plate and supporting beams, the transverse trusses and bracing, and accessories such as guide wheels and seals, was shipped by barge completely assembled and painted. The rocker arms, with bearings assembled, were shipped separately. At other dams, however, the only practicable access was by rail, and consequently the gates were shipped knocked down into subassemblies as few and as large as possible.

The gates conform to conventional design in respect to structure, seals, and accessories. Gate anchorages for the several groups differ greatly because of the variation in loading. Those generally used with gates of the first group consist of a through trunnion supported in a plate cylinder embedded in the pier, which is reinforced to resist the gate thrust. Those for gates of the other groups are weldments of heavy plate-and-bar sections, which project beyond the pier and support the trunnion bearings. For better distribution of shearing stresses over the pier, the anchorage bars were wrapped to prevent bond with the concrete and to deliver their load to a grillage located a sufficient distance upstream to ensure low stresses within the pier.

Cranes and Hoists. Handling methods and equipment for the gates become an essential and integral consideration in establishing the main features of a dam. In general, the equipment is of the mobile type, traveling on a track on the operating deck of the dam, and is usually installed in duplicate, not only to provide adequate handling facilities for periods of great activity, but also to ensure the availability of at least one unit in serviceable condition at all times.

Separate hoisting units for each gate unit, as occasionally installed elsewhere, were given consideration; but, since the number of gates comprising the spillway of most of the TVA's projects varies up to 24 in number, the economic comparison favors the mobile equipment. Furthermore, the adoption of the sectional-gate design for those projects equipped with vertical-lift gates practically precludes the use of fixed hoists. Another factor influencing this choice was the possibility of the use of the same mobile equipment not only for handling spillway gates but also for other services.

The exceptions to the use of this general type of equipment occur at Apalachia, Ocoee No. 3, and Fontana Dams. Apalachia and Ocoee No. 3 Dams are at a considerable distance from their powerhouses, and separate fixed hoists are installed for each of the radial gates of the spillway to permit the ready application of remote-control features, which in the future will undoubtedly be found desirable. At Fontana Dam, there are four radial gates arranged in pairs, each pair operated by a common fixed hoist.

The *sectionalized vertical-lift gates* present a rather unique problem in the development of methods and devices for handling the bottom gate sections whose tops are at an elevation approximately 20 ft or more below pool level, and which must be raised and lowered through the discharge jet flowing over the gate section. The equipment chosen for this service comprises two gantry cranes with lifting beams, lifting extensions, and gate-dogging devices. The lifting extensions serve two purposes: when attached to the control gates, they support the gates in the various regulating positions; and, when attached to the lifting beams, they may be lowered through the discharge jet to engage and lift the submerged bottom gate sections. The gate dogs provide a retractable latching seat, engaging either the tread of the gate wheel or the steps of the lifting extensions, for support of the gate in the regulating or storage position.

The design of load brakes received much consideration. Brakes combining certain features of mechanical and regenerative electrical braking were adopted.

Kentucky Dam. About 20 miles above the confluence of the Tennessee and Ohio Rivers, this dam is the latest and largest of the main-river developments. In its design, the practices previously developed were carried over insofar as possible, but the increased scale of the work demanded independent consideration of many basic features. The sectionalized-gate design was continued with the important differences that the over-all height of the gate was increased from 40 to 50 ft and the gates were subdivided into three sections rather than two. These changes increased the depth of the discharge jet, through which the bottom gate sections must be raised and lowered, from 20 ft to 31 ft 6 in. Gate-lifting loads also increased, primarily because of the increased frictional resistances of the additional water load on the gates. Furthermore, the need for close control of regulation of flow and its distribution became of distinctly secondary importance, since the relatively great depth of tail water in conjunction with the design of the apron removes the danger of serious erosion.

Radial Gates Simpler to Operate. In comparison with the handling equipment for the sectional vertical-lift gates, that for the radial gates of Watts Bar Dam and others of the group is relatively simple with respect to both type of equipment and operating procedures. The gates are suspended at all times—except when resting on the sill in the fully closed position—on the hoist chains from a chain deflector and dogging unit recessed into the spillway deck; they are raised or lowered by means of a hoist traveling on the spillway deck, to which the lifting chains are coupled manually whenever necessary.

The traveling gate hoist comprises two grooved chain drums driven from a single motor through worm-and-spur gearing, the whole mounted on a four-wheel truck. Since close speed control is not necessary, the hoist motors are of the squirrel-cage type. Lowering control is provided by the regenerative braking action of the motor. The fixed hoists installed at Apalachia and Ocoee No. 3 Dams are somewhat similar in design, differing mainly in the omission of the truck and in the use of pocketed chain sprockets instead of grooved chain drums. Also, since remote control of these hoists is contemplated, slack chain switches are provided to prevent overtravel of the hoist in the lowering direction if the motion of the gate should be obstructed.

Unit Costs of Dams and Reservoirs

In using cost data gathered during the period of the last three decades, it is essential to relate them to a cost index. For construction work such as reservoir dams, the so-called "Engineering News Record Cost Index," based upon the year 1913 as 100 per cent, is in common use.

By years, this cost index has been as follows for the period from 1921 to 1946:

Yearly Average Cost Index—Engineering News Record—1913 Base

1921	202	1926	208	1931	181	1936	206	1941	258
1922	175	1927	201	1932	157	1937	235	1942	276
1923	214	1928	207	1933	170	1938	236	1943	290
1924	215	1929	207	1934	198	1939	236	1944	299
1925	207	1930	203	1935	196	1940	242	1945	308
								1946	346

The present cost index (August, 1947) is about 415 per cent, and it has been above 250 since 1941.

Costs during the period 1921 to 1939, inclusive, may be considered as *prewar costs* and taken as approximately 200 per cent of 1913 costs. All the data used here come within this category (except as noted), and costs are therefore approximately "prewar normal."

Handling Water. This item includes the cost of handling the river flow during the period of construction, including all temporary flumes or waterways, cofferdams, pumping, etc. The unit found convenient is the cost per square mile of drainage area, as shown in the following table:

Project	Drainage area, sq miles	Cost of handling water	
		Total	Per sq mile drainage area
Pickwick	32,820	$631,000	$19
Wheeler	29,600	799,000	27
Guntersville	24,450	755,000	31
Chickamauga	20,790	1,020,000	51
Norris	2,912	421,000	145
McIndoes	2,200	20,400	9
15 Mile Falls	1,600	448,000	280
Total	114,372	$4,094,600	
Mean	16,300	585,000	$36

The above data are averaged in order of size of drainage area and show a progressive increase ratio of cost per square mile against size of drainage area. Conditions were difficult at both 15 Mile Falls and Norris.

Concrete. The weighted cost of concrete of all classes as used for seven reservoir projects was as follows:

Project	Time	Cu yds (all classes)	Cost	
			Total	Per cu yd
Norris	1933–1937	936,290	$5,350,000	$5.70
Wheeler	1933–1936	548,960	5,747,000	10.50
Pickwick	1935–1938	236,290	2,860,000	12.00
Chickamauga	1936–1941	164,600	1,819,000	11.00
15 Mile Falls	1928–1930	119,340	1,140,000	9.60
Guntersville	1936–1940	84,570	1,187,000	14.00
McIndoes	1930	8,860	103,000	11.70
Total	......	2,098,910	$18,206,000	
Mean	......	300,000	2,600,000	$8.70

The cost per cubic yard of concrete of the different classes used in reservoir dams will vary between about $6 and $20 or more depending upon its class and amount and place in the structure as well as construction conditions and particularly the amount of mass concrete. Thus ordinary class subdivisions would include mass concrete, surface concrete, walls, piers, bridges, etc., with more or less steel reinforcement in all but the mass concrete. In the foregoing table, the projects are arranged in order of total yardage, including all classes of concrete, and show a fairly definite variation trend in cost per cubic yard with the total amount of concrete.

Based upon this table, the average prewar (200 per cent cost index) cost of concrete per cubic yard for reservoir dams is about as follows:

100,000 cu yd or less	$12
500,000 cu yd	10
1,000,000 cu yd or more	7

Earth Embankment. The cost of earth embankment, as in earth dams, furnished, placed, and compacted was as follows for the 12 projects listed at the top of page 98.

Because improvement in equipment and methods of handling and compacting during the last decade have lessened the cost of earth embankment, it is not consistent with the cost index. Prewar costs ranged from $0.30 to $0.80 per cubic yard depending upon the yardage and working conditions.

Project	Location	Cu yd	Date	Cost per cu yd
15 Mile Falls	Vt., N. H.	367,000	1928	$0.85
Chickamauga	TVA	2,200,000	1933–1939	0.30
Guntersville	TVA	618,000	1933–1939	0.37
Pickwick	TVA	1,700,000	1933–1939	0.15 (hydraulic fill)
Norris	TVA	152,000	1933–1939	0.70
Somerset	Vt.	800,000	1913	0.50 (semihydraulic fill)
Davis Bridge	Vt.	1,500,000	1923	1.00 (semihydraulic fill)

Miami Conservancy, Ohio (1918-1930) All Hydraulic Fill

Project	Cu yd	Cost per cu yd
Germantown	796,000	$0.85
Englewood	3,401,000	0.73
Lockington	795,000	0.72
Taylorsville	908,000	0.74
Huffman	1,350,000	0.72

Other Costs. In the following table are given approximate *unit* costs for certain other items commonly occurring in reservoir dams.

Item	Cost per unit		
	Average	Max.	Min.
Earth excavation, cu yd	$1.00	$2.00	$0.50
Rock excavation, cu yd	4.00	6.00	3.00
Reinforcing steel, lb	0.04	0.06	0.03
Clearing reservoir site, acre	80.00	100.00	50.00

Gates. The costs of different types of crest and other gates and flashboards have been given previously in some detail. Their cost per square foot of water area may be summarized approximately as follows:

Cost of Gates and Flashboards per Square Foot of Water Area

Flashboards:

Pin and board type	$ 3
Same with cableway	5
Stanchion type	6
Same with bridge	11

Gates (with hoists):

Low-pressure gates	30– 60
High-pressure gates	40–200

Crest gates:

Vertical lift	30
Tainter gates	20– 35
Drum and cylinder types	40

Spillway Discharge with Filling Reservoir

If the water level in a reservoir is at or below spillway level and a flood occurs with any outlet gates closed, the reservoir will fill to spillway level if it was initially below that level and will then begin to run over the spillway. As the water level over the spillway increases, water is stored in the reservoir and also flows over the spillway in an increasing amount. Eventually a maximum depth on the spillway will be reached, inflow will just equal outflow, and water will be stored in the reservoir to the depth on the spillway plus the depth of any reservoir drawdown at the beginning of flood flow. This, of course, lessens the flood peak over that which would occur without storage regulation because of water held back in this reservoir storage, and it is an important feature of reservoir effect upon floods.

The problem of determining the amount and time of the maximum depth reached upon the spillway is conveniently solved by the "mass-curve method," as follows:[1] Referring to Fig. 6·6 and considering initial water level at spillway level, *AB* is a mass curve of inflow expressed as inches depth upon the drainage area. The combined mass curve of water stored and water discharged over the spillway is shown by the connected vertical and sloping lines. Each vertical line represents for a given portion of depth on the spillway the corresponding increase in storage, in inches depth; each sloping line shows the rate of discharge over the spillway in inches depth for various successive depths of water upon the spillway. These slopes of spillway discharge for convenience are shown in the auxiliary diagram at the left of Fig. 6·6. Thus, starting with water at spillway level, $ab = 0.35$ in. is the volume of water between 0 and 1 ft depth on the spillway, and *bc* is the average rate of flow over the spillway during this 1-ft increment of depth. Point *c* is the intersection with the mass curve of inflow or, in other words, the point on this curve where the sum of storage plus outflow over the spillway for the first increment of depth of 1 ft is just equal to inflow to the reservoir. This is at about 1.2 days on the time scale and a depth on the spillway of 1.00 ft.

Similarly, *cd* is the next increment of storage for the spillway depth

[1] For details and further discussion of this method, see H. K. Barrows, "Water Power Engineering," 3d ed., pp. 199–201, McGraw-Hill Book Company, Inc., New York, 1943.

interval between 1.0 and 2.0 ft, and *e* is the intersection of the spillway flow rate running from *d* and corresponding to the average cumulative flow over the spillway for the 1- to 2-ft depth interval, shown upon the auxiliary diagram at the left in Fig. 6·6.

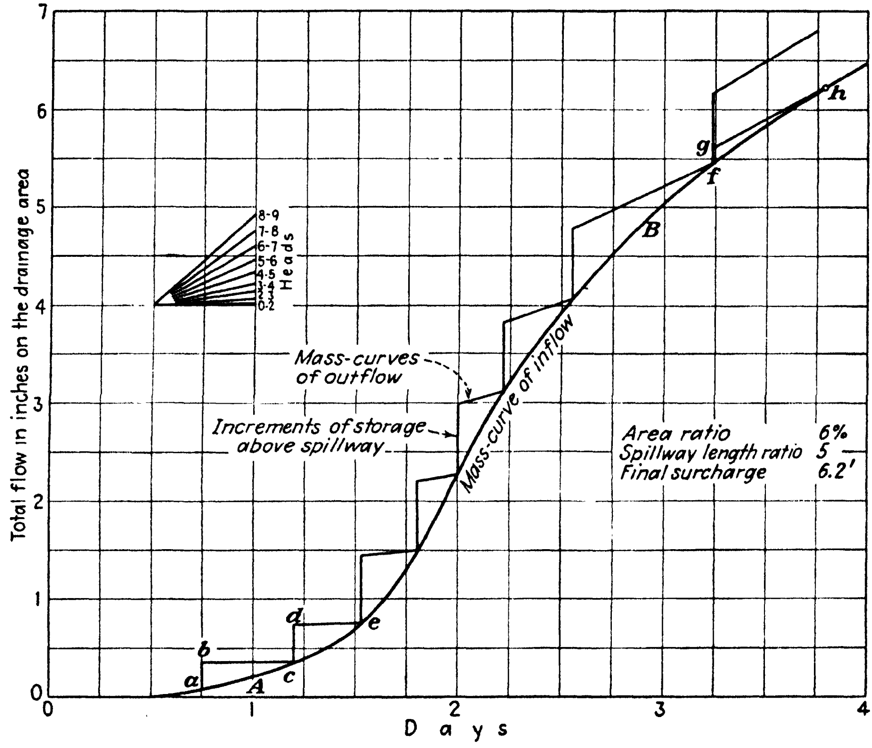


FIG. 6·6. Spillway discharge, mass-curve method.

Proceeding in this manner, it is found that, considering a depth over the spillway between 6.0 and 6.2 ft with storage increment *fg*, the sloping cumulative flow over the spillway line running from *g* just comes tangent to the mass curve of inflow at *h*, at which point the slope of the mass curve of storage plus spillway flow is the same as that of the mass inflow curve *AB;* or, in other words, inflow to the reservoir is just equal to outflow plus storage. This is the maximum outflow, and thereafter reservoir flow will lessen and water level will lower. Maximum flow over the spillway will be about 1.15 in. per day.

In Fig. 6·6, the assumed drainage area is 100 sq miles, spillway 50 ft long, and reservoir area 3,840 acres. In studying reservoir outflow and storage, it is convenient to determine:

1. The spillway "length ratio," or the ratio of the spillway length in feet to the drainage area in square miles—in the case of Fig. 6·6, this is 50/100 = 0.5.

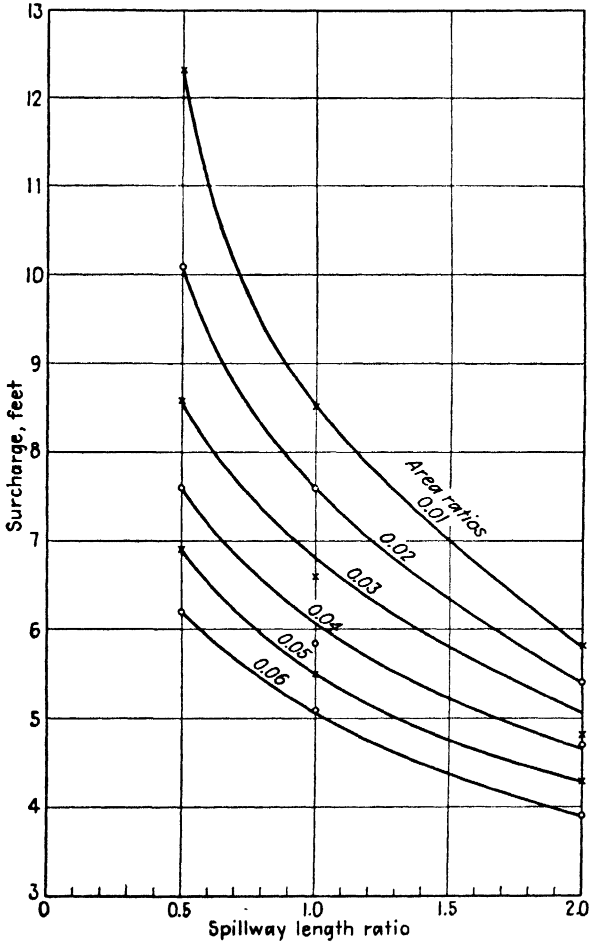


FIG. 6·7. Area and spillway length ratios/surcharge.

2. The "area ratio" of the reservoir, or the percentage of the reservoir area as regards the total drainage area, each being expressed in either square miles or acres. In Fig. 6·6, this is 3,840/(100 × 640) 100 = 6.0 per cent.

These factors were suggested by Lt. Joseph L. Johnson in a thesis study under the direction of the author at Massachusetts Institute of Technology in 1938, entitled "The Effect of Power Reservoirs upon Flood Control," as a means of general study of the reservoir outflow–storage problems.

For a given curve of reservoir inflow, by expressing spillway length and reservoir area by these ratios and obtaining final reservoir surcharge and corresponding spillway flow for a range of values for these ratios, a resultant

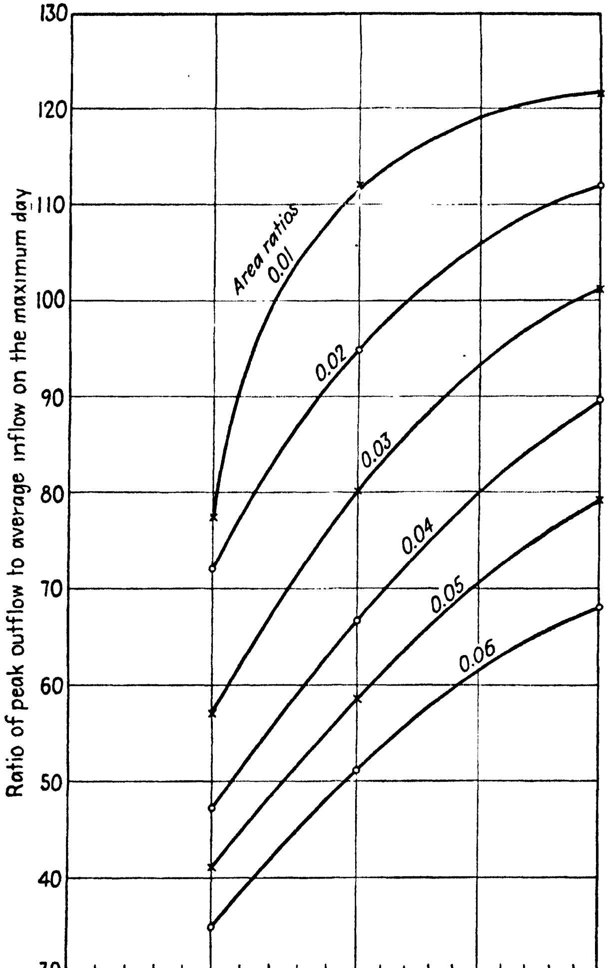


Fig. 6·8. Peak ÷ average daily flow/area and spillway length ratios.

diagram may be obtained as shown in Fig. 6·7, permitting a quick solution of the problem.

Another set of curves as in Fig. 6·8 may be constructed showing, for different spillway length ratios and area ratios, the percentage of maximum regulated peak outflow to maximum reservoir inflow, or the percentage of regulation due to storage above spillway level.

Use of Gates for Reservoir Drawdown. When a reservoir is filled at the beginning of a flood-producing storm, additional storage capacity may be obtained by the use of spillway gates to draw down the reservoir and thus provide more storage for the coming flood.

The mass-curve method is useful in this problem, as will be seen from Fig. 6·9. In this case, it is assumed that gates are opened at *a* and that the rate and mass curve of their discharge are shown by *ae*, the gates being

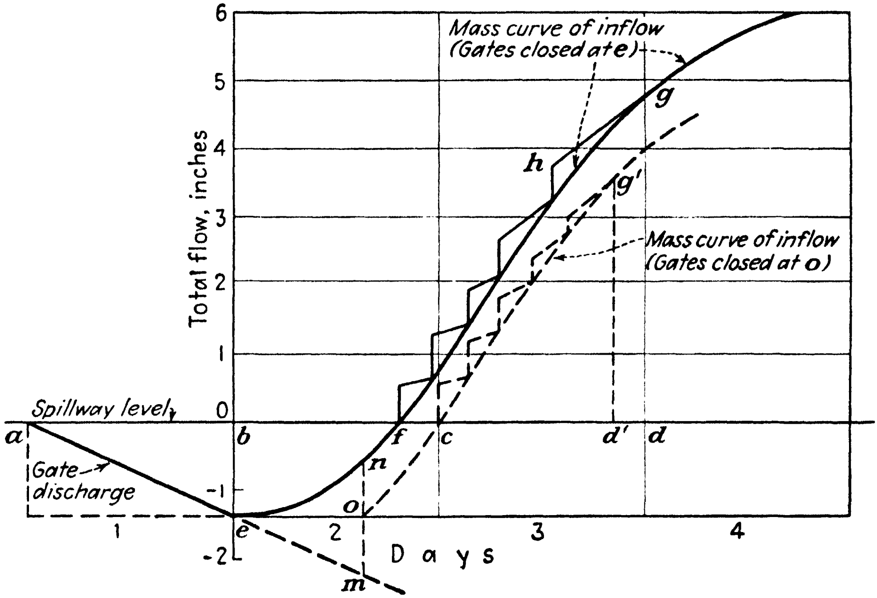


FIG. 6·9. Spillway mass-curve method with gate discharge.

closed at *e*. The storage is depleted by *be*, and the mass curve of inflow begins at *e*. The solution, as previously described, results in *gh* as the tangent to the mass curve of inflow, maximum storage above spillway level of *gd* (and hence maximum depth reached on the spillway), and rate of maximum discharge corresponding to the slope of *gh*.

The gates would not necessarily be shut at *e;* and, if they are kept open until their capacity is reached in respect to inflow, the line *ac* is extended to *m* or to where the slope of the mass curve of inflow equals that of the gate discharge (or *no* = *om*), and the reservoir level is the same at *o* as at *c*. If the gates are now closed, the mass curve of inflow will be drawn at *ocg′* approximately parallel to *ng* and will reach a maximum flow such as *g′d′* instead of *gd*, for the previous case where gates never closed at *e*.

Other assumptions as to the time of gate closure and the rate of gate discharge may be studied by the method here outlined.

CHAPTER 7

NOTABLE FLOODS IN THE UNITED STATES

The more important essentials as to storms, rainfall, runoff, flood characteristics, and resulting damage for 14 floods in various parts of the United States during the past 40 years are arranged chronologically in this chapter. In this compilation are included many of the more noteworthy floods, as well as others of lesser magnitude, which are representative of typical occurrences. Some floods caused by dam failures are also included.

In general, where information is available, each flood is described with reference to location, date, storm causing flood, precipitation, runoff, damage, and references.

Comments are also made where notable or unusual occurrences of flow, damages, or other circumstances are found. In a few cases, some details of storm movements and characteristics are also given.

1. Heppner, Oreg., June 14, 1903

The little town of Heppner, with a population of about 1,400, is located about 45 miles south of the Columbia River in northern Oregon on Willow Creek, which flows northward to the Columbia.

The flood of Sunday evening, June 14, 1903, was the result of a so-called "cloudburst" in which an unknown amount of rain fell probably over less than 20 sq miles of area, lasting only about ½ hr but causing a very sudden rise of the creek.

Ninety per cent of the water came from Balm Creek, which enters Willow Creek about a mile above the town. The peak discharge of Balm Creek was later determined from high-water marks, plainly visible, to be about 36,000 cfs, or approximately 1,800 csm, based upon two measured cross sections, each about 1,700 ft in area with a width of about 260 ft and a hydraulic slope of about 130 ft per mile. The total flood runoff at Heppner was estimated at about 1,100 acre-ft, or only about 1 in. of depth on 20 sq miles. The rainfall was therefore probably about 1½ in. or at a rate of 3 in. per hr.

This sudden flood wave due to very steep stream slopes (that of Willow Creek being about 38 ft per mile) and sudden heavy rainfall evidently overwhelmed the town through which the creek ran, and about one-third of it was destroyed, with a loss of life of about 200 people and damages estimated at $250,000.

The flood plane at Heppner was about 5 ft above the general road level and about 500 ft wide, with a maximum depth at the creek of 15 to 20 ft. The great loss of life was due to the manner of construction of the houses, which rested upon posts or upon stone foundations and were floated away by the flood and crushed against trees or other objects, so that the swift current and floating masses of timber prevented persons from getting to shore.

It is obvious from this occurrence that, even upon small streams in arid and semiarid regions subject to such sudden "cloudburst" floods, ample waterways should be provided under bridges and through towns, and all buildings likely to be swept away should be out of reach of the flood.

References

Water Supply Paper 96, U.S. Geological Survey.
"Destructive Floods in the U.S. in 1903," pp. 9–12.
Eng. News, July 16, 1903, p. 53.

2. Bridgeport, Conn., July 29–30, 1905

Bridgeport is a manufacturing city with a population of about 147,000, located on Long Island Sound in southern Connecticut at the mouth of a small stream, Pequonnock River, with about 25 sq miles of drainage area, upon which occurred the so-called "Bridgeport flood" of 1905. The storm was of local character, intense over a comparatively small area, and of the thunderstorm type. Precipitation as measured at Bridgeport was as follows:

Date	Time	Precipitation	
		In. total	Mean per hr, in.
July 29	11:40 A.M.–1:30 P.M.	0.10	0.06
	1:30 P.M.–4:15 P.M.	5.90	2.15
	4:15 P.M.–7:50 P.M.	4.18	1.17
	7:50 P.M.– 12 M.	0.77	0.18
July 30	3:00 A.M.–5:20 A.M.	0.37	0.16
Total, July 29	11:40 A.M.– 12 M.	10.95	0.89
Total, July 29–30	11:40 A.M.–5:20 A.M.	11.32	0.64

The rainfall of 8 to 11 in. in about 17 hr on a rather quick spilling basin soon overtaxed the river channel, resulting in much debris and the progressive failure of four dams, each one adding to the magnitude of the flood wave and its accompanying debris.

The dams that failed included the following:

1. Toucey Dam on a brook entering the Pequonnock gave way about midnight of July 29. It was 100 ft long, 10 ft high, built of rubble-cement masonry.
2. Wards Mill Dam at Trumbull failed by sliding when the flood wave from Toucey Dam struck it. It was 60 ft long, 15 ft high, of rubble-cement masonry, on ledge-rock foundation.
3. About 1 A.M. July 30, Bunnell Pond Dam, 1½ miles above Bridgeport, failed. It was 800 ft long, 28 ft high, of earth with masonry spillway, and failed from overtopping due in part to debris blocking the spillway.
4. The Berkshire Mill Dam at tidewater, 140 ft long and 7 ft high, failed, probably from undermining.

No exact data of runoff are available, but from rough estimates it was of the order of 250 csm. The principal damage was the loss of the dams, and several bridges and shipping at the mouth of the river were damaged and traffic impeded. Damage totaled about $250,000.

This flood upon a small drainage area is indicative of what may happen almost anywhere sooner or later in many portions of the United States. Although the rainfall was excessive for a period of about ¾ day, damage was largely caused by successive dam failures rather than storm runoff.

The existence of dams with inadequate provision to handle large floods has been given considerable attention since the time of the Bridgeport flood. Many states have made provision for state supervision with reference to the safety and adequacy of new dams, and to some extent possible danger from existing structures has also been considered. There is need, however, for further attention in this field and a better standardization of requirements for safety.

References

Water Supply Paper 162, U.S. Geological Survey.
"Destructive Floods in the U.S. in 1905," pp. 1–3.

3. Ohio River, Mar. 23-27, 1913

This flood covered the entire Ohio Valley but was especially severe in the northern tributaries, the Miami, Scioto, and Muskingum Rivers. Two storm centers passed over the valley in succession so that the rain from both blended. The first developed on the morning of Mar. 22 over Nevada and moved eastward during the next 24 hr, passing over Utah and Colorado. Thence it moved northeastward and, on the morning of Mar. 24, centered just north of the Great Lakes region, thereafter passing down the St. Lawrence Valley. The second storm formed on Mar. 24 over the southwest and, by Mar. 25, had developed into an elongated trough of low pressure extending from Texas to New England, through the Ohio Valley. This trough was held on both sides by high-pressure areas—one off the Atlantic coast and the other in the region of the Great Lakes. The condition was not relieved till Mar. 28, when the HIGH over the Atlantic gave way and allowed the storm to pass off to the northeastward from New England.

Table 7·1 Ohio River Basin—Discharge in March, 1913

River and point	Drainage area, sq miles	Max. discharge	
		cfs	Cfs per sq mile
Ohio River:			
Pomeroy, Ohio	39,950	633,000	15.8
Cincinnati, Ohio	75,800	660,000	8.7
Evansville, Ind.	107,000	900,000	8.4
Metropolis, Ill.	203,100	1,400,000	6.9
Muskingum River Basin:			
Muskingum River, Coshocton, Ohio	4,847	202,000	42
Muskingum River, McConnelsville, Ohio	7,411	270,000	36
Tuscarawas River, Dover, Ohio	1,398	62,000	44
Mohican River, Greer, Ohio	942	55,000	58
Kokising River, Milwood, Ohio	472	40,000	85
Scioto River Basin:			
Scioto River, Dublin, Ohio	988	74,500	75
Scioto River, Columbus, Ohio	1,624	138,000	85
Scioto River, Chillicothe, Ohio	3,847	260,000	67
Miami River Basin:			
Miami River, Sidney, Ohio	545	44,000	81
Miami River, Dayton, Ohio	2,513	250,000	100
Miami River, Hamilton, Ohio	3,639	352,000	97
Stillwater River, Englewood, Ohio	646	85,400	132
Mad River, Dayton, Ohio	632	75,700	120
Twin Creek, Germantown, Ohio	275	66,000	240
Kentucky River, Frankfort, Ky.	5,400	80,000	14.8
Wabash River, Terre Haute, Ind.	12,200	200,000	16.4
Tennessee River, Knoxville, Tenn.	8,990	121,000	13.5
Tennessee River, Chattanooga, Tenn.	21,400	202,000	9.4

Temperatures were not low immediately before, during, or after the storm, and there was no snow or ice stored in any part of the Ohio Basin.

Rainfall for the period Mar. 23–27, 1913, is shown below for selected stations:

Station	Rainfall, In.
Toledo, Ohio	6.14
Columbus, Ohio	6.94
Cincinnati, Ohio	7.47
Dayton, Ohio	9.00
Marion, Ohio	10.70
Bellefontaine, Ohio	11.10
Indianapolis, Ind.	6.01
Madison, Ind.	9.04
Cairo, Ill.	4.60
Louisville, Ky.	5.97
Nashville, Tenn.	2.97
St. Louis, Mo.	5.85
Pittsburgh, Pa.	3.50
Parkersburg, W. Va.	3.01

Area-rainfall relations for the period Mar. 23–27, 1913, were as follows:

Rainfall, In.	Area, Sq Miles
10	2,600
8	19,200
6	73,600

The lowest rainfall for the entire Ohio Basin of 203,000 sq miles was about 2 in. during this period, Mar. 23–27. This occurred in the southern part of the basin, the Tennessee Valley. The maximum of 10 or 11 in. occurred in the headwaters of the Miami, Scioto, and Muskingum Rivers in the north-central part of the basin.

Maximum discharge at various points in the Ohio Basin is given in Table 7·1.

Runoff was an unusually high percentage of rainfall in this flood, as shown by the following data:

Item	Miami River at Dayton 2,513 sq miles	Scioto River at Columbus Storage Dam 1,032 sq miles
Rainfall, in.	9.6	9.34
Runoff, in.	9.3	8.68
Ratio	0.97	0.93

On the main Ohio River, at Cincinnati, the runoff Mar. 24-Apr. 13, inclusive, was 4.4 in. over the drainage area of 75,800 sq. miles.

Estimates of damage in the Miami River Basin were based on 120 replies received from circular letters sent to officials of 200 cities and towns of 5,000 population or over. Dayton, Ohio, reported $100,000,000 and Hamilton, Ohio, $15,000,000 loss. These were the largest single items of damage. The total damage reported, including highways and railroads, was $180,873,000. There is little doubt that the actual loss in the Ohio River Basin exceeded $200,000,000; and 415 lives were reported lost. Upon the Muskingum River Basin, direct damages totaled nearly $14,000,000.

References

Water Supply Paper 334, U.S. Geological Survey.
Water Supply Paper 838, pp. 458-480, U.S. Geological Survey.
Water Supply Paper 771, U.S. Geological Survey.
Miami Conservancy District Tech. Repl., Part VII, p. 31.

4. Arkansas River, Colorado, June 3-4, 1921

This flood was due to very heavy rains or "cloudbursts" in the foothill region of the Arkansas River Valley over a relatively small area, but with runoff accentuated by the steep slopes of mountainous country. Precipitation at certain observation stations was as follows:

Rainfall—June 2-6, 1921

Station	Elevation	Precipitation for 48 hr ending June 4, P.M.
Canon City	5,343	2.65
Florence	5,187	4.11
Pueblo	4,685	3.09
Colorado Springs	6,098	4.09
Victor	10,100	2.11
Fremont	8,850	5.14

No Weather Bureau stations exist in the region of heavy rainfall, which extended from a point north of Colorado Springs to the Wet Mountains on the south, and from Canon City on the west to a point just west and north of Pueblo on the east. Within this region, in small areas, the rainfall was intense, varying from 4 to 12 in. in different localities (according to the best information available) during the night of June 3–4.

There were no gauging stations on the stream within the area of intense rainfall, and thus the maximum discharge had to be determined by the slope method after the flood had passed. The main channels were uniformly free from vegetation and the overflow area small, so that n was taken as 0.035 for the tributary streams and 0.030 for the Arkansas River above Pueblo.

Following are data for certain of these measurement points:

Maximum Discharge of Streams Near Pueblo from Area of Intense Rainfall

Stream	Distance above Pueblo	Drainage area, sq miles	Discharge, cfs per sq mile	Time of crest, June 3
8 Mile Creek	33.5	65.0	134	8.30 P.M.
Brush Hollow Creek	31.0	22.0	243	8.00 P.M.
Rush Creek	16.8	19.6	238	5.30 P.M.
Cameron Creek	12.0	7.3	1,900	9.00 P.M.
Boggs Creek	8.2	26.5	582	6.30 P.M.
Dry Creek	1.8	86	283	11.00 P.M.
Fountain Creek	−1.5	93.2	36	3.00 A.M.

Rate of Progress of Flood Crest. This is shown in the following table, which shows a lessening rate as the flood crest traveled down the river, because of decrease in river slope above Pueblo and chiefly because of valley storage below Pueblo.

Progress of Flood Crest through Arkansas Valley

Point of observation	Distance along river between points, miles	Approximate time of crest	Rate of progress mph	Crest flow, sec-ft	Duration of crest flow
Canon City	.....	8:00 P.M. June 3	....	3,740	
Florence	8	8:30 P.M. June 3	16.0	9,000	
Pueblo	35	Midnight June 3	10.0	103,000	6 min.
Manzanola	52	9:30 A.M. June 4	5.5		
Rocky Ford	10	11:30 A.M. June 4	5.0		
La Junta	13	3:00 A.M. June 4	3.7	200,000	45 min
Fort Lyon	28	11:00 P.M. June 4	3.5		
Head of Amity Canal	23	4:30 A.M. June 5	4.2	170,000	3½ hr
Lamar	7	7:30 A.M. June 5	2.1		
Holly	30	8:30 P.M. June 5	2.3	120,000	

The entire Arkansas Valley from Florence, 30 miles west of Pueblo, to the state line, was severely affected, and the loss of life and property was heavy. The greatest damage was done in Pueblo, the chief city. Below Pueblo, the loss was chiefly agricultural, with more than 57,000 acres flooded, of which 4,700 acres were destroyed for agricultural use.

The total estimated flood damages were placed at about $19,000,000, and in Pueblo over 500 dwellings were washed from their foundations. Loss of life in Pueblo was heavy, estimated at 100 or more. Great damage was done to railroad terminals in Pueblo. Total property loss in the city of Pueblo alone was estimated at about $10,000,000.

At Pueblo prior to 1921, the maximum recorded discharge was about 39,000 cfs on May 31, 1894. The maximum on June 3, 1921, was about 103,000 cfs, but with a comparatively small total discharge due to very rapid rise and fall.

Reference

Water Supply Paper 487, U.S. Geological Survey.

5. San Antonio, Tex., 1921

The city of San Antonio is located in south-central Texas in the westerly portion of the Rio Grande Plain and just below the escarpment of the Edwards Plateau, on the west. From the northerly limits of the city, the country rises to the northwest about 12 miles to the elevation of the plateau, 700 ft above its general level. On the sloping margin of the plateau are three comparatively small drainage areas, the streams of which traverse the city and unite near its southern limits. All these streams were in severe flood on the night of Sept. 9, 1921.

The rainstorm that caused the flood was typical in that it broke on the margin of the Edwards Plateau. A drought of several months' duration was broken on the night of Thursday, Sept. 8, 1921, with a considerable rainfall during the night and following day but with little runoff. From 6 to 9 P.M. on Friday, Sept. 9, a severe electrical storm occurred in the region north of the city, with intense rainfall, which continued with reduced severity until 11.30 P.M. The total rainfall for the entire storm from 6 P.M. Thursday to midnight on Friday, a period of 30 hr, exceeded 20 in. in the northern part of the drainage area and averaged about 12½ in. for the entire basin.

The San Antonio River rises from fissure springs near the northern city line and winds through the city with a well-defined flood plain from ½ to ¾ miles in width. The ordinary flow from these springs is from 50 to 150 cfs; the river channel within the city limits is about 12 miles long, with a fall of about 90 ft. The greater part of the business district, some of the industries, and the older and more thickly settled residential districts lie in the immediate river valley.

The largest of the three drainage areas tributary to San Antonio River, totaling 32.4 sq miles, lies north of the city, drained by Olmos Creek, which is ordinarily dry. Within the city, 2 miles north of its center, San Pedro Creek rises from springs, with an ordinary flow of only about 10 cfs. This creek is augmented by flow from Alazan and Apache creeks (ordinarily dry arroyos), giving it a total drainage area of 44.2 sq miles where it empties into San Antonio River near the southern limits of the city. The lower San Pedro traverses the western part of the business district and with its two tributary creeks runs through some of the cheaper and more thickly settled residential districts. The total drainage area of the San Antonio River and San Pedro Creek at their confluence is about 90 sq miles.

Olmos Creek overflowed its banks and swept into the city about 10 P.M. Friday, reaching its crest within an hour, overflowing its banks immediately north of the business district and inundating a large part of this, and reaching the crest about 2 A.M. The water reached depths as great as 8 to 12 ft in the business streets. In the northerly district, the river rose from 2 to 8 ft above bridge floors, with a great mass of debris caused by the piers and railings. In this part of the city, the rush of water in the meantime had inundated residential and industrial areas. Hundreds of homes were flooded to depths from a few inches to 8 and 10 ft, and some frame dwellings were washed away. A scum of heavy fuel oil from some of the industries coated everything it touched and left clear high-water marks. In the business district, all the basements and most of the first floors were flooded from a small amount to 12 ft.

Alazan Creek, with a comparatively steep grade from the high ground to the northwest of the city, swept suddenly through the thickly settled low areas along its banks, playing havoc with bridges and destroying many frame homes. It was in this section that the principal loss of life occurred, and to a lesser extent on Apache Creek and upon the thickly settled area below the confluence of these two creeks. This loss of life was estimated at 75 and an aggregate property damage of between 5 and 10 million dollars, or about $77,000 per square mile of drainage area of 90 sq miles, a very high amount. The greatest damage was to merchandise in flooded basements and first floors and to furniture and interior finish of homes from the muddy, oily waters. Pavements were destroyed, particularly wooden-block pavement, which simply floated off. Substantial buildings were in general damaged little structurally, although there was a heavy loss of plate glass broken by water pressure and drift. Several concrete floors were burst upward by water pressure before the basements were flooded from the top.

Estimates of peak discharges were made by the slope-area method, as follows:

Stream	Drainage area, sq miles	Peak discharge	
		Cfs	Cfs per sq mile
Olmos	32.4	31,000	956
San Antonio	41	23,700	578
San Antonio	45	16,000	356
Alazan-Martinez	16.9	28,000	1,656
Apache	22	19,000	864

San Antonio had been subject to floods at intervals of 8 to 15 years, with damaging floods in 1913 and 1914, after which the city employed engineers to report upon methods of flood prevention. They reported in December, 1920, and foretold the probability of a flood like that which occurred in 1921.

Reference

Trans. A.S.C.E., Vol. 85, pp. 354–377, 1922.

6. Vermont and New England, 1927

This flood occurred chiefly in Vermont and western New Hampshire and Massachusetts. The storm was of the Atlantic coastal type, of tropical origin, appearing with LOW center in Florida on Nov. 2 at 8 A.M., reaching Cape Hatteras Nov. 3, centering in Vermont Nov. 4 and north of Quebec Nov. 5. A secondary storm center caused excessive rainfall in Rhode Island and as far north as Worcester, Mass.

Three factors joined in producing an extraordinary rainfall in New England.[1]

1. A large supply of warm damp air following the low-pressure, coastal path.
2. A large and relatively immovable mass of cold high-pressure air on the Atlantic east of the flooded area and extending into Maine, Nova Scotia, and Newfoundland.
3. A southerly flow of a considerable body of cold air into northern New England and to the Atlantic Coast.

For 3 days, the northward flow of tropical air was funneled into western New England between the stationary air masses on the east and west, forcing masses of damp warm air together and up over the mountains of western New England, as well as over their northern and western bulwarks of cold air—at the same time being overturned by the cold air above, resulting in an unusual rainfall within a period of 36 hr.

The greatest recorded precipitation occurred in Vermont—with a maximum of 9.65 in. at Somerset, and probably 11 or 12 in. in the higher

[1] WEBER and BROOKS: *Jour. N.E.W.W. Assoc.*, March, 1928, pp. 91–103.

portions of the Green Mountains. The secondary storm in Rhode Island showed nearly as great a maximum—9.37 in. at Westerly.

Rainfall-area relations were approximately as follows:

Rainfall, In.	Area, Sq Miles
5	22,000
6	8,500
7	4,000
8	1,800
9	500

Because of relatively high rainfall during October and the mountainous and hilly character of the region affected—with steep land and stream

Table 7·2 Flood Flows—Nov. 3-4, 1927, in New England

River and point	Drainage area, sq miles	Max. discharge	
		Cfs	Csm
Vermont:			
Winooski:			
Essex Junction	1,010	110,000	109
Montpelier	397	57,000	144
Jail Branch	38	11,500	303
Dog	67	17,200	257
White, West Hartford	695	120,000	172
New Hampshire:			
Ammonoosuc, Bethlehem	97	17,900	185
Peabody:			
Glen House	17.4	7,330	421
Gorham	40	9,920	248
Wild Cat Brook, Jackson	28	14,800	528
Bakers, Wentworth	52	15,000	288
Massachusetts:			
Deerfield, Charlemont	180	36,000	200
Cold:			
Hoosac Tunnel	22.4	6,870	307
Charlemont	32.2	7,760	241

Connecticut River	Drainage area, sq miles	Max discharge		Runoff, in., Nov. 3–10
		Cfs	Csm	
South Newbury	2,750	78,000	28	3.6
White River Junction	4,040	138,000	34	4.1
Vernon	6,220	155,000	25	3.5
Sunderland	7,740	165,000	21.3	3.3

slopes—runoff was generally very high, and high velocities with much scour of river bed and banks occurred, with attendant deposits on adjacent farm lands. Some of the data of excessive runoff are given in Table 7·2.[1]

In importance, this flood in New England approximated the more frequent flood disasters upon the Mississippi and Ohio rivers and was of the order of a national calamity. Eighty-seven lives were lost, and damages totaling $40,000,000 resulted, of which nearly three-fourths was in the state of Vermont alone and nearly one-half upon Winooski River in that state.

This flood stimulated interest in flood control, which was initiated in the state of Vermont through the Advisory Committee of Engineers on Flood Control, who made a comprehensive investigation during 1928 to 1930 and planned a complete power-reservoir system for the state with flood control as a by-product, which has since in part been carried out.

References

Water Supply Paper 636–C, U.S. Geological Survey.

Report of the Committee on Floods, *Jour. Boston Soc. Civil Eng.*, September, 1930.

Reports of Advisory Committee of Engineers on Flood Control, State of Vermont, 1928 and 1930.

7. La Cañada Valley, Calif., 1934

La Cañada Valley is near Los Angeles, Calif., through which runs Verdugo Creek, fed by intermittent canyon streams. The storm was of tropical or semitropical origin and advanced northward or northeastward, crossing the coast line of Los Angeles, Orange, and the upper extremity of San Diego counties. Before the moist air reached the slopes of the San Gabriel and San Bernadino Mountains, it was underrun by a cold easterly wind, causing a great increase in precipitation.

Rainfall began Dec. 30, 1933, and continued for about 3 days, ending Jan. 1, 1934. Records of total storm rainfall for various stations are as follows:

Station	Elevation	Rainfall, in.
Hegees Camp, San Gabriel Mountains	2,650	19.9
Opids Camp, San Gabriel Mountains	4,250	17.9
Squirrel Inn, San Bernardino Mountains	5,700	12.5
Lytle Creek, Lytle Creek Valley	2,250	13.4
Malibu Headquarters, Santa Monica Mountains	750	16.0
Mount Wilson	5,850	15.6
Big Bear Lake Dam, San Bernadino Mountains	6,800	10.3

The elevation of La Cañada Valley varies from 1,200 to 2,100 ft.

[1] See *Water Supply Paper* 636–C, pp. 73–79, U.S. Geological Survey, for more complete data.

Such gauging stations as there were in this area were destroyed in the flood, and flows therefore had to be estimated. Estimates based on rainfall rates gave 645 cfs per sq mile in Pickens Canyon. Using Kutter's formula. an estimate of 320 cfs per sq mile was obtained for Verdugo Creek near the lower end of La Cañada Valley, where the drainage area is 19 sq miles.

In the canyons feeding Verdugo Creek, the floods were of only a few minutes' duration and were in the form of a wall of water, sometimes as much as 20 ft in height. Except in flood time, flow in the canyons is mostly underground or through poorly defined surface channels.

It is estimated that, as a result of this flood, over 600,000 cu yd of debris were moved from the mountain area to the foothill region. Citrus groves, vineyards, villages, and highways were damaged to an amount of about $5,000,000, and more than 40 lives were lost.

Reference

Water Supply Paper 796–C, U.S. Geological Survey.

8. New York State, 1935

This flood occurred chiefly in the upper Susquehanna basin, particularly on Chenango and Tioughnioga Rivers, owing to a succession of heavy thunderstorms. The weather conditions follow in detail, because they are of interest in connection with this type of storm.

July 3 (8 P.M.):

1. Well-defined LOW moving eastward over Hudson Bay region.
2. Ill-defined, slow-moving LOW over Rocky Mountain region and Great Plains district.
3. Polar air mass (P_c) beginning to move southward over northern Canada.

July 4 (8 P.M.):

1. LOW moved east-southeast to lower St. Lawrence Valley.
2. LOW more definite, central over South Dakota.
3. P_c air had overspread Hudson Bay region.

July 5 (8 P.M.):

1. LOW over Gulf of St. Lawrence.
2. LOW over Minnesota and extreme northwest Ontario, moving northeastward.
3. P_c air wedge over James Bay and northern Ontario and blocking the LOW of 2.

2. LOW *blocked* and pushed southeast.
3. P_c front at northern New England and New York.

July 7 (8 A.M.):

2. LOW center near Buffalo, N.Y.

3. P_c front pushing south and southwest.

July 7 (8 P.M.):

2. LOW drifting slowly southward.
3. P_c front from New Haven, Conn., northwest to eastern Lake Erie.

July 7 and 8:

2. LOW merged with another LOW moving northeast from Georgia, centering at New Jersey coast, resulting in a mass of warm, very moist tropical maritime (T_m) air moving north-northwest over eastern and central New York.
3. Convergence and forced connection of T_m air by cold P_c front in New York State, resulting in heavy and prolonged rainfall. Where T_m air was forced over elevated areas, the heaviest rainfall occurred.

Most of the precipitation that caused the flood occurred in the 24-hr period, beginning in the late afternoon of July 7, and was in the unusual form of a succession of heavy thunderstorms. Thus, at Ithaca there were two thunderstorms on July 6, seven on July 7, and two on July 8, or a total of 11 in three days, 9 in two days, and 7 in one day. This succession of thunderstorms was a noteworthy feature of this flood.

Although rainfall intensity was not exceptional, generally speaking, the total amounts for 1-, 2-, and 3-day periods exceeded all records in the vicinity. Results at certain stations are given in the following table:

Precipitation at Stations in South-central New York, July 7-9, 1935

Station	Length of record, years	24 hr		48 hr		72 hr	
		1935	Previous	1935	Previous	1935	Previous
Ithaca	77	7.90	4.70	9.25	5.88	9.50	5.96
Cortland	57	7.67	5.80	10.58	6.47	11.15	7.82
Delhi	22	8.52	5.71	8.68	6.44	9.43	6.49

Areal distribution of rainfall for the two days July 7–8, 1935, was as follows:

In. Exceeded	Area, Sq Miles
9	1,000
7	3,100
5	8,000
3	18,700

The most intense runoff occurred along an east-west line extending from Hornell on the Canisteo River to Oxford on the Chenango River, or generally along the boundary between the Susquehanna River Basin and the Finger Lakes or Oswego River Basin. The storm was confined to a comparatively narrow strip, which fortunately cut generally across the upper sources of main streams rather than extending along the streams.

Steep land and channel slopes, narrow valleys, lack of absorption by the soils, and the severity of the thunderstorms all resulted in some of the highest unit runoffs yet observed in this country. A few exceptional figures of peak flow are given in the following table.

Maximum Flood Flows in Flood of July, 1935, New York

Stream	No.*	Point	Discharge area, sq miles	Peak flow, scm
Glen Creek	120	Townsend	2.9	2,520
Trumansburg Creek . .	130	Trumansburg	11.5	1,550
Glen Creek	121	Watkins Glen	21.3	1,310
Meads Creek	106	East Campbell	46.1	657
Tunkhannock Creek . .	129	Halseyville	56.7	742

* In *Water Supply Paper* 775-E.

In the Finger Lake district in New York, especially severe damage occurred upon certain small drainage areas. At Hammondsport, at the southern end of Keuka Lake, and the center of the wine industry of the state, total flood damages upon Glen Brook were estimated at $1,264,000 upon a drainage area of only 5 sq miles, or some $252,000 per square mile: at Watkins Glen, at the upper end of Seneca Lake, on Glen Creek (21.3 sq miles), total damages were $2,560,000, or $120,000 per square mile: at Montour Falls, a little south of Watkins Glen, on Shequaga Creek (13 sq miles), total damages were $587,000, or about $45,000 per square mile. These were in all cases largely residential and some business losses.

The total flood damages in the above localities together with that in Ithaca and Syracuse totaled nearly $6,000,000 on about 525 sq miles, or about $11,500 per square mile.

For the entire area essentially covered by the storm in New York and Pennsylvania, about 12,000 sq miles, 40 or more lives were lost, and damages aggregated about $16,700,000, or about $1,400 per square mile.

No systematic records of flood heights in this vicinity were kept prior to 1901. The flood of Mar. 17, 1865, at Binghamton was about 9 in. lower than that of July 8, 1935, according to a crest mark of the earlier flood. The Chenango River, with a stage of 24.8 ft, was the highest during the period of record (1901–1935), but the stage of the Susquehanna River (17.17 ft) had been reached on four previous occasions.

References

Water Supply Paper 773-E, U.S. Geological Survey.
H.R. Doc. 846, 75th Cong., 3d Sess.

9. Texas Floods of 1935

Frequent floods resulting from cloudbursts occur in eastern and central Texas. In 1935, there were several exceptional floods of this type, three of which are selected as important. Others might have been included if more facts about them were available. The floods in these three groups caused a loss of 30 lives and property damage of over $28,500,000, most of which was in the Colorado and Nueces rivers and in the Buffalo Bayou at Houston. The drainage area included was about 57,000 sq miles.

1. **Flood of May 31, 1935.** This occurred on Seco Creek, tributary to the Nueces River, in south-central Texas. The maximum precipitation reported was 22 in. The average was more than 9 in. over the watershed of 153 sq miles above D'Hanis. The duration of the storm was about 3 hr. The maximum discharge near D'Hanis, by the slope-area method, was 230,000 cfs on a discharge area of 153 sq miles, or 1,500 cfs per square mile.

2. **Floods during June, 1935.** These were upon the Llano River, tributary to the Colorado River, and the West Nueces River, tributary to the Nueces River, in south-central Texas. The maximum rainfall recorded, for the days June 9–15, totaled 18 in. General rains in the region for 6 weeks before the flood had brought the streams above normal stage. Approximate rainfall-area relations for June 9–15, 1935, were as follows:

Rainfall, In.	Area, Sq Miles
8	8,500
10	3,300
12	1,300
14	650
16	200

Peak discharge and total runoff are given in Table 7·3.

3. **Flood of December, 1935.** This occurred on Buffalo Bayou, near Houston. From 6 to 20 in. fell Dec. 6–8 on the Buffalo Bayou drainage area of 458 sq miles above Houston.

Discharge

River and point	Drainage area, sq miles	Max. discharge	
		Cfs	Cfs per sq mile
Buffalo Bayou, Houston	319	40,030	125
Buffalo Bayou, Houston	458	52,750	115
Whiteoak Bayou, Houston	85.3	14,750	173
Little Whiteoak Bayou, Houston	18.1	4,950	273

Table 7·3 Texas Flood of June, 1935—Discharge and Runoff

River and point	Drainage area, sq miles	Max. discharge		Flood runoff	
		cfs	cfs per sq mile	Acre-ft	In.
Colorado River, San Saba	18,800*	71,000	3.78	405,700	0.4
Colorado River, Austin	26,350*	181,000	18.3	1,526,000	1.1
Colorado River, Smithville	27,850*	305,000	11.0	1,492,000	1.0
Colorado River, Eagle Lake	29,140*	177,000	6.07	1,378,000	0.9
Pedernales River, Spicewood	1,294	105,000	81.1	180,100	2.6
North Llano River, Junction	914	47,400	51.9	63,290	1.3
Llano River, Junction	1,762	319,000	181	303,800	3.2
Llano River, Castell	3,514	388,000	110	650,300	3.5
South Llano River, Telegraph	540	160,000	296		
Paint Creek, Telegraph	218	69,300	318		
Nueces River, Laguna	764	213,000	279	304,300	7.5
Nueces River, Uvalde	1,930	616,000	319	497,500	4.8
Nueces River, Cotulla	5,260	82,600	15.7	635,300	2.3
Nueces River, Three Rivers	15,600	66,700	4.28	1,501,000	1.8
West Nueces River, Bracketville	402	580,000	1,400		
West Nueces River, Cline	880	536,000	609		
Dry Frio River, Reagan Wells	120	64,700	539		
Atascosa River, Whitsett	1,171	38,300	32.7	205,000	3.3

* 11,800 sq miles deducted for noncontributing area.

The total runoff for the flood at Houston was 12 in. on an area of 319 sq miles.

Reference

Water Supply Paper 796-G, U.S. Geological Survey.

10. Northeastern United States, 1936

These floods were of a general nature between Mar. 12 and 31, 1936, in the Northeastern United States from the James and upper Ohio River basins in Virginia and Pennsylvania to Maine.

Four separate storm centers passed over the area. The first, on Mar. 9 and 10, passed north of Lake Ontario and was accompanied by general rains in western Pennsylvania and New York, and by snow in New England. On Mar. 11 and 12, a Gulf disturbance crossed Virginia, Maryland, Pennsylvania, and New York, causing heavy precipitation over the entire area, with a maximum in the White Mountain area in New Hampshire. From Mar. 18–20, another disturbance originating over the Gulf states passed over Virginia, New Jersey, and Connecticut, up to Quebec, with heavy precipitation all over the area, centering also in the White Mountains. The fourth disturbance crossed the area on Mar. 20, 21, and 22, causing heavy snowfalls in western Pennsylvania and New York, and minor rains elsewhere.

The following table shows the areal extent of different depths of precipitation for the period Mar. 9–22.

Rainfall, In.	Area, Sq Miles
4	168,000
6	66,000
8	19,000
10	4,000
12	1,500
14	800
16	300
18	120

The maximum recorded precipitation for this period Mar. 9–22 was 22 in. at Pinkham Notch, N. H. (El. 2,000).

The additional runoff from snow on the ground was considerable in this flood. The water content at the beginning of the storm was 5 to 10 in. in northern New York, most of Massachusetts, Vermont, New Hampshire, and Maine, and 2 to 4 in. over most of the remainder of the area. Except in headwater areas of the large New England rivers, the snow had all disappeared by Mar. 22.

There were two distinct peaks of runoff in most cases, corresponding to the two periods of intense rainfall Mar. 11, 12 and Mar. 17, 18. Usually

FIG. 7·1. Flood of March, 1936, Comerford Dam and Reservoir. Connecticut River.

FIG. 7·2. Flood of March, 1936, Springfield, Mass., Connecticut River.

the second peak was the higher, since the rivers had not had time to unburden themselves of the first flood.

Several long records of river stage were broken by wide margins. The Connecticut River at Hartford, Conn., with some 300 years of flood knowledge, was 8.6 ft higher than any known flood. The Susquehanna River at Harrisburg, Pa., with a 200-year flood record, was 3.5 ft higher, and the Ohio River at Pittsburgh, Pa., with a record beginning in 1762, was 6.1 ft higher than any other known flood. See Figs. 7·1 to 7·3 for views of this flood and its effect upon the Connecticut River. Table 7·4 gives some of the maximum discharges.

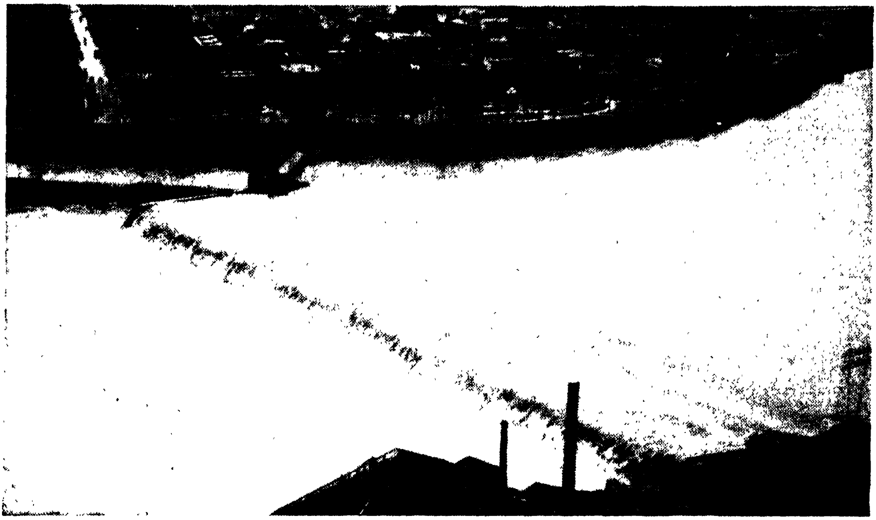


FIG. 7.3 Flood of March, 1936, Holyoke Dam, Connecticut River.

As will be noted, this was distinctly a main-river flood. The smaller drainage areas in general did not have excessive flows. The direct damages, as far as is known, were approximately as follows:

River Basin	Damages
Penobscot	$ 400,000
Kennebec	1,600,000
Androscoggin	4,400,000
Saco	1,600,000
Merrimack	20,100,000
Connecticut	35,000,000
Potomac	2,000,000
Ohio in Pennsylvania	106,000,000
Ohio below Pittsburgh	15,000,000
Total	$186,100,000

Table 7·4 Floods of March, 1936—Discharge and Runoff

River and point	Drainage area, sq miles	Max. discharge		Runoff Mar. 12 31 inclusive, in.
		Cfs	Csm	
Androscoggin River, Brunswick, Me.	3,430	143,000	41.7	9.2
Saco River, Conway, N. H.	386	40,600	105	17.5
Pemigewasset River, Plymouth, N. H.	622	65,400	105	17.0
Merrimack River, Manchester, N. H.	2,854	144,000	50.5	12.1
Souhegan River, Merrimack, N. H.	171	16,900	99	14.8
Connecticut River, Thompsonville, Conn.	9,637	282,000	29.3	10.3
Deerfield River, Charlemont, Mass.	178	32,200	181	16.8*
Mohawk River, Cohoes, N. Y.	3,456	130,000	37.6	9.5†
Delaware River, Trenton, N. J.	6,796	227,000	33.4	9.3
Schuylkill River, Philadelphia, Pa.	1,893	48,400	26	6.1
Susquehanna River, Marietta, Pa.	25,990	787,000	30	8.6
Tioga River, Lindley, N. Y.	770	41,200	53.5	9.4
West Branch Susquehanna River, Renovo, Pa.	2,975	236,000	79	11.5
Juniata River, Newport, Pa.	3,354	215,000	64	9.6
Potomac River, Point of Rocks, Md.	9,651	480,000	49.7	6.7
Cascapon River, Great Cascapon, W. Va.	670	103,000	154	8.4
James River, Richmond, Va.	6,757	158,000	23	4.0
Allegheny River, Parker's Landing, Pa.	7,671	128,000	17	7.5
Youghiogheny River, Sutersville, Pa.	1,715	100,000	58	8.0
Beaver River, Wampum, Pa.	2,235	42,500	19	3.8

* Corrected for storage and based on gross discharge area of 352 sq miles.
† Corrected for storage.

This does not include undetermined loss in the Hudson and Susquehanna river basins and many minor basins. Approximately 150 to 200 lives were lost in the entire area flooded.

References

Water Supply Papers 798, 799, 800, U.S. Geological Survey.
Civil Eng., November, 1937, p. 750.

11. Ohio River, 1937 and Other Floods

The flood on the Ohio River that occurred during January and February, 1937, was of most consequence below Pittsburgh and on the Mississippi River at and below Cairo (see Fig. 7·4). From about Jan. 12 to 25, 1937, there existed in the Eastern United States a peculiar and rare combination of meteorological conditions that produced excessive rainfall over the Ohio River Basin.

By about Jan. 12, a high-pressure area had been built up over the South

Atlantic states, centering over Bermuda. Another high-pressure area centered in the Upper Mississippi Basin. Between these two HIGHS lay a low-pressure trough extending from southwest to northeast over the Ohio Valley. Storm centers, heavily laden with moisture from the Gulf of Mexico, were drawn in a northeasterly direction through this trough,

FIG. 7·4. Flood of February, 1937, Ohio River at Louisville, Ky.

coming in contact with masses of cold air in the Ohio Valley, resulting in moisture condensation and heavy precipitation.

These storms followed each other in rapid succession, with almost daily precipitation, until Jan. 25. The resulting floods in the Ohio River Basin were of note below Pittsburgh and on the Mississippi River at and below Cairo. Table 7·5 shows the recorded precipitation at various stations in the Ohio River Basin for the month of January, grouped in the significant periods of occurrence.

Rainfall-area relations for the period of greatest rainfall, Jan. 20–25, are given below:

Rainfall, In. More Than	Area, Sq Miles
1	200,000
2	171,200
3	138,200
4	102,500
6	50,200
8	29,000
10	16,600
12	4,200

Table 7·5 Ohio River Basin—Rainfall in Inches, January, 1937

Station	Jan. 1–12	Jan. 13–25	Jan. 26–31	Total for January
Ohio River:				
Portsmouth, Ohio	4.03	8.18	0.27	12.48
Maysville, Ky.	4.23	10.08	0.40	14.71
Cincinnati, Ohio	1.78	11.74	0.16	13.68
Madison, Ind.	2.36	15.13	0.34	17.83
Louisville, Ky.	3.70	15.06	0.41	19.17
Owensboro, Ky	3.97	13.32	0.30	17.59
Evansville, Ind.	2.21	12.36	0.21	14.78
Uniontown, Ky.	2.89	14.40	0.10	17.39
Paducah, Ky.	3.76	13.39	0.38	17.53
Kanawha River:				
Charleston, W. Va.	2.06	5.98	0.42	8.46
Muskingum River:				
Coshocton, Ohio	1.52	9.80	0.21	11.53
Scioto River:				
Circleville, Ohio	2.17	10.79	0.32	13.28
Delaware, Ohio	1.13	9.10	0.15	10.38
Kentucky River:				
Frankfort, Ky.	4.13	12.23	0.41	16.71
Wabash River:				
New Harmony, Ind.	2.75	13.35	0.18	16.28
Terre Haute, Ind.	2.83	5.99	0.17	8.99
Green River:				
Calhoun, Ky.	3.82	16.20	0.49	20.51
Bowling Green, Ky.	3.66	15.61	1.43	20.70
Cumberland River:				
Nashville, Tenn.	3.35	10.22	1.18	14.75
Dover, Tenn.	2.15	21.20	0.50	23.85
Tennessee River:				
Johnsonville, Tenn.	3.89	19.23	0.39	23.51
Savannah, Tenn.	6.10	7.54	0.30	13.94

In Table 7·6 are given data of maximum flood discharges and crest elevations for the Ohio River at various points at and below Pittsburgh for the January, 1937, flood, and also, for comparison, other earlier note-

Table 7·6 Ohio River Maximum Discharges and Crest Elevations, 1884-1937

Station	Drainage area, sq miles	February, 1884		March–April, 1913		March, 1936		January–February, 1937	
		Crest el., ft	Discharge, cfs	Crest el., ft	Discharge, cfs	Crest el., ft	Discharge, cfs	Crest el., ft	Discharge, cfs
Pittsburgh, Pa.	19,110	36.5	410,000	33.6	370,000	46.0*	550,000	34.5	320,000
Wheeling, W. Va.	24,800	52.6	430,000	50.6	370,000	55.2*	466,000	48.7	384,000
Parkersburg, W. Va.	38,000	53.9	420,000	58.9*	590,000	48.0	460,000	55.4	530,000
Portsmouth, Ohio	....	66.3	650,000	67.9	670,000	59.2	530,000	74.2*	650,000
Cincinnati, Ohio	76,780	71.1	680,000	69.9	660,000	60.6	570,000	78.8*	894,000
Louisville, Ky.	91,200	46.7	790,000	44.9	770,000	36.5	660,000	57.1*	1,100,000
Evansville, Ind.	107,000	48.0	670,000	48.4	686,000	44.4	640,000	53.75*	1,410,000
Paducah, Ky.	203,000	54.2	1,500,000	54.3	1,500,000	49.15	1,100,000	60.6*	1,780,000

*Denotes highest stage of record.

Table 7·7 Flood of 1937—Maximum Discharge on Ohio and Mississippi Rivers

River and point (1)	Drainage area, sq miles (2)	Max. discharge		In. runoff	
		Cfs (3)	Cfs per sq mile (4)	Jan. 13–19 (5)	Jan. 20–25 (6)
Allegheny River:					
Larabee, Pa.	541	5,430	10.0	2.3	2.3
Red House, N. Y.	1,690	24,400	14.4	2.0	2.7
Franklin, Pa.	5,982	106,000	17.7	2.6	3.1
Parkers Landing, Pa.	7,671	149,000	19.4	2.5	3.3
Ohio River:					
Sewickley, Pa.	19,500	317,000	16.3	1.9	3.0
Wheeling, W. Va.	24,800	384,000	15.5		
Pomeroy, Ohio	39,950	554,000	13.9		
Huntington, W. Va.	55,200	654,000	11.8	2.0	3.4
Cincinnati, Ohio	76,780	894,000	11.6		
Louisville, Ky.	91,200	1,110,000	12.2		
Evansville, Ind.	107,000	1,410,000	13.2	*	7.3
Metropolis, Ill.	203,000	1,850,000	9.1	*	6.6
Mississippi River:					
St. Louis, Mo.	701,000	282,000	0.4		
Cape Girardeau, Mo.	716,000	303,000	0.4		
New Madrid, Mo.	923,200	2,010,000	2.2		
Memphis, Tenn.	932,800	1,980,000	2.1		
Helena, Ark.	941,800	1,970,000	2.1		
Arkansas City, Ark.	1,130,700	2,160,000	1.9		
Vicksburg, Miss.	1,144,500	2,080,000	1.8		
Natchez, Miss.	1,149,400	2,050,000	1.8		

* Included in column (6).

worthy floods. It will be noted that the flood of March, 1936, was highest at and above Wheeling, W. Va.; that of March, 1913, at Parkersburg, W. Va.; and that of January and February, 1937, at and below Portsmouth, Ohio.

Data on peak discharge for the main Ohio and Mississippi River stems are given in Table 7·7. On practically all the tributaries, the 1937 flood was not excessive.

An estimate of flood damages on the Ohio River for the floods of 1936 and 1937 was made by the U.S. Engineer Corps.[1] From these estimates, Table 7·8 was prepared. The flood of 1936, of record in the Pittsburgh district, shows fairly high unit flood damages. That of 1937 shows an increase in these progressively down river. These damage estimates are stated as not including intangible losses.

Table 7·8 Ohio River Flood Damages, 1936-1937

District	Approx. total drainage area	Damages	
		1937	1936
Pittsburgh	25,000	$ 6,800,000	$199,000,000
Huntington	55,000	36,600,000	201,500,000
Cincinnati	90,000	155,600,000	
Louisville	148,000	403,600,000	

References

Floods of Ohio–Mississippi Rivers, January–February, 1937, *Water Supply Paper* 838, U.S. Geological Survey.

The Flood of 1937 in the Ohio Valley, *Civil Eng.*, Vol. 7, No. 5, pp. 326–330, May, 1937.

12. Northern California, 1937

This flood affected practically all the northern two-thirds of the state of California down to and including the Salinas River Basin, and occurred Dec. 9–12, 1937.

Storm. The storm that caused this flood was a well-defined single storm lasting about 48 hr. It formed over the Pacific Ocean and moved into northern California on Dec. 9, covering about two-thirds of the state. Precipitation was in the form of rain rather than snow, up to high altitudes, because of warm temperatures.

[1] Hearings before Committee on Flood Control, H.R. 7393 and 7646, 75th Cong., 1st Sess., pp. 27–28.

Precipitation. The maximum precipitation recorded for the storm period Dec. 9–12 was 20 to 22 in. and for a single day 12 to 14 in. at an elevation of 2,000 to 3,000 ft in the Sacramento River Basin. The maximum for a 2-day period was 18 in. in Feather River Basin. In general, the rainfall was light in the floor of the Great Central Valley, being in the order of 2 to 4 in., increasing roughly as the elevation rose, on either side of the valley, to the order of 16 to 20 in.

Discharge. Data on some of the maximum discharges are given in Table 7·9.

Table 7·9 Flood of Northern California, December, 1937, Discharge

River and point	Drainage area, sq miles	Max. discharge	
		Cfs	Csm
Panjaro River Basin:			
Uvas Creek, Morgan Hill	30.2	8,630	286
Tulare Lake Basin:			
Kaweah River, Three Rivers	520	33,300	64
Kings River, Piedra	1,694	80,000	47
San Joaquin River Basin:			
San Joaquin River, Kerckhoff	1,480	75,000	51
Pitman Creek, Tamarack Creek	22	2,320	105
Merced River Basin:			
Tenaya Creek, Yosemite	47	5,550	118
Tuolumne River Basin:			
Cherry Creek, Hetch Hetchy	111	18,100	163
Mokelumne River Basin:			
Cold Creek, Mokelumne Peak	23	4,100	178
Pit River, Ydalpom	5,350	65,000	12
Feather River Basin, Grizzly Creek, Storrie	6.2	1,570	253
American River, Fair Oaks	1,921	114,000	59
Putah Creek, Guenoc	112	24,100	215
Smith River, Crescent City	613	78,900	129

Damages. Damages to highways, bridges, levees, homes, ranches, transportation and communication facilities, etc., totaled about $15,000,000.

Reference

Water Supply Paper 843, U.S. Geological Survey.

13. "Hurricane Flood" in New England, 1938

Practically all of New England was affected by this flood except the state of Maine and the smaller east-coast streams. It occurred on Sept. 21, 1938, the day of the great hurricane, of that date, in New England (see Fig. 2·6).

From Sept. 12–16, there was moderate rainfall from a storm that came from the southwest, originating over the middle Missouri Valley and passing off to the Northeast. On Sept. 16, a low-pressure area developed over the middle Missouri and Lake Superior region, passing over the Great Lakes and off to the northeast over Labrador on Sept. 20. With a high-pressure area off the coast to the east, a low-pressure trough developed along the Atlantic Coast, with torrential rains resulting on Sept. 19, 20, and 21.

On Sept. 18, a tropical hurricane center was 1,000 miles off Florida. Traveling northeastward, it crossed Long Island at 3 P.M. on Sept. 21, proceeded up the Connecticut Valley at a rate of about 50 mph, passed Burlington, Vt., at 8 P.M., and dissipated in western Quebec during the night of Sept. 21. High wind, in some places up to 100 mph, occurred over most of New England except Maine, inflicting widespread damage to trees and wires. A great wave of water on the shores of Connecticut and Rhode Island caused tremendous damage to the water front.

For the storm period Sept. 17–21, the approximate areal distribution of rainfall was as follows:

Rainfall, In.	Area, Sq Miles
4	63,000
6	38,000
8	21,000
10	8,300
12	2,600
14	800
16	100

For rainfalls of 8 in. and over, this storm exceeded all records in the Northeastern United States in respect to areas covered. The maximum of 16 in. on 100 sq miles is especially noteworthy, as well as the maximum station records of 17 in. at Barre, Mass., and Hartford, Conn.

Some of the more noteworthy data of discharge and runoff during this flood are given in Table 7·10, all of these being in the Connecticut River Basin, mostly in Massachusetts.

Damage was due both to the flood and, to a much greater extent, to the hurricane. Flood damage was confined chiefly to western and central Massachusetts and Connecticut. The hurricane wave caused great damage

Table 7·10 Flood of September, 1938—Discharge and Runoff

River and point	Drainage area, sq miles	Max. discharge		Precipitation (Sept. 17–21), in.	Runoff (Sept. 17–21), in.
		Cfs	Csm		
Connecticut River:					
Montague City, Mass.	7,840	195,000	24.9	6.4	3.7
Thompsonville, Conn.	9,637	236,000	24.5	7.3	4.0
Ottauquechee, North Hartland, Vt.	221	24,000	110	6.5	3.8
Williams, Chester, Vt.	27	12,200	445		
West, Newfane, Vt.	308	52,300	170	8.4	5.0
Millers, Erving, Mass.	370	29,000	78	12.4	7.9
Deerfield, Charlemont, Mass.*	154	56,000*	352	8.8	7.1
Cold, near Charlemont, Mass.	23	14,600	624		
Ware, Gibbs Crossing, Mass.	199	22,700	114	13.0	8.1
Westfield, Westfield, Mass.	497	55,500	112	10.0	5.3

* Exclusive of drainage area above Harriman Reservoir.

on the southern New England shores, and the wind damaged buildings and trees all over southern New England and extending into Vermont and New Hampshire.

Direct flood damages in the Connecticut River Basin totaled about 25 million dollars, 60 per cent of which was in Massachusetts and 20 per cent in Connecticut, while total damage, including indirect, was about 46 million dollars. In the Merrimack River Basin, in New Hampshire and Massachusetts, total flood damages were about $6,000,000. Direct damages upon other river basins were as follows:

River Basin	Damage
Thames, Conn.	$5,870,000
Housatonic, Conn.	2,310,000
Blackstone, Mass. and R. I.	260,000
Pawtucket, R. I.	150,000
Total	$8,590,000

If the same proportion of indirect damages as in the Connecticut River

Basin were used, the total damage upon these four river basins would be brought up to about $15,000,000. Hence a total damage of about $57,000,000 occurred upon these six river basins. Their total drainage area is about 17,000 sq miles, and the average total damage in these portions of New England was therefore about $3,300 per square mile. In the Connecticut River Basin (about 10,000 sq miles), the total damage was about $4,400 per square mile.

On certain tributaries of the Connecticut, total flood damages were much higher. These include:

River	Total damage	Drainage area, sq miles	Damage per sq mile
Millers	$6,700,000	315	$21,400
Deerfield	7,400,000	465	16,000
Chicopee	8,600,000	721	11,800

These are unusually high unit damages and exceed most of those which have occurred in floods in other sections of the United States.

The damage due to the hurricane and its accompanying sea wave cannot be accurately stated. According to Brooks,[1] about 600 people were killed, and the damage to property was the greatest in a single storm anywhere in the world, the economic loss being approximately 250 to 300 million dollars (aside from river flood damages). It was clearly the greatest disaster ever occurring in New England, as well as one of the greatest disasters of national scope, and will be long remembered.

References

The Hurricane Floods of September 1938, *Water Supply Paper* 867, U.S. Geological Survey.

BROOKS, CHARLES F.: "Hurricanes into New England—Meteorology of Storm of Sept. 21, 1938," Blue Hill Meteorological Observatory.

Hurricane of September, 1944.[2] This developed under general atmospheric conditions not unlike those of the 1938 hurricane. It was reported on Sept. 8 from San Juan, Puerto Rico, and on Sept. 13 was centered off Florida moving northwest at about 12 mph. Moving more rapidly, its

[1] BROOKS, CHARLES F.: "Hurricanes into New England—Meteorology of Storm of Sept. 21, 1938," Blue Hill Meteorological Observatory.

[2] BROOKS, CHARLES F., and CONRAD CHAPMAN: The New England Hurricane of Sept. 1944, *Geog. Rev.*, Vol. 35, No. 1, January, 1945.

center passed Hatteras at 30 mph about 9:30 A.M. on Sept. 14 and then traveled the 400 miles from Hatteras to Long Island in 11 hr, or about 36 mph. Continuing at about this velocity, it passed across Rhode Island between Westerly and Providence; thence across southeastern Massachusetts, to near Eastport, Me., and to the southeast of Greenland.

During Sept. 14, peak surface winds were usually about 75 to 90 mph—the highest recorded at 12:20 P.M. of 134 mph at Cape Henry, Va. The usual hurricane rains prevailed on the left of the track with 4.3 in. at Hartford, Conn., and 4.7 in. at Providence, R.I. Fortunately, because of low tides, the effect of tidal waves was not serious.

In the course of the hurricane along the Atlantic Coast, 46 persons were killed and many injured, and property damage was estimated at $100,000,000. Timely warnings by radio and other means saved many lives and much property.

In Fig. 2·6 are shown the paths of the hurricanes of 1815, 1821, 1938, and 1944 along the Atlantic Coast. As will be noted, they are similar in form as far as New England, where they diverge, more commonly following the coast line.

The average frequency of hurricanes that seriously affect New England and the North Atlantic Coast, as shown by these four occurrences since 1815, is about once in 30 years, although those of the severity of that of 1938 have a frequency of probably a 100 years or more.

14. Los Angeles, 1938

This occurred upon Los Angeles and San Gabriel Rivers and their tributaries, Los Angeles County, California, on Mar. 2, 1938.

There were two distinct periods of rainfall between Feb. 27 and Mar. 4, 1938. The first was caused by a disturbance moving in over southern California on Feb. 27, which was met by a cold front from the northwest. Heavy rainfall resulted from Feb. 28 to Mar. 1. On Mar. 2, a second and more intense disturbance originating northeast of the Hawaiian Islands moved in to the Oregon coast. The greatest intensity of rainfall, however, occurred over southern California. As a result of warm-front action and a pronounced orographic effect, the intense rainfall of Mar. 2 was produced, this being directly the cause of the great flood.

The maximum rainfall for the six days, Feb. 27–Mar. 4, occurred in the San Gabriel Mountains and amounted to 30 in. More than 4 in. fell over practically the entire county, the area of which is about 4,000 sq miles. In general, about half the total rainfall fell on Mar. 2.

Approximate rainfall-area relations were as given in the following chart:

Rainfall, In.	Area, Sq Miles
28	1
26	9
24	80
22	150
20	240
18	350
16	460
14	670
12	1,070
10	1,780

In Table 7·11 are given data of discharge and runoff for this flood.

Table 7·11 Los Angeles Flood of March 2, 1938—Discharge and Runoff

River and point	Drainage area, sq miles	Max. discharge		Max. 24-hr runoff, in.
		Cfs	Cfs per sq mile	
Los Angeles River, Van Nuys Boulevard	157	12,100	77	
Los Angeles River, Dayton Avenue	510	68,000	133	
Loop Canyon, Veterans Hospital	1.06	1,820	1,717	
Pacoima River, Pacoima Dam	27.77	8,150	293	4.13
Big Tujunga River, Big Tujunga Dam	81.35	34,000	418	5.73
Arroyo Seco, Devil's Gate Dam	30.62	11,400	372	4.54
Pasadena Glen, Falls	0.78	2,800	3,589	
Little Santa Anita River, Sierra Madre Dam	2.39	700	293	4.19
Santa Anita River, Big Santa Anita Dam	10.82	4,600	425	5.90
Sawpit River, Sawpit Dam	3.27	1,120	342	4.71
San Gabriel River, Dam No. 1	202	94,380	467	7.12
Devil's Canyon, above San Gabriel, Dam No. 2	15.4	23,000	1,493	
West Fork San Gabriel River, Dam No. 2	40.42	23,800	588	7.43
Big Dalton River, Big Dalton Dam	4.49	1,300	290	3.70
San Dimas River, San Dimas Dam	15.9	4,400	276	4.00
Puddingstone River, Puddingstone Dam	33.1	5,850	177	3.01
Live Oak River, Live Oak Dam	2.30	355	154	2.49
Thompson Creek, Thompson Creek Dam	3.91	600	154	2.75
Santa Clara River, Sta. F92R	355	24,000	68	

The flooded area included four streams with numerous tributaries, with drainage areas and flood damages as given in Table 7·12. The overflowed area contained land and improvements valued at over one billion dollars, with a population of about 800,000. The flood affected not only these

Table 7·12 Los Angeles Flood Losses, March, 1938

River basin	Drainage area, sq miles	Overflowed area, sq miles	Total flood damages, March, 1938	Lives lost	Flood losses per sq mile
Los Angeles	753	202	\$25,500,000	43	\$33,800
San Gabriel	698	220	10,500,000	1	15,100
Rio Hondo	137	47	3,100,000	2	22,600
Ballona Creek	129	39	1,300,000	3	10,000
Totals	1,717	508	\$40,400,000	49	\$23,400

people but everyone in the Los Angeles metropolitan area, because of the interruption of industry, power, light, water, and transportation facilities. As will be noted, the flood losses per square mile of drainage are among the highest observed in this country.

References

Flood of March 2, 1938, Los Angeles County Flood Control District, May 20, 1938.
Monthly Weather Rev., May, 1938, pp. 135–143.
Preliminary Examination and Survey of Los Angeles and San Gabriel Rivers, H.R. Doc. 838, 76th Cong., 3d Sess., U.S. Engineer Corps, 1940.

Floods Due to Reservoir Dam Failures

It seems pertinent to include in the description of floods some of those which have been due not essentially to great rainfalls but rather to the failure of reservoir dams, where the sudden release of stored water caused excessive flows and resulting loss of life and damage to property. A few of the worst of these disasters will be described.

1. Johnstown Flood, 1889. This occurred on South Fork Creek and Conemaugh River near Johnstown, Pa., on May 31, 1889, in connection with an Atlantic coastal-type storm in which 8 or 10 in. of rainfall occurred during the 2 days May 30–31.

The cause of the flood disaster at Johnstown was the failure of an earth dam by overtopping, because of inadequate spillway capacity, at the South Fork Reservoir, some 10 miles above Johnstown, where about 500 million cu ft of water was stored, with a tributary drainage area of about 50 sq miles. The dam was about 72 ft high, 20 ft thick at the top; upstream slope 1 on 2, downstream slope 1½ on 1, with a spillway 72 ft long and 8 ft of freeboard.

During the forenoon of May 31, it is said that some 2¾ in. of rain fell in 20 min., and by noon the entire length of the embankment was overflowing. By 3:20 P.M., the highest part of the embankment was undermined, and a section 20 ft wide and 10 ft deep broke away. Within a few minutes, a great mass of the embankment washed out, releasing the water in the reservoir, all of which ran out in about 45 min., the greater part of the overflow occurring in about 30 min. The flood wave thus formed probably exceeded 200,000 cfs (or 4,000 csm) in discharge. It rushed down the 10 miles of narrow, winding valley, sweeping clear the small communities on the way, and burst upon the city of Johnstown, already somewhat inundated by the natural high water.

Already heavily laden with debris, this great wall of water, 125 ft or more in height, rushed down the river to Johnstown at a speed of 50 mph or more and quickly reduced a large area of the city to a mass of wreckage, which, at and near the stone bridge holding it, covered an area of over 30 acres to a height of 30 ft with perhaps the most extraordinary collection of debris ever assembled. To add to the disaster, this mass of wreckage took fire, and many people were burned and others drowned. The total loss of life is uncertain but is said to have been at least 7,000. Property damage was over $20,000,000. It was certainly one of the dire catastrophes of record in this country.

References

Eng. News, June 8, 1889, p. 517; and June 15, 1889, p. 540.
Trans. A.S.C.E., Vol. 24, p. 431, 1891.
Harper's Magazine, September, 1933, p. 443.

2. Failure of Mill River (Mass.) Dam, 1894. Mill River is a small stream of about 21 sq miles drainage area, entering the Connecticut River at Northampton, Mass., from the west. The flood was due solely to the failure of a storage dam at Williamsburg built in 1865 by the Williamsburg Reservoir Co.

This structure was about 500 ft long, of earth embankment, with a top width of 16 ft and 1 on 1½ slopes and a core wall of stone with cement mortar about 40 ft in maximum height, 2 ft top thickness and a batter of 1 in. per ft. A 33-ft masonry waterway, upon an earth foundation, was located at one end, and there was a 16-in. outlet pipe.

At the time of failure, the water level was 4 ft below the top of the embankment. The reservoir covered 111 acres, with an average depth of 20 ft, or approximately 2,000 acre-ft of capacity. Inspection of the remains of the dam showed that the core wall was not upon an impervious foundation in some places, and it was concluded that water seeped under the wall, causing the downstream embankment to slip. About three-fourths of the reservoir contents escaped or about 1,600 acre-ft in 20 min, or at an average rate of nearly 60,000 cfs.

The drainage area below the East Branch at Williamsburg is about 21 sq miles, so that the flow averaged about 2,900 csm. A heavy loss of life occurred, totaling 143, and damages of more than a million dollars, or nearly $50,000 per square mile of drainage area. The great possibilities of disaster due to the failure of a reservoir dam even on a small stream are obvious from this disaster.

Reference

FRANCIS, JAMES B., THEODORE G. ELLIS, and WILLIAM WORTHEN: Report upon Failure of Dam at Mill River, *Trans. A.S.C.E.*, Vol. 3, pp. 118 *et seq.*, 1874.

3. Failure of St. Francis Dam, near Los Angeles, Calif. St. Francis Dam, in the San Francisquito Valley about 45 miles north of Los Angeles, failed about midnight Mar. 12–13, 1928. It controlled a drainage area of 36 sq miles, and its reservoir of 38,000 acre-ft was practically full at the time.

The dam was of concrete masonry in maximum section 205 ft high and 176 ft thick at the base. It was about 650 ft long and curved in plan, with a radius of 500 ft. Failure occurred suddenly with no evidence of prior excessive seepage past the dam. The resulting maximum flow near the dam was of the order of 400,000 to 500,000 cfs, and the flood wave proceeded downstream at the following rates:

Location	Time	Time, min, from preceding station	Distance, miles	Velocity, mph
At dam	11:58 P.M.			
City Power House No. 2	12:03 A.M.	5	1.5	18.0
Southern California Edison Co.—substation	12:38 P.M.	35	7.5	12.9
Southern California Edison Co. Construction Camp	1:20 P.M.	42	7.5	10.7
Fillmore Bridge	2:25 P.M.	15	12.7	11.7

All but 100 ft of the dam was washed away, together with Power House No. 2. Over 200 lives were lost, and damages to the extent of about $10,000,000 occurred.

The failure was due to defective foundation material. A board of engineers and geologists in their report stated:

Material under the left end (of the dam) was mica schist of thin and easily separable laminae. The right end was founded on a reddish conglomerate which even when dry was of decidedly inferior strength and when not became so soft as to lose almost all characteristics of rock. With such a foundation the ultimate failure was inevitable unless water could have been kept from the foundation.

The lesson to be learned from this failure, as pointed out by *Engineering News-Record* in an editorial at the time, was the necessity of having a board of engineers rather than a single man, no matter how competent, responsible for the manner of construction of a dam of this magnitude.

Reference

Eng. News-Record. Nov. 22, 1928, *et seq.*, 1928.

CHAPTER 8

FLOODS—DAMAGES OR LOSSES

The flood plains of rivers have always been favored for settlement and use, often because of nearness to navigation or railroad facilities, and ease of building construction and land use on flat terrain. Although these flood plains are usually above ordinary flood or freshet level, they often may be inundated from time to time by greater or rarer floods that subject their inhabitants to the possibility of severe flood damages. Such conditions are so common that, in the United States as a whole, damaging floods of greater or less degree occur somewhere every year, and there is probably a flood problem of more or less urgency upon every few miles of the thousands of streams in the country.

Data of flood *losses* are ordinarily obtained and recorded for the purpose of determining the need for and extent of relief measures that may be desirable. They also serve as a basis from past experience to determine the *benefits* from such relief, which would be due to the prevention of such losses as well as the increase, if any, in the value and utility of property.

Character of Flood Damages

Flood damages are due to either or both (1) the result of inundation by high stages of flood waters and (2) the effects of high currents produced by excessive discharge. It is difficult to segregate these effects as to damages, and it is a question which kind of action may cause more damage.

1. Inundation. The worst effect of inundation is the deposit of yellow, slimy, fine penetrating mud. This is particularly bad in urban residential and manufacturing areas in its damage to property and in the difficulty and cost of removal. Deposits of silt may be of some value on farm lands, if not too heavy, in improving their fertility, but this is often offset by damage to buildings and other property.

2. Velocity. The effects of high current or velocities include the sweeping away of bridges, houses, and other buildings, the tearing up of urban streets and roads and the erosion of farm lands, with topsoil often washed away and deposits of sand and gravel left covering tillage lands. Often new river channels may be formed by cutoffs and destructive changes made in local river conditions.

Classification of Flood Losses

The losses caused by a flood may be generally classified as

1. *Direct,* or actual and apparent, losses that can be evaluated by the cost of replacement or repair of all physical damage occasioned by the flood.

2. *Indirect,* which may be defined as the value of the business, service, or economic activity either lost or required by the flood. They may be so complicated as to make even a rough estimate difficult and are often taken as a proportion, 0.5 to 1.0 or more of direct flood losses.

3. *Intangible,* which are not subject to direct monetary evaluation, including loss of life, and effects upon health, social, and economic security.

Schedule of Flood Losses. Flood losses of all three above classifications may be scheduled under the following general headings:

1. Residential
2. Commercial
3. Industrial or manufacturing
4. Utilities
5. Rural
6. Highways
7. Railroads
8. Public

1. *Residential Losses.* These include damage to buildings and grounds, furnishings, and personal property, varying in extent with the class of occupancy. Entire houses may be carried away, resulting in total loss, where submerged by swift, deep water, or they may be undermined by erosion of the riverbanks. In other cases, damage to buildings, personal property, furniture, furnishings, etc., is due to immersion, requiring repairs and redecorating and often replacement of articles lost or damaged.

The cost of moving out during the flood and providing temporary habitation, loss of rent, and flood-relief costs are *indirect* items of flood losses under this general heading.

2. *Commercial Losses.* Included in this classification are those losses sustained by establishments engaged in trade; banks, professional, and other offices; service concerns, shops, restaurants, hotels, theaters, etc. Loss of merchandise, often total, business accounts and records, etc., repairs, cleaning, and redecorating of buildings are all usual items.

Indirect losses may include wages, salaries, and fixed charges, business and good will, unfilled contracts, extra operating costs, etc.

3. *Industrial or Manufacturing Losses.* These include losses sustained by manufacturing establishments that process raw material to form a finished

product. Smaller buildings may occasionally be washed away, but the usual kinds are more likely to be flooded and require repairs and replacements of floors, windows, and finish. Machinery is a variable item but usually includes the cost of cleaning and drying. Raw materials or stock may be items of total loss or sometimes can be partly salvaged.

Indirect losses will include items similar to those for commercial losses.

4. *Utilities.* Utilities include all agencies or private and public organizations that provide or sell some form of service to the consumer. These include electric light and power, gas, telephone and telegraph, railroads, water supply, and navigation facilities, all of which may have properties subject to loss or damage due to floods similar to many kinds already described.

Indirect losses may be due to interruption of service and loss of revenue for these utilities.

5. *Rural.* In this classification are farm and rural losses due to damages sustained by agricultural lands and their properties, including buildings, farm equipment, erosion of land or disposition of debris and the rendering of land untillable; crop losses, loss of livestock, etc.

Indirect losses include those similar to residential losses; also the cost of any replantings, of reduction in crop yields, and of the sale of farm products.

6. *Highways.* Bridges washed away or damaged; washing and erosion of streets and roads and deposits of debris; loss of highway transport and operating equipment and goods in transport are included under this heading.

Indirect losses may be due to service and traffic interruptions and the cost of rerouting traffic.

7. *Railroads.* These losses are similar to those for highways, including loss or damage to bridges, track and right of way, equipment and rolling stock.

Indirect damages are similar to those for highways but also include loss of revenue due to traffic and service interruption and increased cost of operation.

Railroad and highway losses, largely similar in nature, are often combined under one heading.

8. *Public.* Including all losses, these are similar to those previously described for property owned by Federal, state, and municipal governmental agencies.

Indirect losses include interruption of service and facilities, and loss of records and accounts.

Recurrent and Nonrecurrent Flood Losses. Recurrent flood losses are those which may be expected from each recurrence of a flood. Nonrecurrent flood damages are those which a second flood would not again produce, because of the nature of replacements or repairs made after the first flood. For example, a bridge replacement should, if practicable, be at such an elevation above flood level that it would not again be subject to being swept away or damaged by another flood. Similarly, highway repairs or replacements may often be made in such a manner as to make future flood loss nonrecurrent or largely so.

Usually only recurring flood losses need be considered in studies of future flood losses and of flood benefits due to flood control. Obviously recurrent and nonrecurrent damages may be both direct and indirect in their nature.

Methods of Collecting Data on Flood Losses

One of the first problems usually encountered in initiating a flood-control study is that of making a survey of past flood losses. Usually also the results must be extended to include a possible higher flood stage than that which has occurred to date.

The general method of survey used will depend upon the size of drainage area under consideration, its importance commercially and economically, and the extent and magnitude of the flood hazard and possibilities of flood loss. The funds available may also often limit the extent and character of the investigation.

In important cases, a detailed survey may be made of the entire flood zone, with an extensive and complete damage canvass. In other cases of less importance or covering larger areas, a less complete investigation using some form of what may be called the "sampling method" may be applicable.

Detailed Surveys. In a typical example, consisting of a flood zone of about 300 sq miles, this was subdivided, each subarea with an engineer and staff of assistants. Every property owner or tenant who could be located was interviewed. The field men carried standardized data sheets for each type of property, listing direct and indirect losses separately.

In Appendix B of this book, page 414, there are given examples of data sheets for listing flood damages. These sheets as used are 11 in. high by 16 in. or more in length. They were furnished through the courtesy of the district engineer of the Cincinnati office of the U.S. Engineer Corps.

An estimate of monetary value was obtained for as many sites as possible. To check possible exaggerations, the field canvassers carried unit costs for poor, medium, and good condition of some 40 items of property.

Visits were made to determine damages to state highways and bridges and to Federal, state, and municipal property, public and private utilities, and large corporations, supplemented by all available ground and aerial photographs taken during and after the flood.

Data collected by field personnel were supplemented by those from several state and local observations, and all available public agencies were consulted with reference to indirect damages. It took 45 days to collect data and an additional 3 months of office work for compilation and study.

Sampling Surveys. This is a method often used, when the detailed method is impracticable or too costly. It consists of selecting groups of similar property units—residential, industrial, or commercial—determining damages to a number of typical units in the group, and obtaining the total group damages by applying this unit value to the total number of units in the group.

This method might be used for a city, grouping 30 to 100 houses in a given area and determining direct and indirect damages for, say, four houses in each area, applying the average damage thus obtained to the whole number of houses in the group. Small stores and other establishments capable of grouping may be similarly handled.

Questionnaires, verified by visit, would be used to obtain flood damages to larger units—railroads, utilities, large stores, buildings, and churches, etc. Indirect losses would be estimated by means of information obtained from public agencies.

Modified Sampling Method. This is illustrated by a case in which it was desired to obtain data of flood damages for all river stages from zero damage to the maximum stage. Two serious floods were of recent occurrence, and the damage forms used provided for data on these two floods, with reference to flooding both to first floor level and to the maximum point. Actual damages were obtained where possible, and estimates were made elsewhere based upon the use of stage-damage curves. To prepare the latter, flood-loss data for the more recent record flood were obtained from selected properties, where accurate damage records had been kept. Over 50 stage-damage curves were prepared, including frame and masonry buildings, building equipment, personal effects, and stock and equipment for many types of business, etc. Controlling elevations of all properties were determined by level—the objective being to get several points on a stage-damage curve.

Other sampling methods may also be made, differing in detail from those above described but utilizing the same general idea.

Stage-damage Relations. In making flood-damage surveys, it is usually desirable to relate damage and river stage by means of suitably located or existing gauges. This will make possible the estimation of damages for more than one river stage. It can be accomplished most readily by the preparation of stage-damage curves such as those shown in Fig. 8·1,[1]

[1] *Trans. A.S.C.E.*, Vol. 107, p. 906, 1942.

in which (1) area overflowed, (2) agricultural damage per acre, and (3) nonagricultural damage are shown.

Agricultural Crop Losses

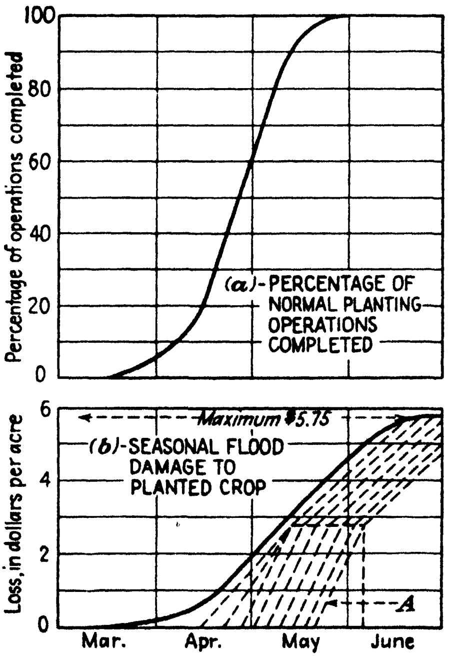
Potential crop losses from floods vary from season to season and with the area flooded. If a crop is flooded sufficiently to be a total loss, no further damage is caused by deeper water. Hence curves of "area flooded" (based preferably upon total rather than subtracted area) and "crop loss" at different seasons of the year must supplement the stage-damage curve. The risk of flood loss in northern latitudes varies from nothing in winter to full value or nearly so in summer. In warmer regions, some loss may occur throughout the year because of work nullified by flood. The "crop-loss" curve showing flood risk by months of the year should give loss per acre.

Crop losses are difficult to estimate because of different types of farming and crop distribution. Production costs will vary with topography as affecting the types of equipment used and hence production costs. Examples to illustrate the methods used follow.

In Table 8·1 and its accompanying diagrams[1] are shown the percentage

Table 8 · 1 Derivation of Factors Used in Plotting Curve for Damages to Prepared Ground and Planted Crop

First date of flood	Preparing and planting		Other operations		Total damages
	% completed	Loss per acre	% completed	Loss per acre	
March 15	0	0.00	0	0.00	0.00
April 1	6	0.17	0	0.00	0.17
April 15	20	0.55	0	0.00	0.55
May 1	63	1.73	6	0.18	1.91
May 15	94	2.58	20	0.60	3.18
June 1	100	2.75	63	1.80	4.55
June 15	100	2.75	94	2.82	5.57
July 1	100	2.75	100	3.00	5.75

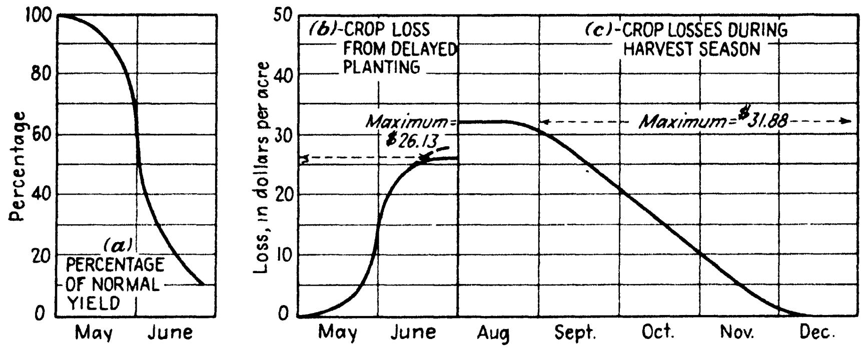


[1] *Ibid.*, pp. 907–909.

of normal planting operations completed, the seasonal flood damage to the planted crop, and the detailed tabular basis for these—for planting and cultivating 1 acre of cotton.

In Table 8·2 and its diagrams are shown (1) percentage of normal yield with respect to times of delayed planting, (2) the corresponding rate of decline in production value, and (3) crop losses during the harvest season.

Table 8·2 Derivation of Factors Used in Plotting Curve for Crop Losses



% yield*	Cotton yield† (lb)	Production costs, in dollars					Value of yield	Col. 8 minus Col. 7	Profit‖	Date
		Preparing‡	Overhead§	Total, Cols. 3 and 4	Harvesting and ginning	Total				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
100	375	5.75	20.25	26.00	15.00	41.00	46.88	5.88	0	April 20
90	340	5.75	20.25	26.00	13.50	39.50	42.20	2.70	− 3.18	May 20
80	300	5.75	20.25	26.00	12.00	38.00	37.50	− 0.50	− 6.38	May 26
70	260	5.75	20.25	26.00	10.50	36.50	32.80	− 3.70	− 9.58	May 30
60	225	5.75	20.25	26.00	9.00	35.00	28.10	− 6.90	−12.78	May 31
50	190	5.75	20.25	26.00	7.50	33.50	23.40	−10.10	−15.98	June 1
40	150	5.75	20.25	26.00	6.00	32.00	18.75	−13.25	−19.13	June 3
30	110	5.75	20.25	26.00	4.50	30.50	14.05	−16.45	−22.33	June 8
20	75	5.75	20.25	26.00	3.00	29.00	9.40	−19.60	−25.48	June 15
10	40	5.75	20.25	26.00	1.50	27.50	4.70	−22.80	−28.68	June 25
0	0	5.75	20.25	26.00	0	26.00	0	−26.00	−31.88	....

* From curve (*a*).
† Lint.
‡ Includes planting, cultivating, and poisoning.
§ Includes work stock care.
‖ Col. 9 minus 100% potential loss ($5.88), to be used in plotting curve (*b*).

The maximum value upon the latter diagram, of $31.88, is the sum of that in curve *b*, Table 8·1 ($5.75) plus that in curve *b*, Table 8·2 ($26.13). Combining these curves, a curve of potential crop loss per acre for the different months of the year may be obtained for a given locality or river reach.

Flood Risk

Floods are uncertain in both their time of occurrence and their magnitude, although often more likely to occur at certain seasons of the year, because of melting snow in the north and in the usual season of hurricanes in the south. On the lesser sized rivers, floods may occur quickly and with little warning. On the large rivers, the Mississippi, Ohio, etc., there is considerable foreknowledge of the coming flood.

Floods, like fires, are uncertain as to time and consequences and hence involve economic risks, rather than a certainty of loss. These include *property or capital risks* to property and goods, and *personal risks* to life, health, comfort, and employment. The former can be evaluated on a yearly basis if the property damage, direct and indirect, is known. Personal risk can be evaluated only indirectly and not upon a monetary basis.

Yearly Flood Losses

If the flood history of a river, including knowledge of flood losses, were complete and extending over many years, the average yearly flood loss could be readily determined by dividing the total period losses by the number of years of record. In some cases where the record includes a diversity of floods, this method may be attempted, but often lack of adequate data, particularly of flood losses, makes this impracticable.

In the usual situation, data of a few floods and their approximate losses may be available, perhaps including one at least of importance. These commonly afford the only means to use as a basis for the prediction of average yearly losses, which should include the effect of all floods reasonably to be expected. Flood surveys might be made and extended to higher stages, to supplement the available information, but often there is not sufficient interest in the matter, except after some damaging flood, to defray the cost of making such surveys. Furthermore, such surveys, to be dependable in extension, should preferably be made soon after a good-sized flood.

Discharge–Flood Losses Relations. Where several floods have occurred in a river basin and data of maximum discharge, as well as resulting flood losses, are available, it is often possible to construct a curve of discharge–flood losses relations. In Fig. 8·1 (curve A) is shown such a curve for the Connecticut River, where flood losses for the entire river basin are available for three recent major floods and are related to maximum flood discharge at Thompsonville, Conn. (9,637 sq miles drainage area), in the lower river. These are total flood losses for the entire Connecticut River Basin, including direct and indirect losses; and, as will be noted, the curve of flood losses against discharge is unusually well defined. The lower portion of the curve is consistent with a flood of 15 csm, or about 145,000 cfs, at Thompsonville as the stage below which flood damage is negligible. This limitation is fixed from experience gained in previous high-river stages and floods.

Frequency–Flood Loss Relations. In Table 8·3 are computations of average yearly flood losses in the Connecticut River Basin based upon the discharge–flood loss relations upon Fig. 8·1 and the use of the dis-

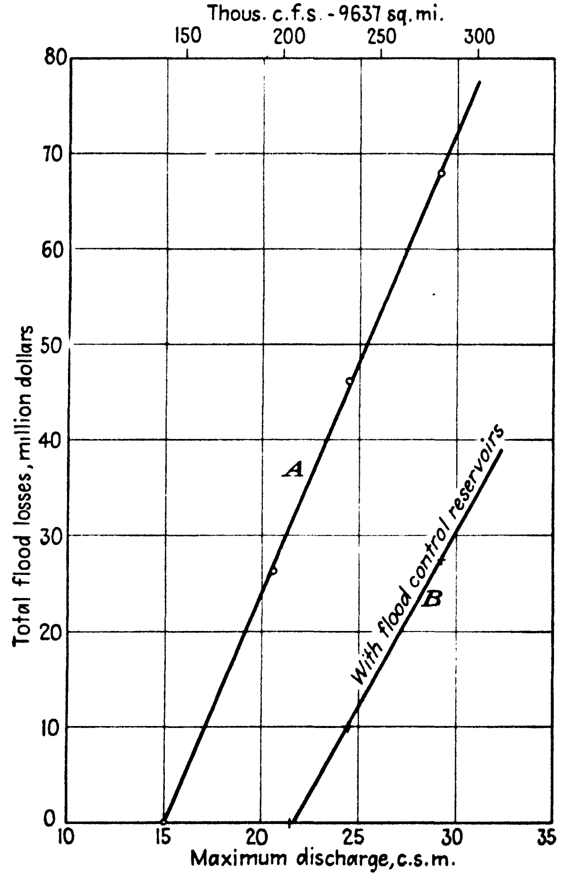


FIG. 8·1. Connecticut River at Thompsonville, Conn. Flood losses/discharge.

charge–frequency curve, previously shown (Fig. 5·11C, page 80) for Thompsonville.

In Table 8·3,

Columns (1), (2), and (3) are based upon Fig. 8·2.

Columns (4) and (5) are based upon Fig. 5·11C.

Column (6) is the increment of flood losses in column (3) divided by the time in years in column (5).

Column (7) is the summation of figures in column (6).

In this case, the increment of flood loss remains substantially constant

Table 8·3 Computations of Average Yearly Flood Losses for the Connecticut River, Based upon Frequency

Peak flow, csm (1)	Total flood losses, million dollars (2)	Increment of flood losses, million dollars (3)	Avg. frequency, years (4)	Avg. frequency of increment, years (5)	Avg. yearly increment of flood losses (6)	Yearly flood losses (7)
15	0	. . .	10			
16	4.5	4.5	12	11.0	$410,000	$ 410,000
17	9.0	4.5	16	14.0	310,000	720,000
18	14.0	5.0	23	19.5	256,000	976,000
19	19.0	5.0	27	25	185,000	1,111,000
20	24.0	5.0	30	28.5	175,000	1,336,000
21	28.5	4.5	33	31.5	142,000	1,478,000
22	33.5	5.0	43	38	132,000	1,610,000
23	38.3	4.8	75	59	82,000	1,692,000
24	43.0	4.7	90	82	57,000	1,749,000
25	48.0	5.0	100	95	53,000	1,802,000
26	53.0	5.0	120	110	45,000	1,847,000
27	58.0	5.0	150	135	37,000	1,884,000
28	63.0	5.0	200	175	29,000	1,913,000
29	68.0	5.0	260	230	22,000	1,935,000
30	72.5	4.5	300	280	16,000	1,951,000
32	82	9.5	400	350	27,000	1,978,000
34	92	10	550	475	21,000	1,999,000
36	102	10	700	625	16,000	2,015,000

at about 5 million dollars per csm of flow. About two-thirds of the average yearly losses occur between 15 to 21 csm flow, or 140,000 to 250,000 cfs, and the relatively high frequency of these floods results in higher incremental losses than those which occur in the greater floods. Thus the largest flood listed, *viz.*, 36 csm, adds only $16,000 to the yearly total; and the fact that the frequency of the greater floods is only roughly determinable, as previously noted, therefore does not sensibly affect results. The average yearly flood loss based upon Table 8·3 is about $2,000,000 for the Connecticut River Basin.

Flood-control Benefits

Flood-control benefits consist in general of the elimination of flood losses. Hence average flood losses will also be average yearly benefits, assuming that all losses will be recurrent and that all damaging floods are eliminated by the control works. Where reservoir systems are used to control floods, they may not be of sufficient capacity to effect this result completely. Further consideration will be made of the Connecticut River Basin to illustrate the method of determining the effect of flood-control measures upon average yearly flood losses and benefits.

Connecticut River Reservoir System. The tentative plan of reservoirs for flood control upon the Connecticut River includes the following: Power storage was effective in the March, 1936, flood of about 380,000 acre-ft; in the November, 1927, flood, about 340,000 acre-ft. In both cases, the effect of this storage appears in the records of flood flow at Sunderland (or Montague) and Thompsonville on the lower river. Additional power storage is contemplated of about 200,000 acre-ft, all above the Comerford plant in the upper river, where the drainage area is 1,650 sq miles. A proposed system of 20 flood-control reservoirs with a controlled drainage area of 2,752 sq miles and 940,000 acre-ft of storage capacity is planned, at a cost of about $54,000,000.

Flood Discharge—Losses with Flood-control Reservoirs. In Table 8·4 are computations to determine the effect of the above reservoir system in reducing the flood peak flow.

Columns (2) and (3) are discharge at Thompsonville in cfs and csm.

Column (4) is available storage in power reservoirs.

Column (5) is total power-reservoir storage, limited to a capacity of 100,000 cfs days.

Column (6) is available storage in the 20-reservoir flood-control system.

Column (7) is total storage in this system, limited to a capacity of 470,000 cfs days.

Column (8) is the sum of available power and flood-control storage, or the sum of columns (4) and (6).

Column (9) is the reduced flow at Thompsonville due to the storage in column (8), or column (2) − (8).

As will be noted, the maximum daily flow at Thompsonville is reduced from 256,000 cfs on Mar. 19 to 198,000 on Mar. 20, the latter figure representing the highest daily flow when storage is available, or a ratio of 198,000/278,000 = 0.71 of the peak day flow. Approximately this same ratio would apply to the maximum flow of 282,000 during the day of Mar. 20, reducing this to 202,000 on Mar. 19, with reservoirs in operation.

Applying this ratio of 0.71 to the flow of 29.3 csm in Fig. 8·2 gives a flow of 20.8 csm, for which the flood losses would be about 27.5 million dollars, representing the flood losses in the March, 1936, flood as affected by power and flood-control losses as shown in Table 8·4, giving a point upon curve *B*, Fig. 8·2, showing flood losses with reservoir systems in operation. Other points were obtained in a similar manner for the flood of September, 1938,

Table 8·4 Connecticut River Basin Flood of March, 1936—Effect of Proposed 20-reservoir System and Existing Power Storage

Date in March, 1936	Discharge at Thompsonville		Power storage 1,650 sq miles 100,000 cfs days capacity		Flood-control storage 2,752 sq miles 470,000 cfs days capacity		Col. (4) + Col. (6)	(Col. 2) — (Col. 8)
	Cfs	Csm (9,627 sq mile)	Daily, cfs	Total, cfs days	Daily, cfs	Total, cfs days		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
11	11,900	1.3	2,000	2,000	4,000	4,000	6,000	5,900
12	57,400	6.0	10,000	12,000	16,000	20,000	26,000	31,400
13	98,600	10.2	17,000	29,000	28,000	48,000	45,000	53,600
14	120,000	12.5	21,000	50,000	35,000	83,000	56,000	64,000
15	114,000	11.8	19,000	69,000	33,000	116,000	52,000	62,000
16	101,000	10.5	17,000	86,000	29,000	145,000	46,000	55,000
17	86,800	9.0	14,000	100,000	25,000	170,000	39,000	47,800
18	135,000	14.0	......	......	39,000	209,000	39,000	96,000
19	256,000	26.7	......	......	74,000	283,000	74,000	182,000
20	278,000	29.0	......	......	80,000	363,000	80,000	198,000
21	249,000	26.0	......	......	72,000	435,000	72,000	177,000
22	220,000	22.8	......	......	35,000	470,000	35,000	185,000
23	189,000	19.7	......	......	......	......	......	189,000
24	154,000	16.0	......	......	......	......	......	154,000
25	118,000	12.2	......	......	......	......	......	118,000

Reservoir effect = 198,000–278,000 = 0.71 of peak day flood flow.

and the flow for no losses with the reservoirs in operation, the latter being at about 21.5 csm.

Although the foregoing method of flood routing is approximate, being based upon daily mean flows and considering the entire basin as to both flow and flood losses, the results are sufficiently accurate for reasonable conclusions because of limitations in accuracy due to the uncertainty in frequency, as shown in Table 8·3.

Flood Losses with Flood-control Reservoirs. In Table 8·5 are given computations of average yearly flood losses with the power and flood-control reservoir systems in operation, based upon curve *B* on Fig. 8·2, and a corresponding revision of Table 8·3. The average yearly flood losses as determined in Table 8·5 are about $330,000.

Table 8·5 Average Yearly Flood Losses for the Connecticut River with Flood Control and Power Reservoirs in Use

Peak flow, csm (1)	Total flood losses, million dollars (2)	Increment of flood losses, million dollars (3)	Average frequency, years (4)	Average frequency of increment, years (5)	Average yearly increment of flood losses (6)	Average yearly flood losses (7)
21	0	. .	33			
22	1.0	1.0	43	38	$26,000	$ 26,000
23	4.8	3.8	75	59	65,000	91,000
24	8.5	3.7	90	82	45,000	136,000
25	12.0	3.5	100	95	37,000	173,000
26	15.5	3.5	120	110	32,000	205,000
27	19.5	4.0	150	135	30,000	235,000
28	23.0	3.5	200	175	20,000	255,000
29	27.0	4.0	260	230	17,000	272,000
30	30.5	3.5	300	280	12,000	284,000
32	37.5	7.0	400	350	20,000	304,000
34	41.5	7.0	550	475	15,000	319,000
36	48.5	7.0	700	625	11,000	330,000

Average Yearly Benefits and Cost of Reservoir Systems. Average yearly benefits are as follows:

Average yearly flood losses *without* reservoir systems, from Table 8.3	= $2,015,000
Average yearly flood losses *with* reservoir systems, from Table 8·5	= $ 330,000
Average yearly benefits	= $1,685,000

Average yearly costs of flood-control reservoirs:

Fixed charges and operation, 6 per cent of $54,000,000	= $3,240,000
Excess of yearly costs over yearly benefits	= $1,555,000

It is obvious that the flood-control reservoir system as at present proposed for the Connecticut River Basin is not economically desirable. The yearly benefits of $1,685,000 indicate an expenditure of not over $40,000,000 as the limit for the reservoir system. The basic reason for this situation in the Connecticut River Basin is an overestimate of flood benefits, by considering property depreciation added to direct and indirect losses. This matter will now be discussed.

Depreciation as a Measure of Flood Losses

A flood loss that has sometimes been assumed, in addition to direct and indirect damages, is depreciation in the value of property as a result of flooding. In this case, depreciation is computed by determining as accurately as possible the difference in the sale value of property in the flood zone before and after flooding and taking the difference as the depreciation loss. Since depreciation in property values follows a flood, it must basically be a result of all the damages occasioned by that flood, *viz.*, the sum of the direct and indirect damages.

For example, consider an industrial property worth $100,000, which in an unusual flood has damages (direct plus indirect) of $20,000. Immediately after the flood, it would sell for somewhat less than $80,000. The cost of putting it back in good condition would be $20,000, which would include all damages and the resulting losses of business. There would remain, however, an element of lessened value due to the fear of recurrence of another large flood; and, for some time after the flood, the property would therefore have a sale value somewhat less than $80,000. In time, however, "flood-consciousness" would tend to disappear, and this element of depreciation due to flood possibility would not apply.

Now consider benefits due to flood prevention for this same industrial property, including the effect of *all* floods on values. The benefits will be based upon an average yearly flood loss for all expectant floods—large and small. This average yearly flood loss will be based upon all direct and indirect losses for the entire range of damaging floods and may be, say, $600 yearly for this particular property. Without flood protection, this property, costing $100,000, is worth less because of flood possibilities by $600, capitalized at, say 6 per cent, or $10,000; and its sale price would be $90,000 instead of $100,000. We may say that, because of flood hazard, it has depreciated $10,000 in value; but this amount is based upon average yearly flood damages—direct plus indirect, and actually depreciation as thus used is nothing more than an expression of the direct and indirect flood losses.

To put it in another way, a fund that would yield $600 per year, if available, would take care of all flood losses and would make the property worth $100,000 at all times. If, instead of a flood fund of $600 per year, flood-prevention works were constructed capable of preventing damaging floods again, all depreciation in value due to floods would be eliminated.

It is obvious, therefore, that the determination of depreciation losses by the lessening in value of property due to floods is merely another—less perfect—method of estimating the total direct and indirect flood losses. The difficulty in using this method of arriving at flood damages is in accurately determining property depreciation from flood effects alone, since natural depreciation from advancing age, as well as from other economic causes, is often difficult to exclude or segregate as it should be. There is also usually exaggeration in the estimates of property owners at times of flood, which, however, becomes less as time goes on.

The use of depreciation as a separate item of flood damages results in magnifying estimates of flood benefits by two times or more, because it is akin to making two estimates of value by different methods and *adding* these two results instead of averaging them.

Thus, in the case of the estimation of flood benefits for the Connecticut River Basin,[1] where depreciation is included in estimating flood benefits, the following yearly figures resulted:

Due to		Ratio to direct benefits
Direct benefits	$ 853,000	1.0
Indirect benefits	808,000	0.94
Depreciation	3,597,000	4.20
Total	$5,259,000	6.15

In this case, depreciation was based upon estimates made for the 1936 flood—the largest on record—whereas the direct and indirect benefits were based upon the average yearly flood over a long period of years. Direct plus indirect yearly benefits total $1,662,100. Allowance for intangible losses may raise this total to, say, $2,000,000. The depreciation method, which is (or should be) another determination of the same quantity, gives $3,597,000. This is obviously based, not upon a long-time average flood but upon the greatest flood of record, of very recent occurrence, and hence inflated in amount.

[1] H. R. Doc. 455, 75th Cong., 2d Sess., pp. 108–109.

The fallacy of adding together two determinations of the same quantity instead of averaging them, if of equal weight in accuracy, is obvious. Considering the uncertainty of the estimated yearly depreciation, it appears that yearly benefits for the Connecticut River project should not rationally be greater than about $2,000,000. Note that this is consistent with the studies made in Table 8·3, page 149. This warrants an expenditure for flood relief of about $40,000,000, instead of about $100,000,000, which resulted when depreciation benefits were added in (as was done in the report previously cited, upon which the approval of this flood-control project was based). The comprehensive flood-relief project for the Connecticut River, including local dikes and other features, is now estimated to cost about $83,000,000. It is clearly of doubtful economy, and such a figure indicates that the project should not have been approved in the form presented.

Depreciation due to flood damages should not therefore be included as a separate item, since it or its effect is already included if all direct and indirect flood damages are included. As a substitute method for determining flood damages, it is uncertain and usually subject to error—especially if computed directly after a great flood—and it should not therefore generally be used.

Enhancement in Property Value Due to Flood-control Projects

A factor called "enhancement in property value" is sometimes included in estimating flood benefits from flood-control projects. It is an estimated increase in value of property that will be developed when the property is put to a higher order of use than was possible when it was subject to flooding. Obviously, it differs from restoration of depreciation. An example of enhancement in property value might occur when pasture land subject to flooding and hence limited in this use, prior to the construction of flood-control works, could under the new conditions be used for tillage purposes and raising crops.

This factor of enhancement in value is, however, often speculative in nature and subject to uncertainty. Occasionally, going still further, an increase in the value of all property in the flood plain due to future increased population and industrial activity has been assumed, fixing this as at mid-life of the flood-control project. This projection of money values into the future magnifies the flood benefits and is really using a factor representing future estimated increased values of property. It is therefore even more speculative and uncertain.

To allow the elimination of losses to prospective development in estimating flood benefits is certainly undesirable. The development of an area given flood protection usually depends upon other factors—such as

Table 8·6 Summary of Flood Losses in the United States

River or flood	Year	Drainage area, sq miles	Flood losses	Kind, direct = D total = T	Flood losses per sq mile of drainage area
1. Hepner, Oreg.	1902	20±	$ 250,000	D	$12,000
2. Bridgeport, Conn.	1905	25	250,000	D	10,000
3. Palouse, Wash. and Idaho	1910	2,980	1,250,000	D	416
4. Ohio River, total	1913	203,000	200,000,000	D	1,000
5. Miami River, Ohio	1913	5,430	115,000,000	T	21,000
6. Muskingum River, Ohio	1913	8,000	13,000,000	D	1,600
7. Scioto River, Ohio	1913	6,300	25,000,000	T	4,000
8. Arkansas River, Colo.	1921	2,670	19,000,000	D	7,100
9. San Antonio, Tex.	1921	90	7,000,000	T	77,000
10. Vermont (4 river systems)	1927	3,596	20,500,000	T	5,700
11. Winooski, Vt.	1927	1,076	13,200,000	T	12,300
12. New England	1927	33,000*	40,000,000	T	1,220
13. White, Ark. and Mo.	1927	28,000	15,700,000	D	560
14. Mississippi	1927	1,253,000	200,000,000	D	175
15. Alkali Canyon, Oreg.	1927	93	317,000	D	3,400
16. Cherry Creek, Colo.	1933	416	957,000	T	2,300
17. N. Y. State	1935	12,000	16,700,000	T	1,400
18. Texas	1935	57,000	28,500,000	D	500
19. Muskingum River, Ohio	1935	8,000	6,000,000	D	750
20. Republican, Nebr. and Kan.	1935	24,955	9,000,000	D	360
21. Fountain, Colo.	1935	940	1,500,000	D	1,600
22. Merrimack River, N. E.	1936	4,400	35,000,000	T	8,000
23. Connecticut River, N. E.	1936	10,600	67,000,000	T	6,300
24. Susquehanna River, Pa.	1936	29,000	60,000,000	T	2,080
25. Potomac River, Va.	1936	9,257	16,000,000	T	1,730
26. Ohio River, total	1936	203,000	400,000,000	T	1,970
27. Concho, Tex.	1936	6,500	2,200,000	D	337
28. Ohio River, total	1937	203,000	600,000,000	T	2,960
29. Northern California	1937	90,000	15,000,000	D	170
30. Los Angeles County, Calif.	1938	1,717	40,400,000	T	23,600
31. Fresno County, Calif.	1938	500	515,000	T	1,030

* Not including Maine.

Table 8·6 Summary of Flood Losses in the United States (*continued*)

River or flood	Year	Drainage area, sq miles	Flood losses	Kind, direct = D total = T	Flood losses per sq mile of drainage area
New England:	1938				
32. Connecticut		11,100	$ 46,100,000	T	$ 4,150
33. Millers.		315	6,700,000	T	21,400
34. Deerfield		465	7,400,000	T	16,000
35. Chicopee		721	8,600,000	T	11,800
36. Merrimack		4,400	35,000,000	T	8,000
37. Thames		1,400	10,000,000	T	7,100
38. Housatonic		1,600	4,000,000	T	2,500
39. Mississippi . . .	1945	1,253,000	100,000,000	T	125
Totals	. . . . . .	3,481,566†	$2,187,039,000		($630)

† Total without duplication = 1,669,663 sq miles.

the general economic conditions and demand for land in that particular area—factors that may easily prevent development, regardless of flood elimination or relief. Hence land-enhancement value should not be combined in any manner with flood losses to obtain a total benefit.

Summary of Flood Losses

In Table 8·6 is given a summary of flood losses, with drainage areas upon which these occurred and the corresponding losses per square mile. Most of these represent approximate total losses, although a few of them, as indicated, are direct losses only.

Table 8·7 Approximate Usual Limiting Flood Losses as Related to Drainage Area

Drainage area, sq miles (1)	Flood losses per sq mile (2)	Corresponding total flood losses (3)
200	$25,000	$ 5,000,000
500	19,000	9,500,000
1,000	15,000	15,000,000
5,000	9,000	45,000,000
10,000	7,500	75,000,000
50,000	4,500	225,000,000
100,000	3,500	350,000,000
200,000	3,000	600,000,000

Figure 8·2 shows by logarithmic plot prepared from Table 8·6 the relation between drainage area and flood losses per square mile, with an approximate limiting line. This shows, as in Table 8·7, approximate relations.

The limiting line in Fig. 8·2 is fairly well defined and generally consistent with all points, except No. 5, for Miami River, 1913; No. 9 for San Antonio,

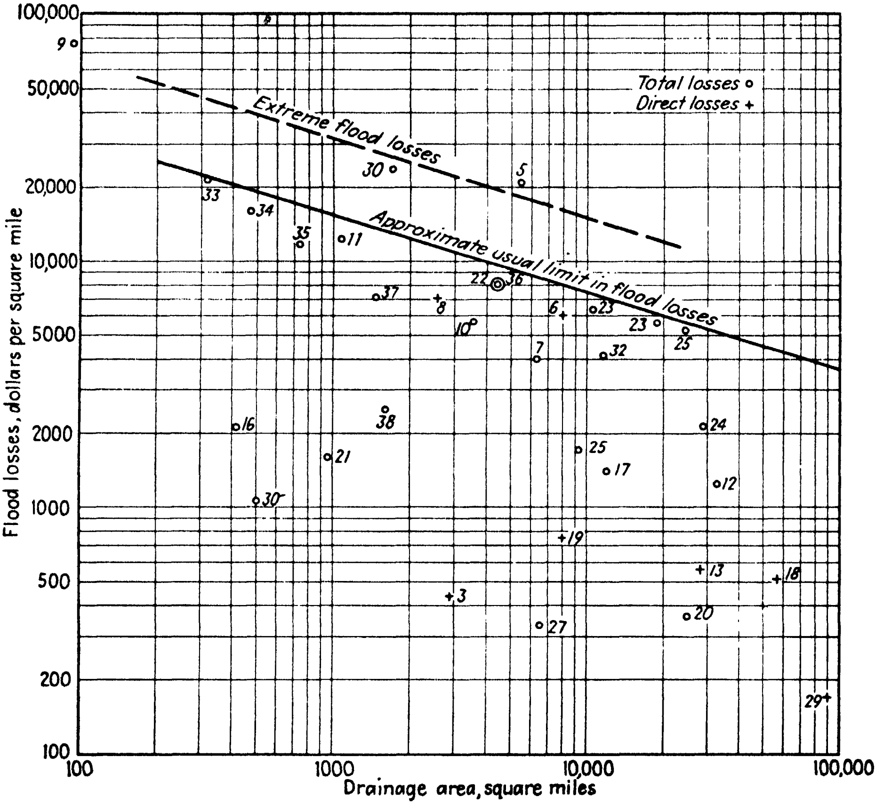


FIG. 8·2. Flood losses per square mile/drainage area in the United States.

Tex., in 1921; and No. 30 for Los Angeles, in 1938, which show about twice the flood losses as compared with the limiting line, indicating the exceptional losses in these floods due to their occurrence in highly developed industrial and urban areas.

As an indication of usual limiting flood losses per square mile of drainage area, for areas exceeding about 100 sq miles, the data in Table 8·7 appear to be well representative of general conditions in the United States, as shown by past experience, except in a few highly developed urban areas that are especially vulnerable to floods and where, as shown, flood losses may be about double these amounts.

Note also that the lower limiting drainage area in Fig. 8·2 is 100 sq miles. Smaller drainage areas or small portions of larger areas may show much larger unit flood losses, as for illustration those upon certain streams in the New York flood of 1935 (see Chap. 7, page 118), which show the following:

Stream or point	Drainage area, sq miles	Flood losses per sq mile
Glen Brook	5	$252,000
Watkins Glen	21	120,000
Montauk Falls	13	45,000

Forms for Estimates of Flood Damage. In Appendix B are forms prepared in 1937 and used for the estimation of flood damages based upon damage surveys as used by the Ohio River division of the U.S. Engineers, with headquarters at Cincinnati. These include a summary form followed by nine forms for listing different classes of damage. All were supplied through the courtesy of the division office.

CHAPTER 9

RESERVOIRS FOR FLOOD CONTROL

River-flow Characteristics

The flow of most rivers is very irregular and varies greatly in the different seasons of the year. A few examples of typical river-flow characteristics are given in Table 9·1 for a group of 25 river stations distributed over the United States, with summarized results in csm as follows:

Mean flow	1.03, ranging from	1.82	to 0.14 csm
Minimum flow	0.08, ranging from	0.50	to 0.00 csm
Maximum flow	33.0, ranging from	132	to 1.1 csm

The minimum flow averages 0.12 and the maximum flow 32 times the mean flow. The average maximum flow is therefore about 265 times that of the average minimum. Individual streams show a much wider variation between maximum and minimum, among which may be noted

$$\text{Los Angeles} \quad \frac{132}{0} = \infty$$

$$\text{Miami} \quad \frac{100}{0.05} = 2{,}000$$

$$\text{Westfield} \quad \frac{112}{0.2} = 560$$

$$\text{Potomac} \quad \frac{50}{0.1} = 500$$

Functions of Reservoirs

Reservoirs are useful in regulating the seasonal flow of rivers—storing flood flows that may later be released and thus increase low-water flows. Storage for seasonal use may be distinguished from pondage, as at a hydroelectric power development where relatively smaller amounts of water are held back at the dam during periods of the day or week to correlate the use of water in the power plant with load requirements.

Reservoirs may also be constructed for the single purpose of flood control where conditions make this economically feasible or desirable. In this case, floodwaters are held back temporarily to prevent or lessen floods in the river below and are gradually released after the flood period.

Table 9·1 River Characteristics of Flow

River	Location	Drainage area, sq miles	Flow, csm Mean	Flow, csm Approx. min.	Flow, csm Max. flood	Ratio to Mean flow Min.	Ratio to Mean flow Max. flood
Connecticut	Montague, Mass.	7,865	1.72	0.10	30.0	0.05	17.3
Westfield	Westfield, Mass.	497	1.82	0.20	112.0	0.11	61.0
Hudson	Mechanicville, N. Y.	4,500	1.63	0.20	26.8	0.12	16.3
Delaware	Trenton, N. J.	6,780	1.70	0.30	33.3	0.18	19.7
Susquehanna	Harrisburg, Pa.	24,100	1.42	0.07	30.8	0.05	21.7
Potomac	Point of Rocks, Md.	9,651	0.98	0.10	50.0	0.10	51.0
Rappahannock	Fredericksburg, Va.	1,599	1.04	0.003	88.0	0.003	85.0
Chattahoochee	Columbus, Ga.	4,670	1.37	0.10	18.2	0.07	13.4
Miami	Dayton, Ohio	2,513	0.92	0.05	100.0	0.06	110.0
Tennessee	Chattanooga, Tenn.	21,400	1.80	0.40	19.2	0.22	10.7
Allegheny	Franklin, Pa.	5,982	1.62	0.06	32.0	0.04	19.7
Ohio	Metropolis, Ill.	203,000	1.2±	0.12	8.8	0.10	7.2
Meramec	Eureka, Mo.	3,788	0.78	0.05	18.5	0.06	23.8
Mississippi	Vicksburg, Miss.	1,144,500	0.44	0.10	1.7	0.23	4.0
Arkansas	Van Buren, Ark.	150,300	0.20	0.002	5.6	0.001	28.0
White	DeValls Bluff, Ark.	23,430	1.00	0.50	6.5	0.50	6.5
Red	Shreveport, La.	59,300	0.39	0.013	4.0	0.02	10.2
Colorado	Lees Ferry, Ariz.	107,900	0.16	0.01	2.0	0.07	12.5
Feather	Oroville, Calif.	3,611	1.63	0.01	63.5	0.05	39.0
Sacramento	Kennett, Calif.	6,600	1.03	0.35	28.0	0.34	29.3
Los Angeles	Los Angeles, Calif.	510	0.14	0.00	132.0	0.00	940.0
Columbia	Grand Coulee, Wash.	74,100	1.42	0.20	6.6	0.14	4.6
Columbia	The Dalles	237,000	0.82	0.15	4.9	0.18	6.0
Snake	Murphy, Idaho	41,900	0.25	0.10	1.1	0.40	4.4
Bear	Evanston, Wyo.	645	0.36	0.00	5.7	0.00	15.8
Mean (25 stations)			1.03	0.08	33.0	0.12	32.0
Range—maximum			1.82	0.50	132.0	0.50	940.0
Range—minimum			0.14	0.00	1.1	0.00	4.0

Generally speaking, however, from the broad viewpoint of economy, it is more desirable to utilize multiple-purpose reservoirs, conserving river flow by this means for power, irrigation, or water supply, with a by-product of flood control, rather than to limit their functions to flood control alone; and construction for the conservation of water resources where practicable is preferable to that for the single end of flood control, particularly where a relatively high degree of storage, as 12 to 15 in. yearly, can be attained at reasonable cost.

Dam and Reservoir Sites

Conditions favoring a good reservoir site include first of all a good dam site, where the foundation material and topography permit a dam to be built at reasonable cost. If the river valley above the dam site widens out into a large basin and the river slope is gradual, it will be possible to store large volumes of water. Streams with steep slopes usually provide few reservoir sites of adequate size; and, on the other hand, large rivers with flat slopes may have only dam sites that are so wide as to prohibit their use, because of the size and cost of the required dam.

Interference with existing structures and other facilities, as towns and cities, railroads, main highways, and industrial plants, may also hinder or prevent the use of a reservoir site that is otherwise favorable.

Characteristics of Dam Sites. As a basis for preliminary estimates of quantities of materials required for dams, the following formulas are useful, where Y = cubic yards of material per linear foot of dam, h = height of dam to ordinary water level at any given section[1] and H = the surcharge in feet on spillway section.

Type of dam	Abutment or bulkhead section	Spillway section
Solid concrete	$Y = \frac{h^2}{66}$ (1)	$Y = \frac{h^2 + 2Hh}{66}$ (2)
Hollow concrete	$Y = \frac{h^2}{180}$ (3)	$Y = \frac{h^2 + 2Hh}{180}$ (4)
Earth	$Y = \frac{h^2}{9}$ (5)	

The formulas for concrete dams are based on heights up to about 150 ft. Somewhat greater quantities are required for greater heights, because of limitations of allowable masonry pressure. For earth dams, the formulas are based on usual top widths and side slopes of 1 on 2.5 to 3.

Valley Cross Sections. The cross section of the valley at dam sites varies in shape with the character of the river and the topography of its valley. Thus, in a mountainous country, as in river canyons, the cross section may be approximately (1) triangular or (2) made up of halves of two *parabolas* concave downward and tangent to each other at the river, with the two vertical axes at each side at the location of the top of the proposed dam. Farther down the river or in less rugged country, the cross section may approximate (3) a single parabola concave *upward*, with a vertical axis at the river. In relatively flat country, the cross section may in extreme cases be about (4) rectangular or (5) approximately trapezoidal, where, for instance, a dam is being built in a wide, flat valley.

Based upon the formulas previously given and assuming H as the maximum height of dam, it will be seen that the content of masonry in the dam will always be less than that based upon the $H \times L$ section, since the average height of the dam is less than H.

[1] Barrows, H. K.: "Water Power Engineering," 3d ed., pp. 336 and 348.

Theoretical Yardage Factors. For the various theoretical valley cross sections noted previously, these ratios to be applied in quantities computed by the formulas from the $H + L$ section, or what may be called the "yardage factor," are as follows:

Section	Yardage Factor, or $\frac{\text{Actual Cu Yd}}{\text{Cu Yd in } H \times L \text{ Section}}$
1. Triangular	0.33
2. Double parabola (concave downward)	0.20
3. Single parabola (concave upward)	0.53
4. Rectangular	0.37
5. Trapezoidal	0.56

For these theoretical sections, the yardage factor would vary from 0.2 to perhaps 0.3 in narrow valleys and from 0.4 to 0.6 in open country and would reach as high as 0.9 for very wide, flat valleys.

Actual Yardage Factors. In Table 9·2 are given data of dimensions and quantities of materials in 44 *earth dams*, of which 36 are based upon estimates and 8 upon construction figures; also 22 *concrete* dams, of which 16 are based upon estimates and 6 upon construction figures. In addition to dimensions and quantities, yardage ratios for the $H \times L$ section, as previously discussed, are given with the ratios of L/H.

The following conditions apply in Table 9·2.

For earth dams:

Column (4). Maximum height varies from 45 to 318 ft.

Column (5). Maximum length ranges from 500 to 4,800 ft.

Column (6). Actual or estimated cubic yards of earth fill run from 150,000 to 6,000,000 cu yd.

Column (7). Yardage of earth fill, based upon maximum height and length and the formula $Y = h^2/9$, and varying from about 270,000 to 14,000,000 cu yd.

Column (8). Yardage ratio, or Column (6)/Column (7), varying from 0.18 to 0.58.

Column (9). Ratio of H/L, varying from 3 to 70.

For concrete-masonry dams:

Column (4). Maximum height, 60 to 726 ft.

Column (5). Maximum length, 700 to 6,340 ft.

Column (6). Actual or estimated cubic yards of concrete masonry 14,000 to 10,500,000.

Column (7). Cubic yards of concrete based upon H/L section and $Y = h^2/66$; 32,000 to 69,300,000.

Column (8). Yardage ratio, 0.35 to 0.90.

Column (9). Ratio L/H — 2 to 88.

Table 9·2 Reservoir Characteristics—Quantities of Materials in Dams

No. (1)	Reservoir or site (2)	State (3)	Max. height of dam, ft $= H$ (4)	Max. length of dam, ft $= L$ (5)	Cu yd materials: Actual or estimated (6)	Cu yd materials: In $H \times L$ section (7)	Ratio $\frac{\text{Col. (6)}}{\text{Col. (7)}}$ (8)	Ratio $\frac{L}{H}$ or $\frac{\text{Col. (5)}}{\text{Col. (4)}}$ (9)
	Earth Dams							
1	Pittsburg*	N. H.	105	2,100	1,100,000	2,500,000	0.43	20
2	West Canaan	N. H.	70	500	150,000	273,000	0.55	70
3	Mad River	Vt.	125	2,000	1,100,000	3,500,000	0.31	16
4	Newfane	Vt.	130	1,800	1,100,000	3,300,000	0.33	14
5	Harriman*	Vt.	200	1,250	3,700,000	5,500,000	0 34	6
6	Mean of 20 (including 1–5)	Vt. and N. H.	45–200	500–2,500	....	....	0.38	(12)
7	Mean of 17	Wash., Oreg., Idaho, Wyo., etc.	60–170	700–4,800	....	....	0.47	(25)
8	Fort Loudoun Ext.*	TVA	100	3,120	2,035,000	3,500,000	0.58	31
9	Chatuge*	TVA	140	2,850	2,350,000	6,200,000	0.38	20
10	Nottely*	TVA	184	2,305	1,535,000	8,400,000	0.18	13
11	South Holston*	TVA	285	1,530	6,200,000	13,800,000	0.45	5
12	Watauga*	TVA	318	860	3,250,000	9,500,000	0.34	3
13	Blue Ridge*	TVA	167	1,000	1,500,000	3,100,000	0.48	6
14	Nantahala*	TVA	250	1,042	1,829,000	7,250,000	0.25	4
	Mean of 2 (9–10)	TVA	162	2,070	....	....	0.28	(16)
	Mean of 4 (11–14)	TVA	255	1,108	....	....	0.38	(4)
	Mean of 7 (8–14)	TVA	....	....	....	....	0.38	(12)

Concrete Dams

1	Gaysville	Vt.	185	700	127,000	363,000	0.35	4
2	Blackwater	N. H.	70	1,200	43,000	89,000	0.48	17
3	East Berkshire	Vt.	60	1,900	55,000	137,000	0.40	30
4	West Dummerston	Vt.	180	2,100	449,000	1,000,000	0.45	12
5	Westford	Vt.	55	700	13,600	32,200	0.42	13
6	Mean of 16 (including 1–5)	Vt. and N. H.	55–185	600–2,100	....	....	0.43	(11)
7	Wilson*	TVA	137	4,860	1,240,000	1,380,000	0.90	35
8	Wheeler*	TVA	72	6,340	630,000	700,000	0.90	88
9	Apalachia*	TVA	150	1,183	275,000	400,000	0.70	8
10	Hiwassee*	TVA	307	1,287	793,000	1,840,000	0.43	4
11	Norris*	TVA	265	1,860	1,022,000	1,960,000	0.51	7
12	Fontana*	TVA	480	2,330	2,800,000	8,000,000	0.34	5
13	Mean of 2 (7–8)	TVA	104	5,600	....	....	0.90	(54)
14	Mean of 4 (9–12)	TVA	300	1,660	....	....	0.49	6
15	Grand Coulee*	Columbia River	550	4,200	10,500,000	19,300,000	0.55	8
22	Marshall Ford*	Tex.	190	2,325	910,000	1,260,000	0.72	12
23	Central Valley*	Calif.	290	3,330	1,600,000	4,220,000	0.26	12
24	Stony Gorge* (Ambrose)	Calif.	142	868	43,000	97,000	0.45	6
25	Boulder*	Colorado River	726	1,282	3,250,000	10,200,000	0.31	2
26	Mean of 5 (15–25)	....	380	2,400	3,260,000	8,810,000	0.46 0.37	8

* Constructed.

Summarizing. A plot of yardage ratios against L/H shows no definite relation between these factors, with a variation of approximately 0.2 to 0.6 (except for a higher value in wide, flat valleys) in yardage ratio for both earth and concrete dams, and from 4 to 50 in values of H/L.

The general shape of valley cross section is of most importance, and the above range in values corresponds approximately to a triangular section as a minimum and a trapezoidal section (with base = $L/2$) as a maximum, including parabolic sections (concave upward) as an intermediate.

Table 9·2 will be found useful in approximate preliminary studies where only the maximum height and length of the dam are available.

Reservoir Characteristics

I. Reservoirs Constructed. In Table 9.3 are given data for 37 reservoirs that have been constructed at different sites in the United States. These are grouped into three divisions according to their use, *viz.*,

1. Multiple use (including usually power and flood control and in some cases navigation).
2. Power.
3. Flood control.

Referring to Table 9·3.

Columns (2) *and* (3). The designation of the reservoir or site as commonly used and the state in which located.

Column (4). Drainage area, 84 to about 40,000 sq miles.

Column (5). Maximum height of dam, 40 to 308 ft.

Column (6). Storage capacity, 32,000 to 5,825,000 acre-ft, the smaller reservoir in New Hampshire and the upper limit that of the large Denison Reservoir on the Red River in Texas. The Kentucky Reservoir of the TVA, with 5,000,000 acre-ft, is another of great size. Note that these are both for multiple use.

Column (7). Storage per square mile, 86 to 2,730 acre-ft, these being, respectively, for the Bagnall Reservoir, 1,200,000 acre-ft on a large river, 14,000 sq miles, and for Lake Almanor, 1,300,000 acre-ft, upon a relatively small river, 470 sq miles. These are both *power* reservoirs.

It is of interest to note the extent of storage capacity expressed as inches depth over the drainage area. This is related to acre-ft as follows:

Acre-ft per Sq Mile	In. Depth
53	1
265	5
530	10
1,060	20

The Lake Almanor Reservoir has a storage capacity of 51 in., or about 1.7 times the average yearly runoff of 30 in., an unusually high degree of storage, made possible by raising a large lake and utilizing especially favorable storage conditions. Bagnall Reservoir stores only 1.6 in., and the great Kentucky Reservoir of the TVA only 2.3 in., since they are both upon large rivers. These are both multiple-use reservoirs.

For the group of reservoirs, storage per square mile averages as follows:

	Acre-ft per sq mile	In.
A. Multiple use:		
1–7	655	12
1–5	860	16
B. Power	843	16
C. Flood control	312	6

It will be noted that reservoirs constructed for power use have been developed to a high degree of storage. In general, this is also true for multiple-use reservoirs, except upon large rivers, as may be seen by omitting Nos. 6 and 7, multiple use, which results in an average of 860 in place of 655 acre-ft per square mile. Flood-control storage is commonly less, and the average of 6 in. is typical.

Column (8). Cost—$800,000 to $105,000,000—these being, respectively, Brassua Lake, raised in level, and the great Kentucky Reservoir of the TVA.

Average cost by groups:

Type	Cost
A. Multiple use:	
1–7	$33,000,000
1–5	15,600,000
B. Power	4,350,000
C. Flood control	3,000,000

Column (9). Cost per acre-foot of storage—$1.07 to $60.00—which are, respectively, for Lake Almanor in California, an unusually low-cost reservoir highly developed, and Birch Hill Reservoir in Massachusetts, which required some 3 miles of railroad relocation at a cost of about $950,000.

By groups, average costs per acre foot of storage are as follows:

A. Multiple use	$21.90
B. Power	11.74
C. Flood control	28.20

The significant feature in this comparison is the relatively low cost of the power group because of the necessities of commercial use and the requirement of relatively low storage cost, which must be carried by power alone.

The group of seven reservoirs for power storage utilizes it through a

Table 9·3 Reservoir Characteristics

No.	Reservoir or site	State	Drainage area, sq miles	Max. height of dam	Storage capacity, acre-ft	Acre-ft per sq mile	Cost		
							Total	Per acre-ft	Per sq mile of drainage area
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	I. Reservoirs Constructed								
A	*Multiple Use*								
1	Nottely (TVA)	Tenn.	215	184	184,000	850	$ 5,270,000	$ 28.70	$ 24,500
2	Chatuge (TVA)	Tenn.	189	140	229,000	1,200	6,840,000	30.00	36,200
3	Hiwassee (TVA)	N. C.	977	308	365,000	372	12,580,000	34.40	12,900
4	Norfork	Ark.	1,765	231	1,983,000	1,120	27,500,000*	19.80	15,500
5	Norris (TVA)	Tenn.	2,950	265	2,281,000	770	25,800,000	11.30	8,750
6	Kentucky (TVA)	Tenn.	40,200	160	5,000,000	124	105,000,000†	21.00	2,600
7	Denison	Tex.	38,290	165	5,825,000‡	152	48,290,000	8.20	1,260
(7)	Total	. . . .	84,586	1,453	15,867,000	4,588	$231,280,000	$153.40	$101,710
	Mean	. . . .	12,080	208	2,270,000	655	33,040,000	21.90	14,530
B	*Power*								
1	Pittsburg	N. H.	84‡	100	100,000	1,200	$ 2,357,000	$23.60	$27,800
2	Stillwater	N. Y.	172	40	103,000	120	1,400,000	13.60	8,100
3	Harriman	Vt.	184	200	115,000	630	2,000,000‖	17.50	10,800
4	Brassua Lake	Me.	675	45	206,000	305	800,000	3.90	1,180
5	Sacandaga	N. Y.	1,044	80	870,000	830	12,000,000	13.80	11,500
6	Bagnall	Mo.	14,000	100	1,200,000	86	10,500,000	8.75	750
7	Lake Almanor	Calif.	477	100	1,300,000	2,730	1,400,000	1.07	2,920
(7)	Total	. . . .	16,836	665	3,894,000	5,901	$ 30,457,000	$82.22	$63,030
	Mean	. . . .	2,405	95	558,000	843	4,351,000	11.74	9,000
C	*Flood Control*								
	New England:								
1	Surry Mountain	N. H.	100	83	32,500	325	$ 1,620,000	$50.00	$16,200
2	Knightville	Mass.	164	160	39,300	240	2,318,000	59.00	14,100
3	Birch Hill	Mass.	175	56	50,000	286	3,005,000¶	60.00	17,100
4	Franklin	N. H.	1,000	136	170,000	170	7,883,000	46.50	7,880
(4)	Total	. . . .	1,439	435	291,800	1,021	$ 14,826,000	$215.50	$55,280
	Mean	. . . .	360	109	73,000	255	3,706,000	53.90	13,800

	Muskingum–Ohio:								
1	Seneca Fork	Ohio	84	60	65,000	770	$ 1,037,000	$ 11.90	$ 12,300
2	Sugar Creek	Ohio	300	53	71,700	238	3,046,000	43.00	10,000
3	Sandy Creek	Ohio	502	85	150,000	300	5,015,000	33.60	10,000
4	Tuscarawas	Ohio	1,397	74	203,000	146	5,360,000	26.40	3,850
5	Walhonding	Ohio	1,501	105	285,000	188	5,060,000	17.80	3,350
(14)	Total and mean	....	4,267	976	1,539,000	....	$ 33,347,000	$132.70	$ 39,500
	14 reservoirs, including 1–5	....	304	70	110,000	360	2,380,000	21.40	7,900
	Miami–Ohio—1916–1917:								
1	Lockington	Ohio	255	78	70,000	274	$ 1,780,000	$ 25.40	$ 6,950
2	Germantown	Ohio	270	110	106,000	393	1,835,000	17.30	6,800
3	Huffman	Ohio	671	73	168,000	250	7,639,000	45.40	11,300
4	Taylorsville	Ohio	878**	78	187,000	213	4,647,000	24.90	5,300
5	Englewood	Ohio	651	125	311,000	475	5,153,000	16.50	7,900
	Total	....	2,725	464	842,000	1,605	$ 21,054,000	$129.50	$ 38,200
(5)	Mean	....	545	93	168,000	321	4,211,000	26.00	7,640
	II. Reservoirs Proposed								
1	15 reservoirs with concrete dams	Vt. and N. H.	2,025 135	1,502 100	637,500 42,500	315	$ 27,000,000 1,800,000	$ 42.20	$ 13,300
2	20 reservoirs with earth dams	Vt. and N. H.	1,951 97	1,866 93	1,003,000 51,500	530	$ 32,000,000 1,600,000	$ 32.00	$ 16,300
3	Totals and means 35 reservoirs, 1 and 2	....	3,976 113	3,368 96	1,640,500 57,000	412	$ 59,000,000 1,700,000	$ 36.00	$ 15,000
4	Connecticut River flood-control projects, 20 reservoirs (1940)	Vt., N. H., and Mass.	2,752	2,140	940,000	340	$ 54,300,000	$ 58.00	$ 20,000
5	Ohio River flood-control, 74 reservoirs	Penn. to Ky.	203,000	....	16,298,000 220,000	80	$375,000,000 5,000,000	$ 23.00	$ 1,850
6	Ohio River above Pittsburgh, 10 reservoirs	Pa.	1,900	....	2,272,000 227,000	119	$ 71,000,000 7,100,000	$ 31.00	$ 3,720

NOTES: * Including $5,600,000 for power. † Including four power units. ‡ Including 2,000,000 acre-ft flood control and 1,000,000 silt storage. § Below first Conn. L. ‖ Power use, cost = ½ cost of dam. ¶ Including $959,000 for railroad relocation. **Not including area above Lockington Reservoir.

total head of approximately 2,300 ft, or an average of about 330 ft for each. The cost per acre-foot per foot of fall is thus $11.74/330 = 3½ cents. An acre-foot would give 1 cfs for 12 hr; if used through a head of 1 ft at 70 per cent over-all efficiency, this would be about ¾ kw for 12 hr, or 9 kwhr. The yearly cost would be about 0.10 × $0.035 = 3.5 mills, or about ⅓ cent per kilowatt-hour, as the cost of storage, which would be economical. It is obvious, however, that sufficient head downstream from the storage reservoir must be available to make a power reservoir commercially practicable.

Flood-control storage, where all cost is allocated to it, is greatest, with a cost per acre-foot of $28.20 in the foregoing group comparison; and multiple-use reservoirs, with a cost of $21.90, are intermediate in cost between power and flood-control reservoirs.

Column (10). Cost per square mile of drainage area. This is upon the basis of reservoir drainage area and does not include area below the reservoir which may be affected by its use.

II Reservoirs Proposed. In Table 9·3 are also compiled group data for *proposed* flood-control reservoirs, including 35 in Vermont and New Hampshire, 20 in the Connecticut River flood-control project, and 84 in the Ohio River Basin.

Referring to column (9), costs per acre-foot are as follows:

1. The 35 reservoirs in Vermont and New Hampshire:		
15 Concrete dams	$42	
20 Earth dams	32	
35	$36	
2. Connecticut River project.		
20 reservoirs	$54	
Mean of 1 and 2		$45
3. Ohio River.		
Main stream, 74 reservoirs	$23	
Above Pittsburgh, 10 reservoirs	$31	
Weighted mean		$25

Summarizing. The average weighted cost of the 168 flood-control reservoirs in Table 9·3, including constructed and proposed, is $30.80, with a range from about $60 to $12 per acre-foot, the higher costs being in New England. Costs are approximately those of 1935 to 1939, or prewar (200 per cent cost index), with the exception of the Miami Reservoirs built in 1916–1917, when the cost index was about 150 per cent. Note that the 1945 cost index was a little over 300 per cent, 1946 346 per cent, and 1947 about 407 per cent.

Variation in Cost of Reservoirs with Capacity

As has been previously noted, materials in and cost of the dam vary as the square of the height. The other usual items of cost may include land, clearing, railroad and highway relocations, and sometimes other structures or plants affected by flowage. These other items will generally vary in cost more nearly in proportion to the depth of the reservoir.

In Table 9·4 is given a classification of costs for five typical reservoirs that show the general make-up of cost items as follows:

Item	Per cent of cost	
	Mean	Range
1. Land and rights	30	10–55
2. Clearing	4	3–6
3. Highways and railroads	16	2–48
4. Dam and intake	50	22–75

As will be noted, the dam is usually the largest portion of cost; land and rights next, followed by the items of highways and railroads and of clearing. There is, however, much variation in the cost of these items depending upon circumstances and particularly upon the location of the reservoir with reference to the other works of man affected by its flowage. The unusual cost of item 3 in the case of Sacandaga Reservoir was due to prolonged litigation with the railroad company.

The cost of item 1 for Hiwassee Reservoir should include an additional amount for water rights, under litigation.

While the cost of a reservoir increases with its depth—all things considered as about the $3/2$ power of its depth—the storage capacity generally increases at a much greater ratio, perhaps about the $5/2$ power. The result is that the cost per acre-foot tends to decrease as the depth of reservoir (or height of dam) increases. This is clearly shown by the curves in Fig. 9·1, which are based upon reservoir-cost estimates for different heights of dam at Clendenin Reservoir site in the Kanawba River Basin, on the Elk River, made in 1932 by the U.S. Engineer Corps.

Details used as a basis for Fig. 9·1 are given in Table 9·5. Cost, height, and storage capacity are all plotted as a ratio to those values for the dam of lowest height, of 50 ft. Curve *A* shows total costs, curve *B* storage capacity, and curve *C* cost per acre-foot. Lines marked, respectively, first, second, and third power are given to show the approximate exponential rates of increase of the first two factors. It will be noted that cost varies about as the $1\frac{1}{2}$ to $1\frac{3}{4}$ power, while storage varies approximately as the $2\frac{3}{4}$ power of the height. This results in a *lessening* cost of storage per acre-foot as the reservoir increases in size, with a minimum value of about half that for the lowest dam.

Data for eight other similar studies of reservoir sites in the Ohio River Basin were available, and the Clendenin site appears to be fairly typical of these. The decrease in cost per acre-foot, as reservoir size increases, is important in the relative economy of multiple-purpose reservoir sites, developed to a high degree of storage—up to 12- or 15-in. depth or more. Smaller flood-control reservoirs, usually with 3 to 5 in. of storage, obviously tend to be more expensive per acre-foot of capacity. It should be further

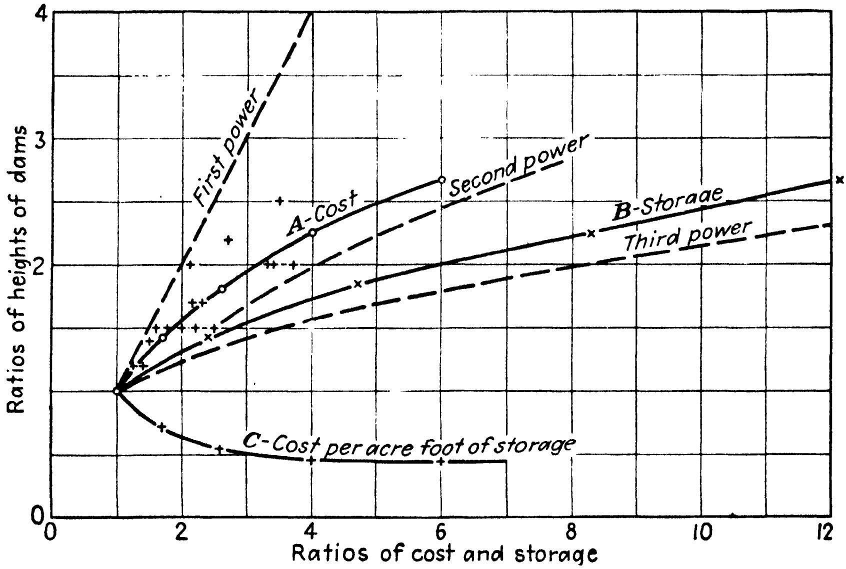


FIG. 9·1. Cost of reservoirs/height of dams.

noted that Clendenin Reservoir as tentatively recommended for construction had a 100-ft dam with a storage capacity of 161,000 acre-ft at an estimated cost of about $6,500,000.

Operation of Storage Reservoirs

Flood-control Reservoirs. Storage reservoirs for flood protection must be capable of operation so that storage capacity for floodwaters will be available when needed. Where great floods are possible at any season of the year, this may mean a retarding-basin type of reservoir, where, except during the passage of floodwaters, the reservoir is kept substantially empty and its storage capacity kept available for flood storage at all times. Obviously, such a reservoir serves no other purpose than flood control and must depend upon the benefits of flood relief for its justification.

Power Reservoirs. The operation of power reservoirs may be based upon two general methods, which are presented in the following tables:

Table 9·4 Reservoirs—Classification of Costs

Items	Hiwassee, TVA	%	Nottely, TVA	%	Chatuge, TVA	%	Norris, TVA	%	Sacandaga, N. Y.	%	Mean, %
1. Land and rights	$ 1,260,000	10	$1,750,000	33	$3,763,000	55	$ 8,600,000	33	$ 2,820,000	24	30
2. Clearing	410,000	3	232,000	4	194,000	3	....	....	720,000	6	4
3. Highways and railroads	1,535,000	12	113,000	2	11,000	....	4,457,000	17	5,780,000	48	16
4. Dam and intake	9,375,000	75	3,175,000	61	2,872,000	42	12,743,000	50	2,650,000	22	50
Total	$12,580,000	100	$5,270,000	100	$6,840,000	100	$25,800,000	100	$12,000,000	100	100

Table 9·5 Cost of Clendenin Reservoir on the Elk River, Kanawba River Basin, Based upon Estimate of U.S. Engineer's Office, Louisville, Ky., September, 1932

(Drainage Area 1,152 Sq Miles)

El., ft	Max. Height of dam, ft	Storage capacity		Cost		Ratios			
		Acre-ft	Acre-ft per sq mile	Total	Per acre-ft	Height, Col. (2)	Storage, Col. (3)	Cost, Col. (4)	Cost per acre-ft Col. (5)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
600	...	0	...	...	...	...	...	...	...
650	50	35,000	30	$ 2,500,000	$71.50	1.0	1.0	1.0	1.0
675	75	85,000	74	4,000,000	47.00	1.5	2.4	1.6	0.66
700	100	160,000	140	6,500,000	40.50	2.0	4.6	2.6	0.57
725	125	285,000	246	10,000,000	35.20	2.5	8.2	4.0	0.49
750	150	450,000	390	15,000,000	34.40	3.0	12.8	6.2	0.48

1. To utilize, as far as possible, the flow of the river, water is stored beyond the needs of plant capacity during the high-water season and this stored water is used during the drier seasons to maintain the power output at such times. For a typical situation in many parts of the United States, this means a reservoir substantially empty at the beginning of spring floods, usually in March, a full reservoir by May or June, and a gradual lowering of level until the next March. This means using the reservoir capacity each year for larger reservoirs; and sometimes, with small reservoirs, they may be filled in part or as a whole more than once during the year. The desired result is a maximum of kilowatt-hours of output. The use of storage capacity as affected by its relative amount in New England is about as follows:[1]

Storage capacity above regulation point		Yearly use of storage capacity as a proportion of storage used during average year
In. depth	Acre-ft per sq mile	
2	107	1.20
4	213	1.00
6	320	0.85
8	427	0.75
10	533	0.65
12	640	0.60

This may be called the "yearly-use" method of storage operation.

2. It may sometimes be desired to maintain a maximum amount of primary power by using stored water. This means adopting what may be called the "dependable-flow" method of storage operation whereby a certain primary flow is not to be exceeded at any time, when drawing from storage capacity. This primary, or dependable, flow may be approximated by the results of studies of previous years of recorded flow and checked up as time goes on by the results of actual storage operation. A good example of this kind of storage operation is the reservoir system above Berlin, N.H., on the Androscoggin River in New England, which is operated so as to maintain a minimum flow of 1,550 cfs at Berlin. Obviously it contemplates in a drought of long duration the storage of water over more than one season in order to maintain the dependable flow.

This method of storage operation will not give as many kilowatt-hours of total power as that described in (1) but will maintain a greater primary

[1] Barrows, H. K.: "Water Power Engineering," 3d ed., p. 161.

output, since more water is likely to be wasted over the reservoir spillway. It is not now, however, as common a method of storage use as the "yearly-use" method.

Power Reservoirs and Flood Control. When power reservoirs are located in parts of the country where major floods are practically limited to a certain portion of the yearly season, their use may be harmonious with needs for flood control, and the normal operation for power by the "yearly-use" method may afford as a by-product a large measure of flood control.

Thus, in New England, the major floods have been practically limited to late fall when a good portion of power storage capacity is available and early spring, when practically the full storage capacity is available. In the Tennessee River region, a similar situation also exists, since the season of major floods is confined to the period December to April of the yearly season.

This harmony of power and flood-control operation and the obtaining of the latter as a by-product of the former, which exists in many localities, is a most important aspect of the greater use of multiple-purpose reservoirs. In the interests of true economy, it should be availed of whenever possible.

Power and Navigation. The purposes of reservoir operation for power and navigation are entirely harmonious. Like power, navigation benefits from storage occur when floodwaters are stored and later, in the dry season, released to build up low-water flow. This is the same cycle of operation pursued in the "yearly-use" method of power operation.

Irrigation. This again is a seasonal use of water similar to power operation in regimen. It is consequently in harmony with power and navigation requirements and may be combined with flood control as a by-product if, as with power reservoirs, major floods are limited to a portion of the year.

Multiple-purpose Reservoirs

The foregoing discussion has shown that there is a large degree of harmony in the operation of reservoirs for the different purposes of flood control, power, irrigation, and navigation. This accounts for the increasing number of what may be termed multiple-purpose reservoirs, with which two or more of these purposes may be achieved by one reservoir or system of reservoirs.

Table 9·3, page 168, concerns a group of seven multiple-purpose reservoirs—five of them being TVA. Note that these are generally developed up to an average of 655 acre-ft per sq mile (about 12 in.). Considering Nos. 1 to 5 and omitting the two upon very large rivers, storage averages

820 acre-ft per sq mile, or about 15 in. These are typical reservoirs of this kind, with a relatively high degree of storage capacity.

The group of power reservoirs in Table 9·3 includes two reservoirs that have been important in providing flood relief as a by-product of power storage.

Harriman Reservoir upon Deerfield River in southern Vermont was completed in 1923. At the time of the November, 1927, flood in Vermont,

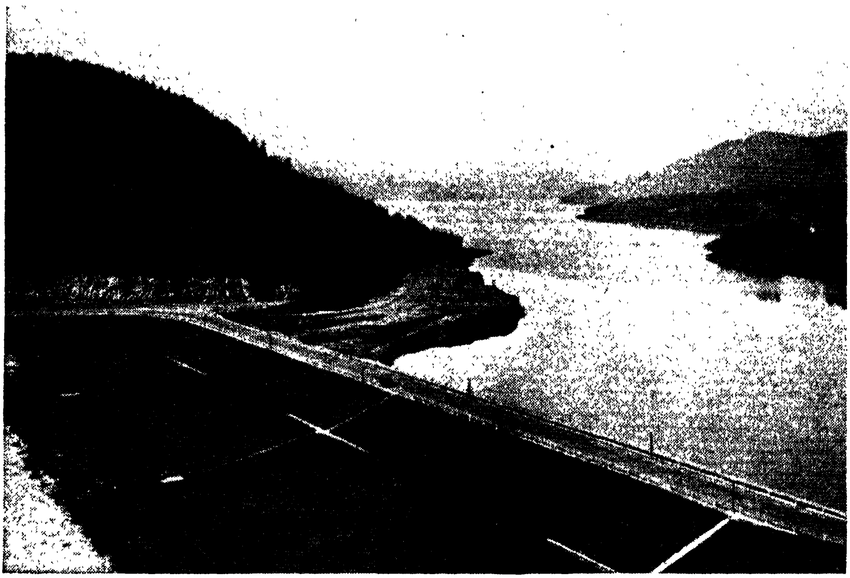


FIG. 9·2. Sacandaga Dam and Reservoir, Hudson River Basin, New York.

it was about one-third full and held back the flood. In March, 1936, the reservoir was fully drawn down and absorbed the flood. In September, 1938, it greatly diminished flood flow upon the Deerfield. It has an effective record of flood relief, all as a by-product of power storage.

Sacandaga Reservoir, primarily for power use, in the upper Hudson River Basin, was completed in 1931 and has been very effective in flood control. In the flood of March, 1936, it reduced the flow at Albany from about 270,000 cfs, which would have exceeded the largest flood of record, to about 220,000 cfs—a crest reduction of about 4 ft. In Fig. 9·2 is a view of this dam and reservoir.

The Wisconsin River. The Wisconsin Valley Improvement Co.,[1] organized in 1907, constructs and operates the reservoirs on the Wisconsin River,

[1] KYLER, M. W.: Storage Reservoir Capacity on the Wisconsin River to Be Doubled, *Civil Eng.*, October, 1946, pp. 455–456.

and its stockholders are the water-power users on the river. It now has about 400,000 acre-ft of storage capacity and has planned the new George Meade Reservoir at Dancy upon the Little Eau River, which will more than double this total storage. The new reservoir will be supplied chiefly by pumping floodwaters from the Wisconsin River with a pump capacity of 5,000 acre-ft. About a month's pumping will be required to fill it.

At present, 10 dams with an aggregate head of 236 ft use stored water. Other potential sites, with increased river flow, will bring the head to 320 ft with a generation of 92 million kwhr yearly, which with improved turbines at other sites can be made 110 million kwhr.

This is an excellent example of efficient power storage by a private corporation with attendant benefits of flood control and increased low-water flow, which has paid its way entirely by a local, state, and Federal taxpaying, investor-owned organization.

Multiple-purpose Reservoir Operation. The largest and most important group of multiple-purpose reservoirs now being operated is that upon the Tennessee River of the TVA. Method and routine of operation have been developed and will now be described[1] in some detail.

The drainage area of the Tennessee River is 41,000 sq miles with a mean yearly precipitation of about 52 in. In limited mountain areas in the headwaters, it is as high as 85 in. yearly. Storms that produce major floods come from the Gulf with a generally southwest-northeast storm track and occur from mid-December to about the first of April. *West Indian hurricanes* occurring between June 1 and Oct. 15 occasionally strike the northeasterly portion of the drainage area but do not produce major floods in the Tennessee River. These limitations as to time of occurrence of major floods are well established by upward of 100 years of records and are of much basic importance in the requirements for flood-control storage.

The Tennessee Valley system now includes 9 dams on the main river and 15 on tributary streams. There are also 6 hydroelectric plants of the Aluminum Co. of America on the Little Tennessee River that operate under TVA direction. Thus a total of 30 installations operate as one system, making the Tennessee River one of the most highly developed rivers of the world.

The combined gross storage capacity is 22 million acre-ft with 15 million acre-ft of usable storage; slack-water navigation is available, with a navigable depth of 9 ft for 650 miles from Paducah, Ky., on the Ohio River to Knoxville, Tenn.; the present hydroelectric installation at the 26 plants

[1] Blee, Clarence E.: Operation of Multiple Purpose Reservoirs on the Tennessee River System, *Civil Eng.*, 1945, p. 219; also Parker, T. B.: T.V.A. River Engineering, *Jour. Boston Soc. Civil Eng.*, January, 1944, p. 21.

is over 2 million kw under a total head below reservoirs of from 600 to 1,400 ft, and 450,000 kw of steam. Electric capacity is also available.

In Fig. 13·1, page 282, is a map of the Tennessee River drainage area showing the river system of the TVA; also profiles of the main river and important tributaries with plant locations and showing approximate heads at plants and distances.

Operation of the river system includes (1) long-range, or seasonal, control and (2) short-range, or day-to-day, control.

1. *Long-range Control.* This provides the proper reservation of space for flood control and determines the general method of operation for power production to meet the forecast power load. For flood control, a chart or rule curve is prepared for each reservoir showing limitation in filling throughout the flood season, except in controlling a flood. The limitation of the major flood season to a definite period, as previously noted, is of great importance in the matter of reserve space for floods.

In Fig. 9·3 are shown typical curves for multiple-purpose operation of Fontana Reservoir, of the Tennessee River system. As will be noted, the requirements call for the storage capacity of about 582,000 acre-ft (or 7.82 in. upon a drainage area of 1,443 sq miles) to be available on Mar. 15. Thereafter filling will be limited by available stream flow. The range between maximum and minimum levels during drawdown for the summer and fall seasons is also shown by curves.

The locality most vulnerable for flood damage in the Tennessee Valley is in the city of Chattanooga, where the largest flood of record occurred in 1867, which if uncontrolled and repeated would cause great damage. Hence operation of the reservoir system above Chattanooga is directed primarily toward its protection, although incidentally it benefits other localities on the Tennessee, Ohio, and Mississippi rivers below.

The total storage capacity available above Chattanooga, as of Mar. 15, is about 4.2 million acre-ft in tributary reservoirs and 0.8 million acre-ft in main-river reservoirs above Chattanooga—a total of about 5 million acre-ft, or about 4.5 in. of runoff from the total drainage area of 21,000 sq miles above Chattanooga. A considerably larger flood reservation is made as of Jan. 1 to provide against possible duplication of a major flood at a short interval. This does not, however, impose restrictions upon power production. There is sufficient storage to control effectively the largest floods of record at Chattanooga; and, with a system of levees at Chattanooga, it would protect against a considerably larger flood than that of record.

Filling of reservoirs after Mar. 15 is restricted until Apr. 15 at least, depending upon the condition of good weather, progress of the season and of

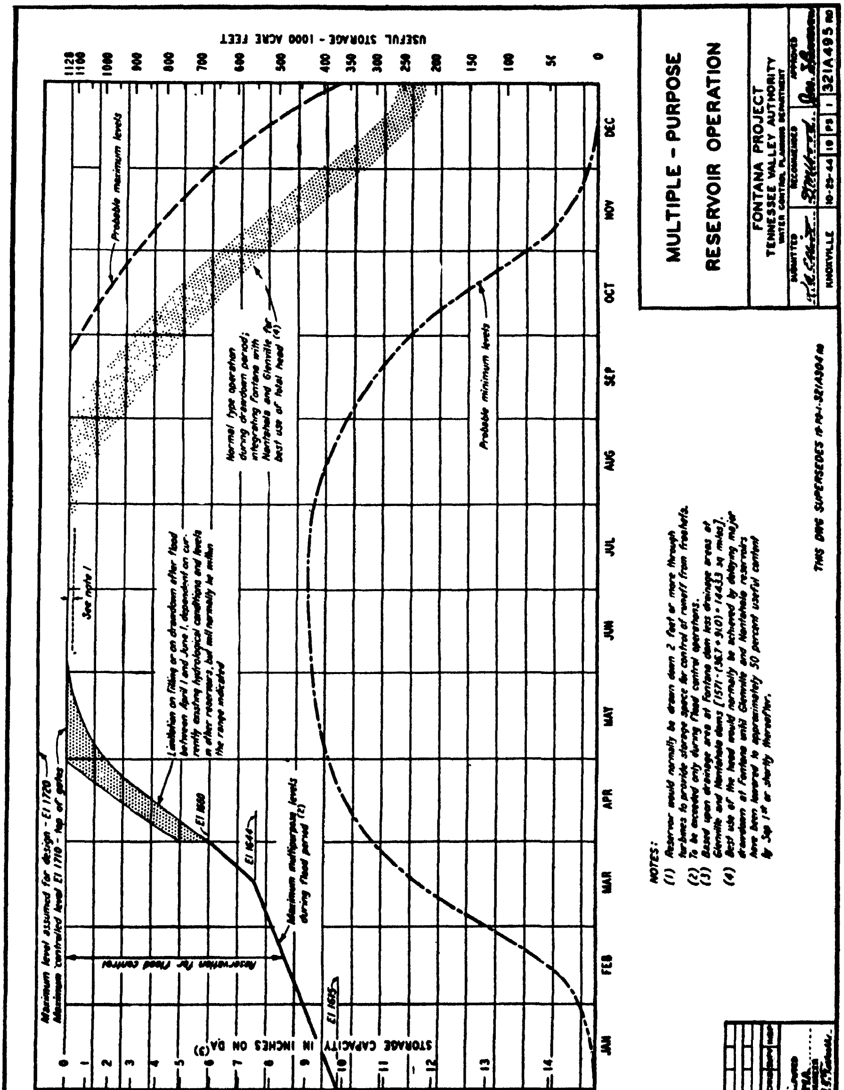


FIG. 9·3. Fontana Reservoir operation, TVA.

stream flow, and weather conditions prevailing during the last half of March. In other words, likely flood conditions for later March or early April are usually evident shortly after Mar. 15 and can be allowed for in lessening storage increases.

Below Chattanooga, reservoirs upon the main river have a combined flood-control storage of 5 million acre-ft, of which 4 million are behind Kentucky Dam practically at the mouth of the Tennessee River. This latter storage is of particular importance in controlling floods upon the Lower Ohio and Mississippi rivers and would have reduced the 1937 flood level at Cairo about 2 ft (see page 221).

Seasonal control for hydroelectric power production is complementary to flood-control operation, starting in where the latter leaves off. A master guide is prepared each year based upon the predicted power load or range for the year. This is an envelope of a number of curves showing required storage during certain critical dry years of record to carry the designated load with all steam plants at capacity.

In general, when stored energy is above this curve, no steam or purchased power is essential, subject, however, to questions of economy, best operation of different grades of steam plants, and the avoidance of wasting water if possible. These are largely matters of judgment, and the curves are only guides.

Data of actual loads, energy in storage, and stream-flow conditions are plotted as available, as a help to judgment. The master guide curve and operating record for the 1944 season are shown in Fig. 9·4. Near the bottom is a chart of average weekly flow in 1,000 cfs at Chattanooga at about the mid-point upon the drainage area (21,000 sq miles). Curves for 1925 and 1939, two years of low water, and for 1944 (to November) are shown. The top diagram shows the average weekly system load and its make-up of steam, as well as occasional small amounts of power from other systems. The approximate load is 1,400,000 kw, of which steam power varies from small amounts to about 300,000 kw. The middle chart in Fig. 9·4 is "stored-energy curves" of power reservoir storage for continuous loads of 1,400,000 and 1,300,000 kw as well as an actual curve for 1944 (to November).

In addition to the master guide curves, others are prepared for each reservoir to indicate the desirable distribution among the reservoirs. With storage toward the headwaters at high elevation, it is possible to carry the power load largely by the main-river plants, during the wet season, with the upper-tributary storage dams filling, and then to draw heavily upon storage in the dry season, utilizing released water at all downstream plants. Thus Fontana Reservoir water will ultimately be used in 13 plants with a combined gross head of about 1,400 ft.

Little in the way of special operating conditions is required for navigation, since its needs are harmonious with those of power operation.

Malaria control has developed as a necessary reservoir-operating procedure in the control or prevention of breeding of the malaria-bearing

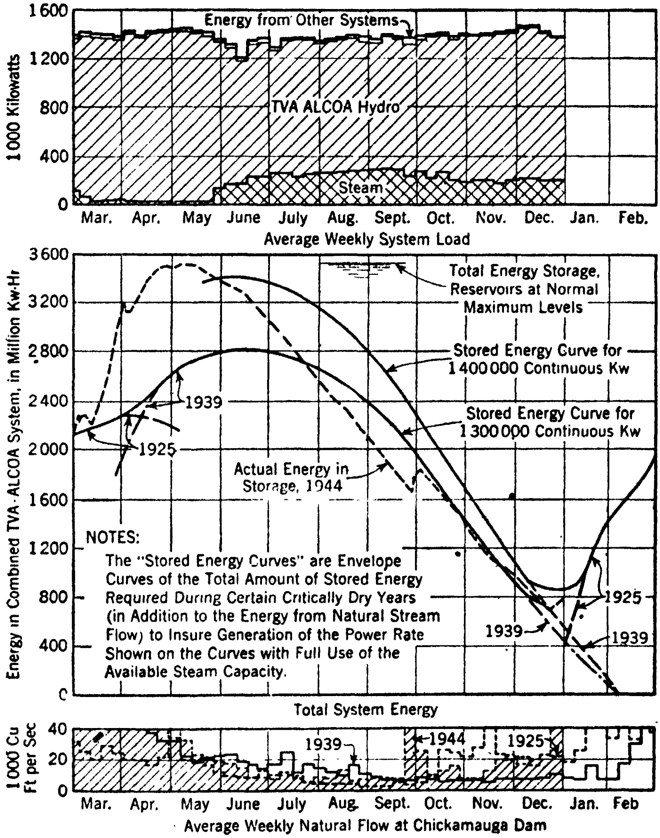


FIG. 9·4. TVA master guide and operating curves for 1944.

mosquito in and along the shores of the reservoirs. This is done by filling reservoirs to maximum elevation during early spring, holding water for a short period, and then drawing it to normal upper operating level, which tends to strand floating debris and inhibit vegetable growth around the margin of the reservoir. Then, during the mosquito-breeding season (about May 15 to Oct. 1), water levels are subjected to cyclical fluctuations of about 1 ft, at 1-week intervals, combined with a gradual water-level recession. This strands the eggs and larvae and draws them out from protective vegetation, exposing them to their natural enemies. Fluctuation of water level is not possible upon large reservoirs, where reliance must be upon extensive drawdown and clean shores.

2. *Short-range, or Day-to-day, Control of Reservoirs.* Basic data required for these operations include rapid transmission of rainfall and stream-gauge records from over the drainage area; weather predictions, ground-water and ground-surface conditions; curves of discharge for turbines, sluiceways, and spillways; storage capacities; time of wave travel, etc.

Water control is centralized in the office of the chief engineer at Knoxville under an engineer in the Water Control Planning Department. To supply information on rainfall and stream flow, an extensive network of stations is maintained; and 150 key stations normally report each morning and 90 more stations during flood periods. From each of the dams are transmitted through the Department of Power Operations reservoir elevations and rate of water discharge at 2-hr intervals. For remote locations, radio signals at 22 rain and 15 river gauges are used and found reliable.

The U.S. Weather Bureau cooperates by making a preliminary forecast each day at 8 A.M., followed by another at 11 A.M., giving weather predictions and rainfall expected for 3 to 5 days in advance. In case of an important storm, special forecasts are made at 6-hr intervals.

Daily information normally starts to come into the Knoxville office about 6 A.M. As rainfall records are received, they are written on an outline map with carbon copies. This information, in conjunction with preliminary weather forecasts, stream-flow data, and reservoir information, is grouped according to tributary drainage basins and is then analyzed by engineers assigned to specific parts of the system. Runoff is estimated for the current day and for 3 days in advance. It has been found preferable to base the primary forecasts of runoff on the rain that has actually fallen to date. These estimated runoffs are reviewed in connection with the amount of rainfall forecast, the storage existing in each reservoir, and the needs for power production. Then decisions are reached about noon as to how each reservoir shall be operated, the amount of water to be stored, the amount to be released through the turbines, and the amount, if any, to be wasted over the spillways or through the sluices. A daily bulletin is issued giving the current inflow, discharge, elevation, and the change in elevation from the previous day for each reservoir; the discharge, stage, and change of stage at key points on the main river and tributaries throughout the valley; and the forecast inflow, discharge, and elevation at each reservoir for 3 days in advance.

The most important factor in the successful operation of a multiple-purpose water-control system is that the system shall have been planned and designed for the method of operation to be carried out. Three different methods of approach or conception as to the design of multiple-purpose projects involving flood control are based on the thought (1) that a reservoir must at all times be kept empty for flood control; (2) that there must be a designated portion or definite layer of the reservoir reserved for flood control and another portion for other of the multiple uses with no common use of the same space; (3) that the same space in a reservoir may be used both for flood control and for other purposes at different seasons of the year.

Each of the above conceptions has its proper place, and each has been successfully and economically employed. The first is essentially the detention-basin type of development and can have little if any other use than for flood control. The reservoirs of the Miami Conservancy District in Ohio are an example of this type of flood-control project. In studying the flood-control problem of the upper French Broad River in the Tennessee Valley, the authority's engineers have tentatively proposed the use of retarding-basin-type reservoirs for such a project. Here the stream-flow distribution is not favorable for power development, economical sites for reservoirs of large capacity are not available, and the record indicates that major floods may be expected throughout the year.

The second method—that of reserving a definite portion or layer of the reservoir for flood control—has been used in the design of a number of multiple-purpose projects. The Norfork Dam, recently constructed in Arkansas by the Corps of Engineers, is an example of this method of approach to multiple-purpose operation. The planning of the Watauga and South Holston projects in the northeastern end of the Tennessee Valley is based on reserving exclusively for flood control the space above a certain fixed elevation in each reservoir. In these projects, the topography and the lack of development in the area are such that reservoirs of comparatively large storage capacity may be obtained at reasonable cost, but damaging floods may occur in any season of the year.

The third method—that of utilizing the same space in a reservoir for flood control during one season and for other purposes during another—is the method generally used as the basis of planning and design for the system of water control in the Tennessee Valley as now constructed. The following conditions favorable to such a method of multiple-purpose operation are present to a high degree in the Tennessee Valley:

1. Major floods consistently confined to a definite flood season.
2. Topographic conditions and extent of improvement in the reservoir areas such as to favor the production of relatively large storage capacities at moderate cost.
3. An annual runoff cycle such as to favor the required degree of filling of the reservoirs after the end of the flood season.

The operation of the multiple-purpose reservoir system in the Tennessee River Basin has now been carried on for several years, and during that time

Table 9·6 Characteristics of Reservoir Operation—Based upon Period 1935-1936

No.	Reservoir			Discharge area, sq miles	Capacity		Min. stage		Max. stage		Period of drawdown	
	Name	River	State		Acre-ft	Acre-ft per sq mile	% of capacity	Time	% of capacity	Time	Months	Manner of drawdown and remarks
						Power						
1	Harriman*	Deerfield	Vt.	182	116,000	635	8	Mar. 1	90	June 1	9	Fairly uniform—some rise Nov.-Dec
2	Union Water Power Co. system	Androscoggin	Me. and N. H.	1,095	685,000	625	21	Mar. 1	94	June 1	9	Uniform
3	Sacandaga	Sacandaga	N. Y.	1,044	867,000	830	16	Mar. 1	87	June 1	9	Uniform
4	Burton-Terrora	Tallulah	Ga.	151	139,500	930	28	Jan. 1	100	Apr. 1	9	Uniform 2 or 3 months, little drop or some rise during summer, then again uniform
5	Bartletts Ferry	Chattahoochee	Ga.	4,200	136,000	33	33	Nov. 1	100	Mar. 1	8	Similar to Burton-Terrora
6	Cle Elum	Yakima	Wash.	500	358,500	717	25	Oct. 1	100	June 1	4	Uniform—fills mostly Mar. to June
7	Lake Shannon Diabolo	Skagit	Wash.	2,500±	247,500	100±	29	Feb. 1	100	June 1	8	Uniform—except minor rise in Nov.-Dec.
8	Big Creek system	San Joaquin	Calif.	448	288,700	640	2	Mar. 1	100	July 1	8	Uniform
9	Lake Almanor	Feather	Calif.	479	1,358,000	2,850	23	Oct. 1	39	June 1	4	Uniform
						Irrigation						
10	Lake Walcott	Snake	Idaho	....	96,700	....	24	Oct. 1	100	June 1	4	Uniform—filling Nov. to June
11	American Falls	Snake	Idaho	....	1,700,000	....	3	Oct. 1	100	June 1	4	Uniform—filling steadily Nov. to June
12	Arrowrock	Boise	Idaho	....	276,500	....	1	Nov. 1	100+	June 1	5	Uniform—filling mostly Apr.-May

* Based upon period 1924–1929.

a number of moderately large floods have occurred and been successfully controlled. No very large floods have as yet been experienced.

Reservoir Operation—Power and Irrigation Reservoirs

In Table 9·6 are given data showing the method of use of water in nine power and three irrigation reservoirs in different sections of the United States. These show actual operation and cover the period of 2 years, 1935–1936, except for Harriman Reservoir in Vermont, which is for the period 1924–1929.

The group of three power reservoirs in New England and New York are similar in regimen, showing a minimum storage on Mar. 1, with fairly complete drawdown, of about 12 per cent for the yearly-use type (Nos. 1 and 3) and 21 per cent for No. 2 on the Androscoggin River, which, as previously noted, is operated upon the "dependable-power" basis. The two southern power reservoirs, Nos. 4 and 5, reflect an earlier time of minimum storage, in January and November, but about the same time of maximum storage—Apr. 1, and Mar. 1—for New England.

The western group of power reservoirs, Nos. 6 and 9, are more variable in time of minimum storage, since they occupy quite different time conditions of yearly rainfall, varying from Oct. 1 in Washington to Mar. 1 in California. The time of maximum storage is quite consistently June 1. Lake Almanor shows only 39 per cent of capacity as a maximum in 1935–1936, owing to its unusual degree of storage, which, as previously noted, is about 1.7 times the mean yearly runoff of 30 in.

The group of three irrigation reservoirs, all in Idaho, are consistent in showing a minimum storage in October or November and a maximum storage June 1. Note that the usual time of drawdown of power reservoirs is about 8 months of the year as compared with 4 months (the growing season) for irrigation reservoirs.

CHAPTER 10

MISSISSIPPI RIVER BASIN

The *Mississippi River* (see Fig. 10·1) rises in northern Minnesota and flows in a generally southerly direction for about 2,470 miles to the Head of Passes near the Gulf of Mexico. Next to the Amazon, it is the largest river basin of the world, with about 1,244,000 sq miles of drainage area and a mean yearly precipitation of about 30 in. In Table 10·1 are listed the principal tributary river basins, with their drainage areas and maximum flood discharges.

Table 10·1 Mississippi River Basin—Areas and Floods

River basins	Drainage area, sq miles	Per cent of total area	Maximum flood discharge	
			Cfs	Date
Ohio	204,000	16	1,800,000	February, 1937
Upper Mississippi	172,000	14	366,000	June, 1903
Missouri	529,000	43	589,000	June, 1903
Mississippi below mouth of Ohio	921,300			
Lower Mississippi Valley	42,700	5		
Arkansas, White, Red, Ouachita	280,000	22	1,500,000±	April, 1927
Total—lower Mississippi	322,700	27		
Total—Mississippi	1,244,000	100		

The actual maximum flows in cfs during February, 1937, at points upon the Lower Mississippi were as follows:

Memphis	1,980,000
Arkansas City	2,160,000
Vicksburg	2,080,000

The *Mississippi River Basin* includes about 41 per cent of the United States with a drainage area lying within 31 states. The outflows from the northerly part of the basin meet at the confluence of the Missouri and the

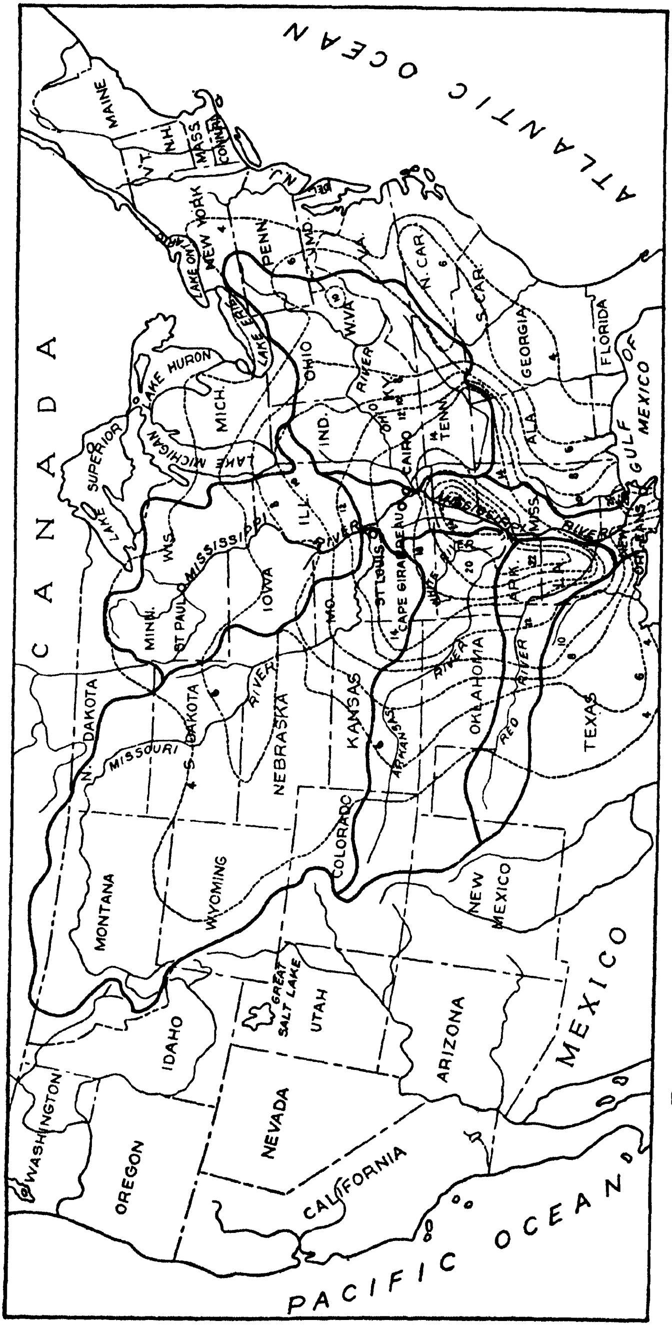


FIG. 10·1. Mississippi River drainage basin and precipitation for March and April, 1927.

Upper Mississippi just above the city of St. Louis. The discharge from the Ohio, the major eastern tributary, is added to the river at Cairo, 218 miles below St. Louis. From near Cairo to the Gulf of Mexico, the river flows through a wide and fertile plain known as the "alluvial valley of the Mississippi," which has originally covered frequently by river floods. About halfway between Cairo and the Gulf, the Arkansas and White rivers enter from the west, and the Red River enters somewhat lower down. All three may contribute to high floods.

The *Ohio River*, with 204,000 sq miles of drainage area and a mean yearly precipitation of about 44 in. flows through a comparatively narrow valley of upland territory for about 1,000 miles between Pittsburgh and Cairo. Riverbanks are high, and the area overflowed in ordinary floods is small. The Ohio Valley is one of the most extensively developed regions of the United States; and many cities, towns, industrial establishments, railroads, and highways occupy lands that are inundated by extreme floods. The valley is in the path of winter and spring storms and, as previously noted, is the principal contributor to floods upon the Lower Mississippi.

The *Missouri River* Basin, the largest tributary of the Mississippi, with 43 per cent of its total area and about 529,000 sq miles of drainage area, has a very diverse terrain, largely of plains but with much rugged area in its mountainous portion in the Rocky Mountains, is about 2,460 miles long lying in or along seven states, and enters the Mississippi River 17 miles above St. Louis. It has large areas of fertile soil, and agriculture is predominant, although many major industries and important railroad facilities are found at Sioux City, Kansas City, Kans., Kansas City, Mo., Omaha, and Council Bluffs. Its normal yearly precipitation is about 21 in.

Its floods commonly occur in March from melting snow above Sioux City or in June from snow thaws in the headwater mountains, accompanied by rainfall. It is to be noted, however, that Missouri floods have not yet synchronized with those of the Ohio, which come in winter or early spring.

The *Upper Mississippi* Basin, with a drainage area of 172,000 sq miles above St. Louis and a mean yearly precipitation of about 31 in., is developed for navigation between Minneapolis and the Ohio River by a series of 26 locks and dams. There are also six reservoirs in Minnesota for low-water navigation use.

The *Arkansas-White* Basin, with 189,000 sq miles and a mean yearly precipitation of about 30 in., extends to Colorado on the west and enters the Mississippi about midway between Memphis and Vicksburg. The Red River Basin, extending through Texas to eastern New Mexico, and the Atchafalaya, entering the Red River near its mouth, have a total area of 91,000 sq miles and a mean yearly precipitation of about 33 in. They enter the Mississippi near Angola. These streams, comprising about 22 per cent of the Mississippi Basin (or 7 per cent more than the Ohio), are important contributors to floods on the Lower Mississippi.

Although the Missouri River is the largest tributary of the Mississippi. its maximum flood discharge is only about one-third that of the Ohio. The latter basin as well as those grouped just previously, entering from the west. are the principal contributors to floods on the Lower Mississippi. It is also noteworthy that the largest floods upon the several tributaries have thus far not coincided in time.

Lower Mississippi River[1]

The Lower Mississippi Basin (see Fig. 10·2) includes the portion of the river between St. Louis and its mouth below New Orleans with an intervening drainage area of about 339,000 sq miles in which are tributary the St. Francis, White, Arkansas, Ouachita, Red, and Atchafalaya basins on the west and the Yazoo on the east. The drainage area directly included in the main valley of the Lower Mississippi is about 59,000 sq miles.

The runoff from 41 per cent of the area of the United States must pass to the sea through this alluvial valley of the Mississippi. At Arkansas City, river discharges vary between 86,000 and 2,500,000 cfs.

The riverbanks and beds are easily eroded, and wide variations in discharge and lack of cohesion have created a meandering stream of constantly changing alignment. Large floods could not be carried by the river within its banks and would spread all over the valley, but maximum floods did not greatly exceed full-bank stages.

Overbank stages came too late in the growing season and lasted too long to permit the development of a dependable agriculture. Hence the early settlers constructed levees along the banks of the river to prevent overflow of their lands. But, as these low local levees were extended and connected, the river was deprived of valley storage, overbank stages were raised, and the levees were overtopped and crevassed. However, the levee system has been gradually extended, raised, and strengthened, and has now become the object of Federal action.

The first levees on the lower Mississippi were built by the French in 1717 and were gradually extended under the provisions of land grants, which required the grantee to construct and maintain a levee line along the river front of his property.

Floods in 1849 and 1850, however, resulted in widespread damage, and pleas were made for Federal aid, which granted the several states all unsold swamp and overflowed lands within their limits to provide funds for reclaiming overflowed districts.

[1] H. R. Doc. 509, 78th Cong., 2d Sess., 1944; also H. R. Doc. 359, 77th Cong., 1st Sess., 1941.

The surveys of the Mississippi River by Humphreys and Abbott were also authorized at this time. Ellet in 1851 also made an extensive report upon Mississippi River floods, and it is of interest to note that he included headwater reservoirs as an essential part of his plan for flood relief.

The Mississippi River Commission was formed in 1879, consisting of seven commissioners—three from the U.S. Corps of Engineers, one from the U.S. Coast and Geodetic Survey, and three (including two civil engineers) from civil life. This commission exercised direct control of the Lower Mississippi with the aid of the U.S. Engineers until 1935, when Congress placed it under the control of the U.S. War Department.

Between 1890 and 1917, periodic appropriations were made for levees, revetments, and dredging. The Flood Control Act of Mar. 1, 1917, made flood control a definite part of the work. This act required local interests to provide necessary rights of way, pay not less than one-third the cost of levee construction, and assume the entire maintenance cost. Revetment and dredging work was to be paid for entirely from Federal funds.

In 1927, the Lower Mississippi River experienced the highest flood of record to that time (see Fig. 10·1 for rainfall distribution in March and April, 1927). More than 200 breaches in the levees occurred, with few, however, in levees built by the commission and only one in a levee built to the commission's standard of dimensions. About 20,000 sq miles of valley was submerged and 250 lives lost.

The Flood Control Act of May 15, 1928, based upon the report of the late General Jadwin, provided for the control of floodwaters in the alluvial valley of the Mississippi from Rock Island, Ill., to the Head of Passes near the Gulf of Mexico, by means of levees—including tributaries affected by the main river. (The alluvial valley of the Mississippi also includes the St. Francis, Yazoo, Tenses, and Atchafalaya basins and the alluvial lands around Lake Ponchartrain.)

The plan included raising existing levees by 3 ft, strengthening them, and permitting flood waters in excess of the safe capacity of the levee channel to find their way to the Gulf through the natural floodways in the lowlands adjacent to the river which have always carried the waters of extraordinary floods. These proposed floodings included the Boeuf on the west side of the main river approximately northwest of Vicksburg and the Atchafalaya also on the westerly side of the river about opposite Baton Rouge.

An appropriation of $325,000,000 was authorized by the Act of 1928 and provided for local cooperation by the states or levee districts, in maintaining the flood-control works except the controlling and regulating structures, after their completion, also in providing necessary rights of way without cost to the United States and a contribution of one-third the cost of the

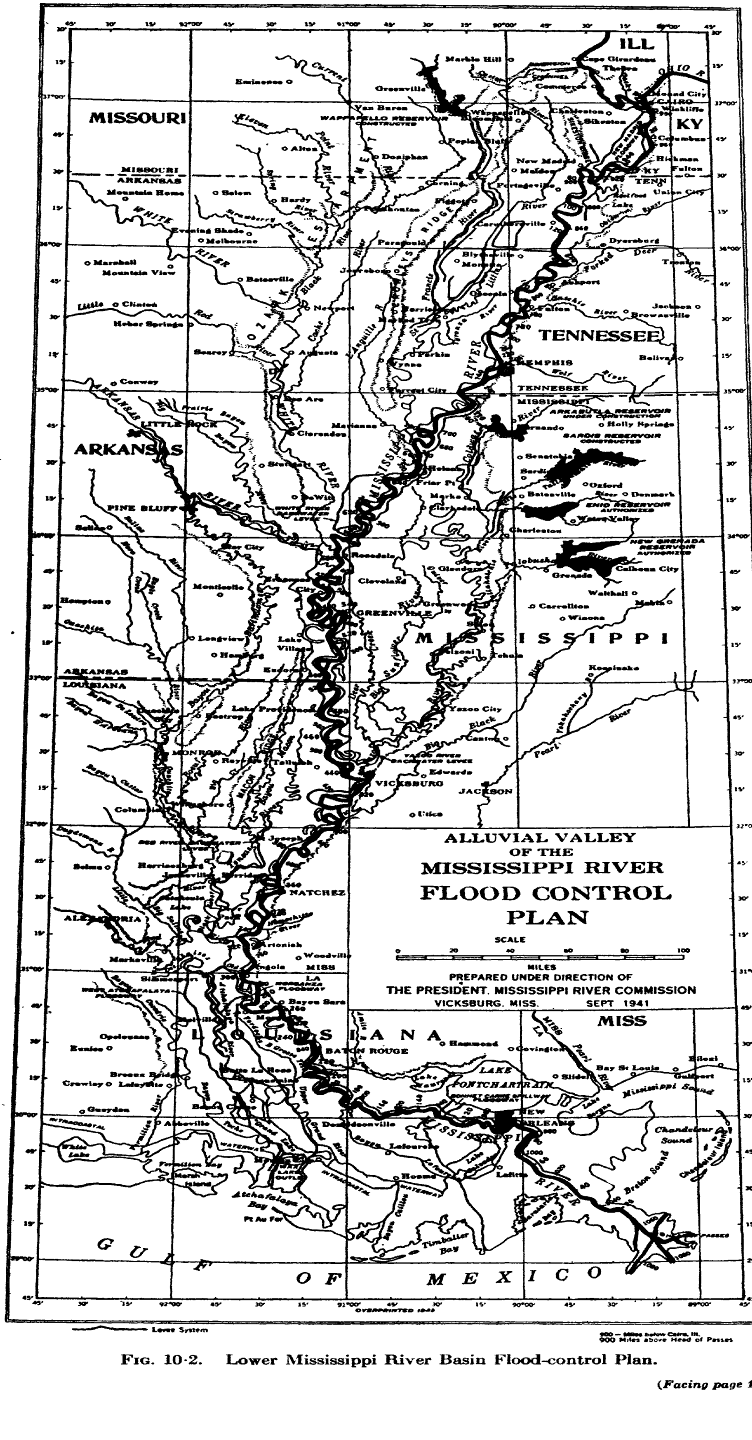


Fig. 10·2. Lower Mississippi River Basin Flood-control Plan.

(*Facing page 190*)

project work between Rock Island, Ill., and Cape Girardeau, Mo. The Act of 1934 provided for reimbursement for the local cooperation in respect to cost of rights of way and easements.

The Flood Control Act of June 15, 1936, adopted the following modification of the general project: (See Fig. 10·2.)

(1) The abandonment of the Boeuf floodway, and in lieu thereof the construction of the Eudora floodway (west of the Mississippi River, and extending from the latitude of Eudora, Ark., into the Red River backwater area), with a control structure at its head; (2) the construction of a rear-protection levee extending from the head of the Eudora floodway north to the Arkansas River, located so as to afford adequate space for the escape of floodwaters without endangering the levees on the east side of the river; (3) the maintenance of the present river levees between the head of the Eudora floodway and the northern junction with the protection levee, at the 1914 grade and section, except in front of densely populated areas, as a part of a ring levee; (4) the construction of a floodway extending from the Mississippi River north of Morganza, La., to the Atchafalaya River backwater, with a control structure at its head; (5) the raising of the levees from the head of this floodway to the head of the Atchafalaya River to full standard grade and section; (6) the immediate completion of the guide levees in the Atchafalaya Basin to afford full protection to all lands outside of these levees; (7) the construction of an additional outlet to the Gulf of Mexico, west of Berwick, La. (8) the increase of the discharge capacity of the leveed channel of the Atchafalaya River and of its outlets; (9) a 6-year program for the improvement and regularization of the Mississippi River, including the continued maintenance of the navigation channel provided in the previous project; and (10) the flood control of the St. Francis and Yazoo rivers.

It was contemplated that the unfinished parts of the levees and structures authorized by the Act of 1928 and not modified by the Act of June 15, 1936, would be completed as planned. The total estimated cost of all work added to the previous project, including the acquisition of reimbursement for land rights, was $313,000,000, or $245,000,000 in excess of the unappropriated balance of the appropriation previously authorized.

Existing Mississippi River Project

Since 1936, amendments to the acts for control of floods on the Mississippi River have been made in 1937, 1938, 1941, and 1944. An important feature of the Act of 1941 was the abandonment of the Eudora floodway project, which had become impracticable because of the stage lowerings produced by the cutoffs.

Until recently, the general project has provided for the maintenance of a channel for navigation 300 ft wide with a minimum depth of 9 ft between Cairo, Ill., and Baton Rouge, La. The Act of 1944 now provides authorization for a navigation channel 12 ft deep and 300 ft wide. It was also contemplated that the main river below Cairo, Ill., be improved in its flood-carrying capacity and for navigation, by securing better alignment and by the construction of stabilization works. In the last 10 years, the alignment of the river from mile 677 above Head of Passes to mile 338 above Head of Passes has been radically changed by cutting off long bends and by other dredging operations that have shortened the length of the low-water navigation channel between Memphis and Baton Rouge about 170 miles. Numerous troublesome crossings have been eliminated, and the reduced mileage has more than compensated for the swifter currents encountered at high stages by upstream navigation.

At the same time, the realignment of the river has reduced the heights of major floods by about 12 ft at Arkansas City and about 7 ft at Vicksburg with lesser reductions above Arkansas City and below Vicksburg. The straightening of the river has increased flood slopes and high-water current velocities.

Although the alignment of the river has been fixed at a number of bends by bank revetment and in some places by dikes, dredging is the main reliance for the maintenance of the project channel in the shifting river during low water. The channel is maintained by five pipe-line dredges in the Memphis district, two in the Vicksburg district, and two in the New Orleans district.

During the high-water seasons, the crossing bars in and immediately below unstable reaches generally shoal up as much as 5 to 15 ft above the heights that obtained during the prior low-water season. In addition to the shoaling in crossings, the deposition of materials from caving banks at a number of locations obliterates the low-water channel of the previous year.

Although the shoaled crossings scour generally to their former level, they act as broad-crested weirs over which the currents pass without sufficient concentration of flow to scour an adequate channel. Dredging is therefore necessary at the beginning of the low-water season to maintain the project channel through such shoaled locations.

The extent of dredging at a given location is largely dependent upon the stability of the reach upstream. If active bank or bar recession continues during the low-water season, immediately above a crossing, recurrent dredging is usually required after each minor rise and fall in stage.

For most of the 737 miles of river channel between Cairo and Baton Rouge, regular reconnaissance surveys are required during the low-water season. For most of this distance, some 210 crossings are regularly examined, of which 30 to 75 may require dredging one or more times during each low-water season. The greater portion of the crossings requiring dredging lie within the limits of the Memphis district, between Cairo and the mouth of the Arkansas River.

Beginning in 1933, an extensive program of channel-improvement dredging has been carried out, this being additional and supplemental to the excavation of cutoffs and also additional to the regular low-water maintenance dredging. Its purpose has been to secure a single and efficient channel of good alignment in reaches where needed and to improve indefinite location and inefficient hydraulic characteristics. The work has been cutting back projections in bank lines, making deep cuts on channel location, depositing dredged spoil in the form of sand bars or sand dikes to guide flow into the excavated channel, and closing chutes and back channels with dredged spoil. Each is a major undertaking. Some have been highly successful and others have not. None has been considered permanent, because the constant shifting of the river's alignment through the caving of banks upstream sooner or later will obliterate it. If the river's alignment is stabilized upstream from an improvement dredging operation, substantial permanent channel improvement both for navigation and for increased flood-carrying capacity can be obtained.

Navigation. Since there are no locks or dams south of Cairo, the capacity of the river to carry commerce is practically unlimited and is controlled by the towboats and barges. The most recent tonnage figures available are as follows:

From Mouth of Ohio River to but Not Including Baton Rouge, La.

Year	Tons	Year	Tons
1935	5,595,991	1939	7,009,130
1936	5,786,129	1940	7,055,675
1937	6,574,573	1941	9,605,602
1938	6,915,034	1942	9,474,591

Use of Levees. The main Mississippi River levees have a definite effect on navigation. Under natural conditions, the river spilled out over the alluvial valley as soon as it rose above bank level in floods. The effect of the construction of the main-line levee system has been to produce a river with greater flood discharges and greater variations between low and high stages than ordinarily existed.

The problem of maintaining the integrity of the main-line levee system against attack by the meandering river is inseparable from the problem of obtaining the best possible river channel for both navigation and flood control.

The project or design flood with a proper factor of safety for fixing the net grade of main-river levees is as follows:

Between Cairo and the mouth of the Arkansas River. . . .	2,450,000 cfs
Arkansas City.	3,065,000 cfs
At the latitude of Angola or Old River	3,000,000 cfs

Levee grades have been fixed, with the freeboards prescribed in the project, to correspond to the estimated crest flow lines of the project flood under

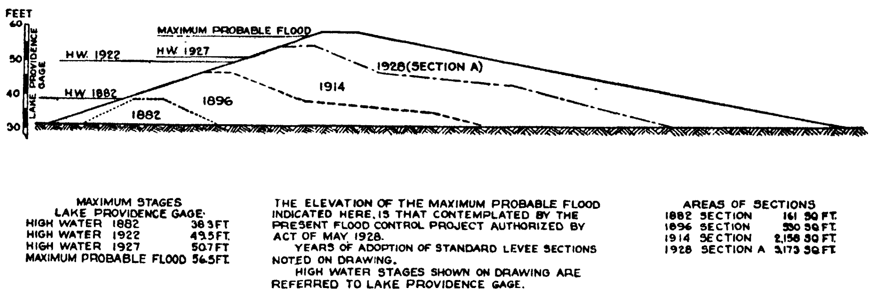


FIG. 10·3. Mississippi River, standard levee sections.

existing channel and floodway conditions, including all cutoffs, channel improvements, and the elimination of the Eudora floodway in Arkansas and Louisiana.

The levees are massive embankments 25 ft high or more and 240 ft or more through the base with frequent wide seepage berms, or banquettes, extending landward from the landside toe. In some locations, which are inaccessible in times of high-water, they have gravel roads on the crown. The progressive development in standard levee sections since 1882 is shown in Fig. 10·3. The present minimum section of levee, which includes a roadway, is 25 ft top width, upstream slope 1 on $3\frac{1}{2}$, bottom total width $10 \times$ height for random fill and $9 \times$ height for compacted fill. This results in a landside slope of about 1 on 4.

In its meandering, the river is always caving its banks and working toward the levees at many points. At some locations, the movement is slow and regular; at others, rapid and erratic. The rate of bank caving depends upon the materials in the bank and the direction and force of the attack against it. In Fig. 10·4 is shown a typical bend upon the lower Mississippi with cross sections at frequent intervals, showing current effects and tendencies and the relative position of bars and bends.

Straightening the river has increased its flood slopes and current veloci-

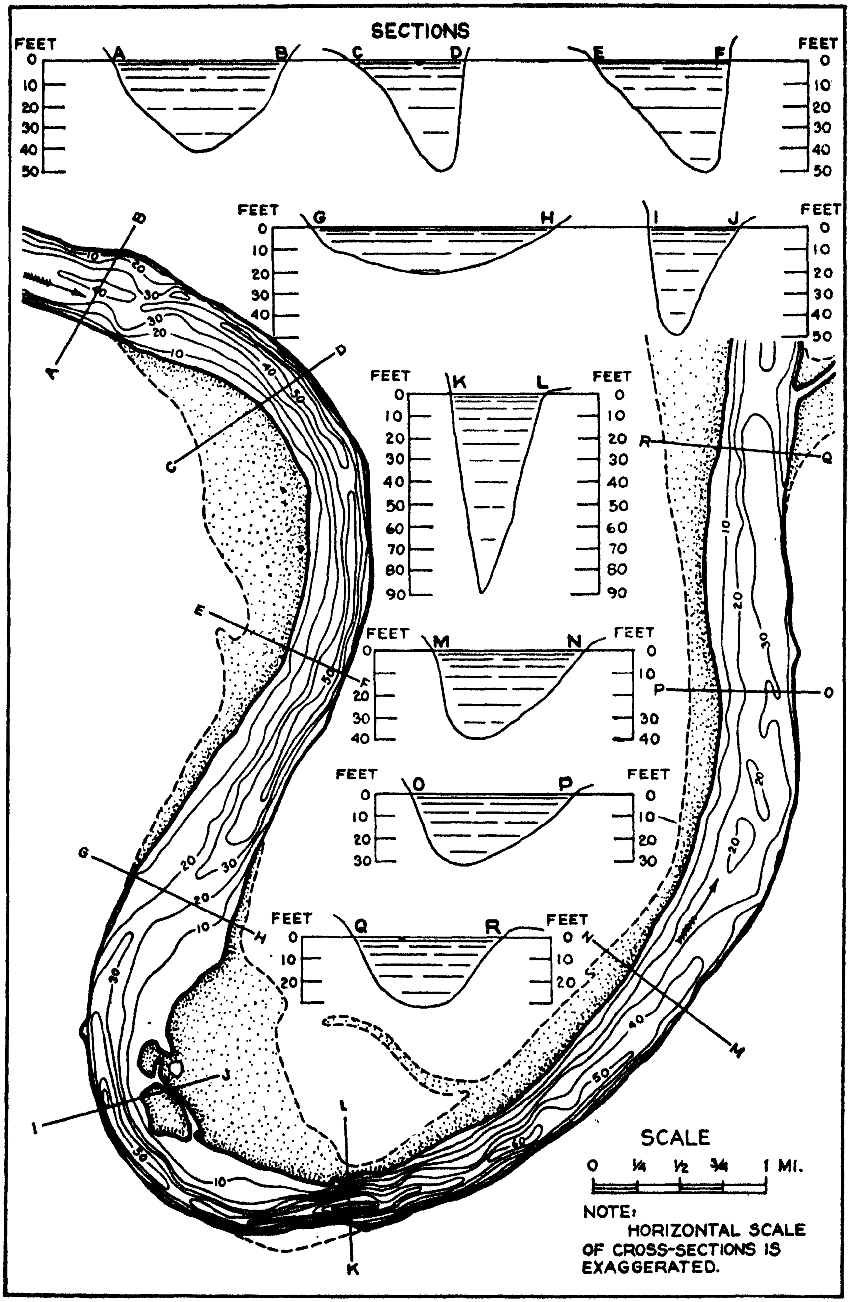


FIG. 10·4. Mississippi River, typical bend.

ties, causing both increased bank caving and meandering. Except where the river is held, it will regain a considerable portion of its former length by creating new bends and extending existing ones. This would sacrifice a proportional part of the reduction in flood heights that has been obtained in shortening the stream and would involve the loss of much of the cost of the channel work that secured those reductions. It also would require an increase in levee grades to compensate for loss in the stream's flood-carrying capacity.

When it becomes evident that the river is going to breach the levee line, one of three possible things must be done. A levee loop farther back from the river must be built; the bank caving must be halted by revetment work; or, in some cases, extensive dredging operations may divert the current from the caving bank. Each is a major operation.

A levee loop or *setback* must extend far enough upstream and downstream to ensure that neither of its connections with the existing levee will lie within the danger zone, and it must be far enough back behind the existing levee to give promise of a number of years of useful life. Generally a setback will have a length of 2 to 4 miles, and usually it contains so large a yardage of earth that it cannot be built in one working season. Hence, if for any reason, such as lack of funds, the work cannot be started in time or the river speeds up its rate of bank caving, a serious situation develops.

Usually a setback is not a permanent cure of the trouble. At many points, the river is now near the third successive levee location made within the past 15 or 20 years. A setback sacrifices the highest and best land. The river makes as much land as it destroys, but what it makes is a sand bar across the channel from the setback. This secretion will not be fit for cultivation within a lifetime and can never be protected at any reasonable cost. Each setback is a local retreat of considerable magnitude, and after a term of years the results are amazing. For example, the lower Yazoo levee district has about the same length of levee line now, 178 miles, that it had when the district was organized; but none of the existing levee is on the original location, and since 1880 the district has lost 305 miles of levee which have been replaced by setbacks.

Alignment (see Fig. 10·3). If the river's alignment is good at the point where bank caving threatens a levee, the bank may be economically held by *revetment work*. The alignment is good if the curvature is moderate at the location and also for several miles both upstream and downstream. If the river upstream from the location is changing its alignment, the attack on the revetment will be shifted upstream or downstream, and the work will be destroyed by flanking unless extensions up- or downstream are made. It follows that stabilization should proceed downstream from a fixed point. As a general rule, revetments are successful where they are used as training works for the maintenance of good alignment. They prevent a good alignment from deteriorating and setting up a series of destructive changes downstream.

In some situations, it has been possible to improve the alignment of the channel by extensive *dredging operations* to remove the attack from a caving bank and thereby save the levee line. Such an operation involves the pumping and deposition in the right location of several million cubic yards of sand. Thus, by moving the sand out of the channel, it can be placed where it will serve a useful purpose by directing flow in the channel. The dredge can also close chutes and auxiliary channels. By dredging, good alignment can be obtained, but it cannot be held permanently by dredging alone. Hence protection of the levee system depends upon the control of the meandering of the stream, and this has become the most urgent and difficult problem to obtain.

River Meander. River meander is a highly complex phenomenon and has been given much study by the Mississippi River Commission. These studies have included model experimental work carried on as a research plan at the Vicksburg Experiment Station, geological examinations, and the development of the geological history of the river. It has been found practicable to produce small, meandering streams, which cave their banks, build point and blanket bars, scour the bends and build up the bars on the crossings during high stages, fill in the bends and scour through the crossing bars during low stages, and in all respects behave like full-sized meandering rivers.

The small, meandering streams reduce not only the dimensional and flow scale but the time scale as well, making it possible to observe in a few days the effects of what corresponds to many seasons of high and low stages. The model studies have also made it possible to watch the movement of material transported by the stream from its point of origin in a caving bank to the point where it has been deposited on a bar and to observe the direction of top and bottom currents, change in location of bank attack with change in stage, the relation of turbulence to bottom scour, and the changes that result in river regimen at downstream points from changes occurring farther upstream. The effect of slope and volume of flow on meander pattern as well as the relation between the character of the materials forming the banks and the rate of meander have been investigated with the following general conclusions:[1]

1. In a stream having banks of little cohesion, the sands caved out of the banks are transported only a short distance downstream and are laid down on the first bar where relatively low velocities obtain.

[1] H.R. Doc. 509, 78th Cong., 2d Sess., 1943, pp. 11–12.

2. With the same discharge and slope, the most rapid bank caving and meander will occur with bank materials of the least cohesion.
3. With the same bank materials and discharge, the most rapid bank caving and meander will occur on the steepest slope.
4. With the same slope and discharge, a stream having cohesionless banks will develop wide and shallow cross sections that are hydraulically inefficient, whereas a stream with tough banks will develop relatively narrow and deep cross sections of greater hydraulic efficiency. The first stream will meander rapidly, the second will meander slowly and will be narrower and deeper than the first.
5. If the banks of a stream are composed of cohesionless materials, the prevention of bank caving by any artificial means will cause the stream to modify its characteristics and to approach those of a stream with tough banks.

The Mississippi River Commission conducted a laboratory study of the meandering of alluvial rivers between 1942 and 1944.[1]

This investigation had two main purposes:

1. To determine the basic principle of meandering.
2. To determine the basic principles as to the changes brought about in the channel of a meandering river by stabilization of the caving banks.

In the laboratory, small-scale rivers in erodible materials develop naturally a series of continually changing bends by eroding their concave banks and by building convex bars. This is shown in Fig. 10-5, which shows (1) the initial straight-channel conditions, (2) conditions after 48 hr of flow, and (3) conditions after 72 hr of flow.

Meandering is essentially a natural trading process of sediment from banks to bars. Sand entering at the head of an alluvial river travels only a short distance before it deposits on bars on the inside of bends and is replaced by sand eroding from concave banks. In turn, the sand from the caving banks of a bend travels only a short distance before it deposits on bars and is replaced by sand from other caving banks. In uniform materials and on a uniform slope, a series of uniform bends will develop. The radii of bends increase with increase in discharge or slope for the same alignment of flow into the bends. For the same flow and slope, the size and shape of bends depend upon the alignment of the flow into the bends. Cross sections of a meandering river are deeper along the concave banks of bends because of the impingement of the flow against these banks. The resistance of banks

[1] A Laboratory Study of the Meandering of Alluvial Rivers, War Department, Corps of Enginneers, U.S. Army, Mississippi River Commission, U.S. Waterways Experiment Station, Vicksburg, Miss., May 1, 1945. 40 pp., 61 plates.

to erosion results in deep cross sections, and easily eroded banks result in shallow cross sections.

Every phase of meandering represents a changing relationship among three closely related variables, *viz.*, the flow and the hydraulic properties of the channel, the amount of sand moving along the bed, and the rate of bank erosion. These three variables strive to reach a balance but never do

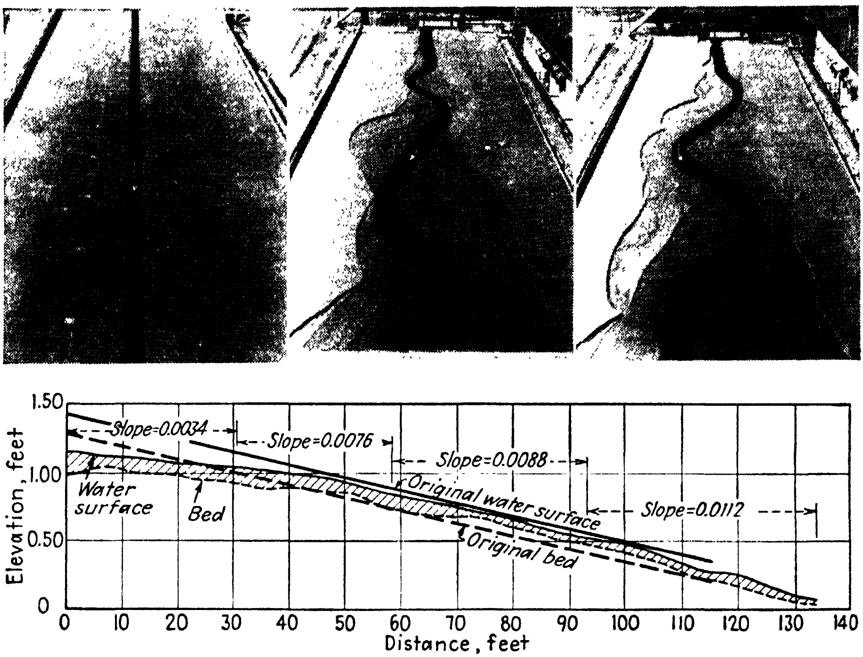


FIG. 10·5. Development of a meandering channel.

even with a constant rate of flow. The bends of a meandering river have limiting widths and lengths. When the bend reaches this width, a chute forms, and a new bend develops farther downstream. Distorted bends and natural cutoffs are caused by local changes in the bank materials.

A test was conducted in which a constant rate of flow was passed through a straight channel molded in uniform material. This stream developed a meandering course. In Fig. 10·5, the photograph on the left shows the initial straight channel molded in uniform sandy material, through which a constant rate of flow was passed that had a velocity sufficient to move sand along the bed and to erode the banks, which were the only prerequisites. No sand was fed at the entrance of the stream. As noted, the stream developed naturally the characteristic shifting, sinuous channel of a meandering river. The scallops along the boundaries of the meander belt are the bank lines of earlier courses. The degree of meandering increased downstream because no sand was fed at the entrance. In the upstream section, the stream deepened its channel and flattened its slope, as shown in the profile, until the flow no longer had the velocity to move material and erode the banks in this section.

This test indicated that, aside from the changing conditions and irregular bank lines found in nature, rivers that erode their banks will meander simply because a flow of water has a limited capacity for carrying sand along its bed. The only requirement for meandering is bank erosion.

A study was made to determine the changes brought about in the channel of the meandering river by stabilization of the caving banks. These experiments were made (1) with reference to a laboratory river with a series of uniform bends developed from a straight or slightly sinuous channel and (2) with reference to small-scale rivers that initially had the alignment of a section of the lower Mississippi River.

The revetment used consisted of canvas strips with sheet-metal ends that were forced into the bed at the toe of the bank. Neither dredging nor dikes was used in connection with bank stabilization. After the banks stabilized, stages were maintained and became lower, and the channel either maintained or increased its flow-carrying capacity. In both cases, stabilization of caving banks resulted in deepening of the thalweg along the stabilized banks that were under attack.

It follows that bank-stabilization works, if properly located to retain good river alignment, in combination with dredging can improve both the low-water navigation channel and the flood-carrying capacity of the river.

It also follows that the river will regain the length by which it has been shortened unless its present alignment is fixed by stabilization works. If it is allowed to regain its former length, the lowering of flood heights that has been obtained will be lost.

A study has been made of the channel conditions that obtained during the 9-year period 1935–1943 to determine the approximate additional amount of maintenance dredging that would have been required in each of those years had the project authorized the maintenance of a minimum depth for navigation of 12 ft instead of 9 ft. This study indicated that this could be maintained through the low-water season by doubling the number of dredges, which would mean the operation of 18 dredges for a period of about 6 months in years of unfavorable conditions and the maintenance of all 18 dredges in readiness to operate in all years.

The river now has the best alignment, the best navigation channel, and the greatest flood-carrying capacity it has ever had; but it will deteriorate unless this good alignment is maintained.

As an example, the present alignment of the 100 miles of river, as shown on the chart from Caulk Cutoff (mile 500 below Cairo) to Fitler Bend (mile 600), was studied in detail by Dr. H. H. Fisk, who developed a chart showing changes in river channels from prehistoric times for comparison with the present channel, which had a length of 167 miles before channel realignment was begun in 1933. This map shows in a striking manner the probability of extensive changes in river alignment if works are not built to prevent them. A portion of this stretch of river is shown in Fig. 10·6. It will take about 30 miles of new and additional work to hold this 100 miles of river in its present alignment.

Geological Investigation. An interesting and valuable investigation, relating to the alluvium of the Mississippi River as affecting engineering works, extending over several years under the direction of Dr. Fisk, has recently been completed.[1] This report is important to engineers concerned with levee construction, seepage problems, and river-channel maintenance in the Mississippi Valley and elsewhere.

The report shows that alluvium, long taken for granted as made up of haphazard deposits of silt, sand, and gravel, possesses a distinct pattern, which tends to repeat itself with little variation except where affected by tributary deposits. This pattern furnishes the key to subsurface conditions that influence the stability of structures, promote or block seepage, and determine the degree of erodibility of river beds and banks.

By 1941, many additional borings in the valley, excellent topographic maps, and aerial photographic mosaics made it possible to make interpretations of earlier stream courses extending back over a period of more than a thousand years and to trace fault lines in the basement rocks through the 100- to 300-ft depth of alluvium.

Alluvial deposits of more recent geological times have been relatively light and have not obliterated early river courses. No noticeable aggradation of the present river bed has taken place over 100 years, and the natural levees built up by the river during this period are no higher than those of many centuries ago.

Bank Protection—Lower Mississippi River.[2] The term "bank protection" includes all works designed to defend the riverbanks against attack by the current. A properly designed protection fulfills two requirements: it provides protection against floods, and the structure is of reasonable permanence. Caving is found principally in bends. Normal caving depends

[1] Fisk, Harold H.: Geological Investigation of the Alluvial Valley of the Lower Mississippi River, Mississippi River Commission, 781 pp., 1944; also review by Gerard H. Matthes, *Eng. News-Record*, Oct. 3, 1946, p. 95.

[2] "The Improvement of the Lower Mississippi River for Flood Control and Navigation," Vol. 2, Chap. VIII, 1932.

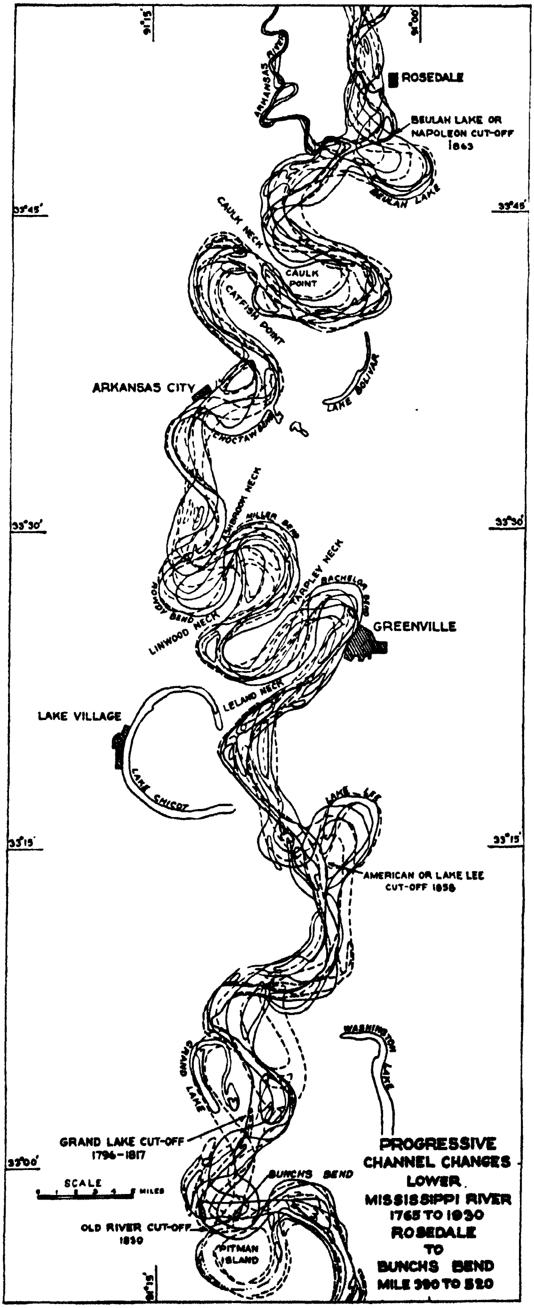


FIG. 10·6. Mississippi River. Channel changes, 1765–1930.

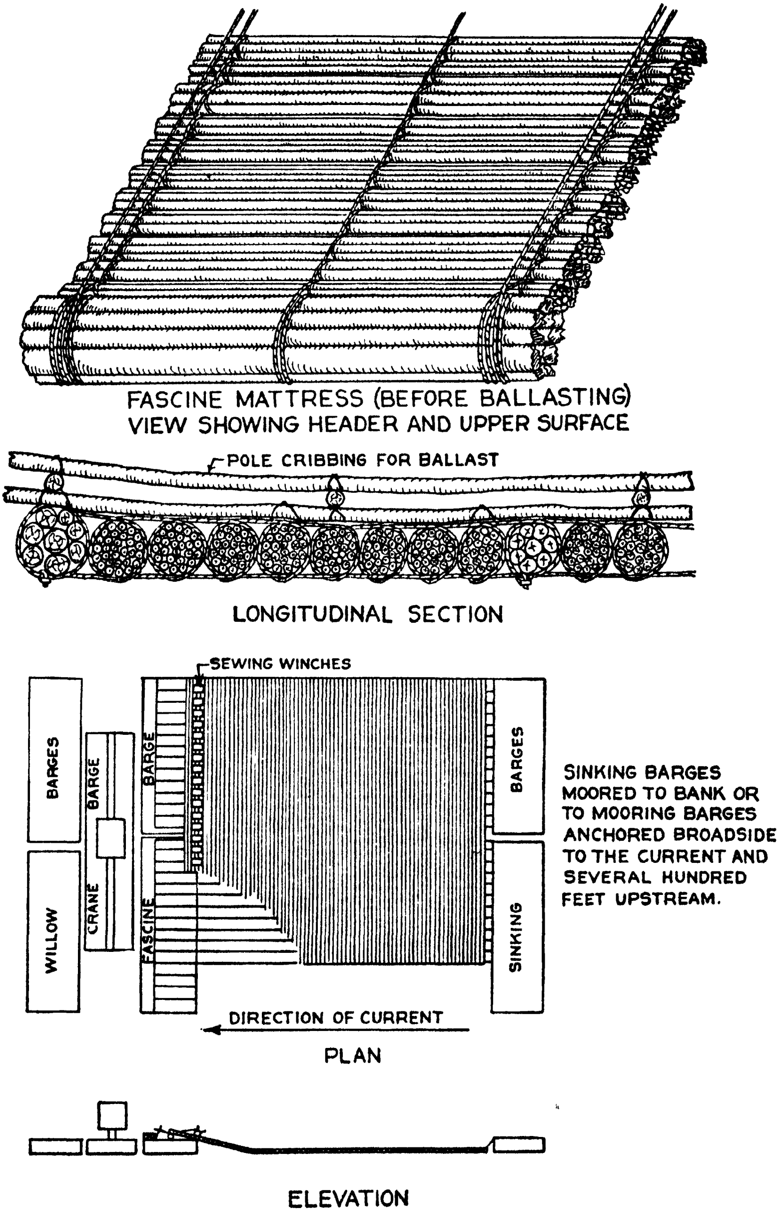


FIG. 10·7. Mississippi River. Fascine mattress.

on the curvature of the bend. Low-water caving is normally uniform and fairly free from eddy action. High-water caving is usually accompanied by eddies, although the water pressure tends to hold much insecure bank material in place. When the stage falls, however, large portions of the bank frequently break off and fall into the water.

The immediate purposes of bank protection are to prevent cutoffs; to prevent widening of the channel by bank erosion; and to protect levees from bank caving, particularly where conditions of habitation are such

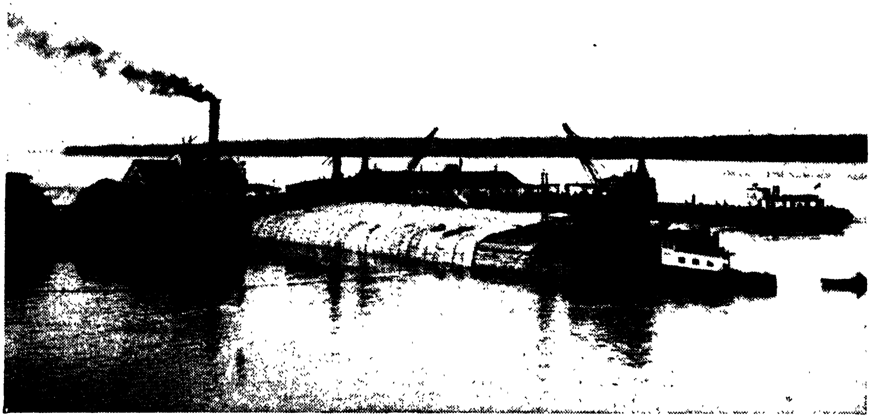


FIG. 10·8. Mississippi River. Concrete-mat sinking plant at Milliken Bend.

that it is impractical to set back the levees. Bank protection also reduces the silt load that the river must carry. Bank protection on the lower Mississippi has been in two classes—noncontinuous works such as spur dikes, and revetment. The former class has proved inadequate and has been discarded; and revetment is the only satisfactory type of bank protection yet developed.

Revetment extends from the top to the bottom of the bank and for some distance beyond the toe of the bank slope. The forms of revetment that have been found to be most practical and that are now in use are

1. A *frame mattress* consisting of a three-ply willow mat 16 in. thick secured between frames and built on sloping ways to facilitate launching.
2. A *fascine mattress* made up of willow fascines sewed securely together. This was evolved in an attempt to improve the woven mattress to give strength and compactness without too great a sacrifice in

flexibility. It was first used in 1893 and is now a standard type except as limited by scarcity of willow for the fascines. The fascines are 16 in. in diameter and from 250 to 300 ft long with a section of mattress usually from 1,000 to 1,200 ft long, built on the bank it is designed to protect (see Fig. 10·7).

3. *Articulated-concrete* revetments are composed of reinforced-concrete blocks in units 25 ft long by 4 ft wide made up of 20 blocks, each

FIG. 10·9. Mississippi River. Concrete revetment at Milliken Bend.

3 ft, 10¼ in. long by 14 in. wide and 3 in. thick, spaced 1 in apart on heavy reinforcing fabric (see Figs. 10·8 and 10·9).

4. *Concrete-slab* revetment. The butt slab now in use is 9 ft 11 in. long, 6 ft wide, and 3 in. thick, beveled at the corners and heavily reinforced. The assembly loops and handling loops exposed to corrosion and erosion are made up of noncorrosive metal.
5. *Reinforced-asphalt mattress* constructed of asphalt, loess, and sand, about 2 in. thick, poured on metal fabric. This is a reinforced asphalt laid in sheets 30 by 218 ft on the deck of the barges at the site of the works. On the banks above the water line, it is 2½ in. thick.
6. *Concrete blocks* dumped overboard from barges at places needed for emergency work.

Comparison of Revetment Types. Of the two *willow types*, the framed mattress is cheaper and less permeable; it has a longer life and is easier to construct; and it can be built in a convenient location, towed to position, and sunk. The fascine mattress must be built on the site where it is sunk. The framed mattress is thus particularly adapted for harbors or where river traffic is heavy.

Willow mattresses protected by a covering of silt and free from direct current attack last for long periods, but under direct current attack they disintegrate rapidly. Under such conditions, the fascine mattresses, being more vulnerable since they are not so compactly bound together, are subject to greater vibration and chafing and wear rapidly and rot. All willow mattresses are particularly vulnerable at the water surface, where rapid rotting necessitates costly maintenance, and at the toe (outer edge), where undermining is likely. This can be overcome only by extending the mattresses to a line well beyond the thalweg. Willow mattresses subjected to strong current last only about 10 years.

Concrete articulated revetment is cheaper than willow in first cost and maintenance. It is more stable, since its effective weight in place is about 18 psf as compared with 5 psf for willow. The concrete mat is more flexible, conforms instinctively to irregularities of bank and bottom, and ensures against undercutting at the toe. Some difficulty has been experienced, however, in foundation washing and settlement because of the spaces between the blocks. In general, it lasts longer than willow.

Concrete-slab mat has been more expensive than the articulated type, but improvement in methods of production and sinking have reduced its cost. It is, however, less adaptable to changes in bank conformation, is difficult to sink, and is likely to pile up rather than form a continuous covering. The increasing scarcity of willow growth has resulted in a lesser use of willow mats during the last decade.

Revetment in Use. There are at present on the Mississippi River below Cairo about 147 miles of revetment in effective use. There are in addition about 21 miles of revetment that is no longer effective because of channel changes. In addition, about 19,000 lin ft of revetment has been placed by the Mississippi River Commission at the mouth of the Ohio and on the south bank of the Arkansas River near its mouth.

The banks in the main-river reaches above Red River are more susceptible to caving than those below. From the mouth of the Ohio to the Arkansas River, the length of effective revetment averages about 772 lin ft per river mile. Between the mouth of the Arkansas and the mouth of the Yazoo River (at Vicksburg, Miss.), there are about 1,224 lin ft of effective bank protection per river mile. Between Vicksburg and the mouth of the Red River, the average is 732 lin ft of effective bank protection per mile. Between the Red River and New Orleans, the average is only 109 lin ft of effective bank protection per mile, excluding 68,000 ft of bank protection in New Orleans Harbor proper. No bank protection has been placed by the Mississippi River Commission below New Orleans.

Between Cairo and Baton Rouge, 737 miles, there are about 98 miles of effective bank revetment in place. To hold the river in its present alignment would take 230 miles of revetment in addition to that in place, if it could all be placed at once. The work must, however, be done by reaches, working downstream from naturally stable or previously stabilized locations and in accordance with a carefully worked out plan to get the best possible alignment through the reach under treatment. Thus each stabilized length of bank will serve as training work to direct the current to the next stabilized bank downstream. If work is placed in widely separated localities, it will prove uneconomical since much will be flanked out by the river, and most of it will prove to be badly located because the river will change its position and direction between the scattered works. A 50-mile section of the river will require about 25 miles of stabilization work, a reasonable and economical program for a season.

Cost of Bank Protection. Average costs of bank-protection work are as follows:

Type of revetment	Cost per square of 100 sq ft	
	1929–1930	1940–1944
Concrete slab	$22.38	$25.00
Articulated concrete	13.36	
Willow fascine	18.47	16.90
Framed willow	15.42	
Asphalt	23.00*	30.00
Upper-bank paving:		
Concrete-monolithic	11.75	
Riprap	16.18	

* 1934.

Total costs and costs per mile have been as follows:

Years	Miles	New works cost		Repair and maintenance cost
		Total	Per mile	
1881–1917	149	$26,708,000	$175,000	$ 7,027,000
1918–1931	90	32,546,000	350,000	22,966,000
1928–1943	93	49,040,000	530,000	43,894,000

The effective length of bank protection below Cairo as of June 30, 1942, was about 118 miles and of construction works (pile and crib dikes) about 31 miles. As previously noted, it will take about 230 miles of new bank-protection works to meet requirements of the proposed navigation channel of 12 ft depth, extending over a period of perhaps 15 years. This will involve a cost of about $165,000,000 with an additional amount of about $35,000,000 for improvement dredging and low dikes, making a total of about $200,000,000.

Backwater. The confining of floods between levee lines on both sides of the Mississippi River, with gauges permitting the outflow of tributary streams, has caused what are locally known as backwaters. These extend around the end of the main-line levee upstream and up into the tributary basin. The area flooded in each case depends on the height of the flood and the volume of discharge of the tributary. The principal backwaters are the St. Francis, the White, Arkansas, and the Red on the west bank and the Yazoo on the east bank.

Originally the status of backwater areas was comparable with that of lands elsewhere in the basins. Floods, however, spread with equal freedom over both banks of the river, inundating higher lands to comparatively shallow depths for comparatively short periods of time; but, since the main-river levees were made stronger and extended farther downstream, the lands upstream from the backwater areas were not overflooded at all, while the lands in the backwater areas were submerged to greater depths. The main-line levees have protected the backwater areas from Mississippi River floodwaters coming down overbank from upstream, but these same main-line levees, by confining greater floods, have increased the area of the backwaters and the depths of their flooding. Under earlier conditions, low, private levees gave sufficient protection to some of the higher backwater lands to permit of profitable operation of large plantations. Now, because of floods, many are no longer profitable and have been allowed to grow up in timber.

In the Yazoo backwater, it is proposed to improve this condition by backwater leveeing, limiting this, however, so as not to increase the height of floods that approach the top of the main-line levee.

Cutoffs and Corrective Dredging (see Fig. 10·11). The objections to the diversion of excess water down the Boeuf and Tensas Rivers helped to focus attention on the possibility of increasing the flood-carrying capacity of the main stream itself. It has been found after much investigation and study that this can be accomplished by remolding its bed and altering its alignment so as to provide an unobstructed and more direct, deeper and more efficient channel.

Carrying out this work has included development of cutoffs, corrective dredging in the reaches between cutoffs, and the rectification of the channel at other selected points. The objective of the plan was to direct the river's energy in the improvement of the channel so that floods could be carried at lower stages. This was to be accomplished by reducing excessive curvatures and correcting the alignment and shortening the channel, reducing the crossovers, enlarging primary and closing secondary channels, controlling the erosion of bars and banks, deepening channels, enlarging cross-sectional areas, and removing obstructions and constrictions.

The principal method of accomplishing this work was by corrective dredging and construction of sand dikes. Corrective dredging is that which tends to improve channel conditions by guiding the river's energy to accomplish erosion or deposition or both. The volume of material actually excavated is small in comparison with the volume of material moved by natural forces as a direct result of the character of this dredging. It is an operation that causes the natural forces to excavate and move material to the selected location for its final disposition. This may be along the bank, in a near-by secondary channel, or at a deeper pool downstream.

Construction Plant Used in Dredging and River Improvement. The project for channel alignment, corrective dredging, and cutoffs has required the movement and often the accurate disposal of large quantities of material dredged from the alluvial bed. This material varies from loose fine white quartz sand found in the bed of the stream to hard and well-compacted blue clay and mud. In some locations, underlying ledges of soft rock were encountered; and, in others, submerged forests of cypress trees and logs had to be disposed of. Some of the materials were well above the water surface even during high-water periods where cutoffs were made and on high banks of the river. The depth below the water surface of the moved materials ranges from 10 to 90 ft.

These requirements resulted in the development of various types of special plants to serve requirements best. In certain locations, it was found desirable to transport the dredged material through long pipe lines and deposit it in predetermined areas for the construction of sand fills that were expected to aid in the direction of flow of the main stream. In other locations, material was removed only from the bottom of the river and was allowed to disperse at the will of the river currents. In a few cases where roots, stumps, and logs formed an obstruction to operation, these materials had to be removed and disposed of.

Prior to 1932, few, if any, dredges were equipped to remove new materials at a depth in excess of 45 ft below the water surface. For the removal of subaqueous materials from 70 to 90 ft below the water surface, some very special items of plant capable of dredging at these depths had to be developed later.

The plants employed may be classified as follows:

Class I. Large-capacity, heavy-duty cutterhead dredges, which employ high-powered main-pump and cutter-drive prime movers. These were for the most part secured by contract from private dredging contractors and included the "George W. Catt," "The Cartagens," "Lake Fithian," and the "G. A. McWilliams." Dredges of this class are very flexible in their operation and power; for the main-pump drive varies between 3,500 and 4,500 hp, with that for the cutter-drive equipment varying from 500 to 1,000 hp. They are capable of undertaking the most severe class of dredging, including the handling of heavy clay, stumps, and in some cases soft-rock formations. The effective dredging depth below the water surface is between 35 and 45 ft, and various types of cutters and dredge-pump impellers are employed to suit the actual class of material being handled and are proportioned by necessary length of pipe line and static lift. Such dredges are capable of transporting material up to a total length of 6,000 to 7,000 ft and against heads caused by static lift varying from 35 to 65 ft. They are generally used under most extreme conditions requiring work of high capacity and head. Special cutter heads were provided for the plant capable of removing snags, roots, stumps, and even submerged forests.

Class II. Medium-capacity cutter-head dredges owned generally by the Corps of Engineers. These included the dredges "Barnard," "Gulfport," "Omega," "Currituck," "Raymond," and "St. Genevieve." This class is characterized by dredge-pump prime movers varying in size from 750 to 1,000 hp and cutter drives from 150 to 300 hp. Materials can be pumped to a depth of 30 or 40 ft and deposited through pipe lines varying in length from 1,000 to 3,000 ft against static lifts varying from 10 to 35 ft.

Class III. Bellmouth suction pipe-line dredges for maximum dredging depth. These normally obtain material through a bellmouthed type of suction head without mechanical agitation and transport the material through pipe lines controlled by baffle plates. In some cases, water-jet agitation methods were employed to assist in the procurement of material from the bottom of the river. This pump-barge class, developed in 1932 by General Ferguson, utilizes only the agitation provided by suction of the main dredging pump. It included pump barges Nos. 1 to 5 and several other large dredges. It is very flexible in operation and is equipped to remove materials at the extreme depth of from 70 to 90 ft below the water surface. It is a form of equipment peculiar to the Mississippi River. Materials are transported through pipe lines up to 2,000 ft in length (see Fig. 10·10).

Class IV. Known as "agitators," included the original arrangement of pump barges Nos. 1 and 2, as agitator unit No. 1 and pump barges Nos. 3 and 4, as agitator unit No. 2. The dredges "Detroit," "Wayne," and "Lake Fairfax" were also in this class. All units used side suction pipes equipped with drag heads, but without discharge pipe lines, the material being pumped overboard direct for disposal by river currents.

Class V. Included drag-line machines, tower excavators, and bucket dredges, employed principally for removal of overburden in the construc-

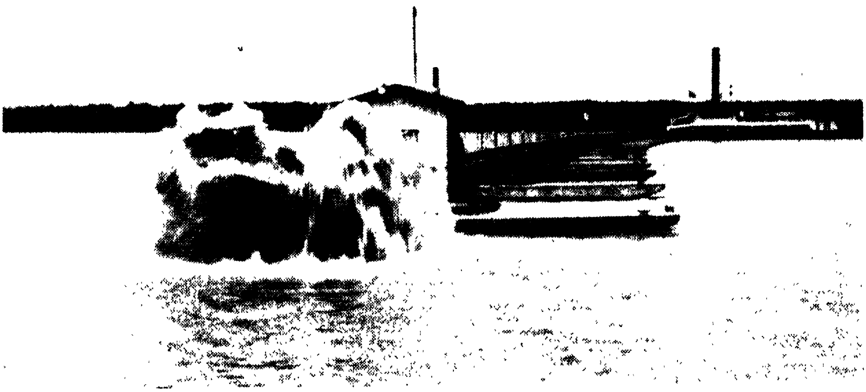


FIG. 10·10. Mississippi River. Pump barge in operation.

tion of cutoffs, etc. Drag-line machines have been obtained principally by contract and vary in capacity from 2½ to 6 ft bucket measure. The tower excavating machines were also used as levee-building units using a head and tail tower with a cableway between, on which the excavating bucket traveled, and were equipped with buckets from 10 to 12 cu yd capacity, capable of spanning an excavation pit from 750 to 1,000 ft in width. Included also in this class have been drag-line machines mounted on barges.

Effectiveness of Cutoffs and Channel-improvement Work. The program of cutoffs for improving the Mississippi River was initiated in 1932. Twelve cutoffs were made between the Arkansas and Red rivers and three between the Memphis and the Arkansas rivers. A natural cutoff occurring at Yucatan in the fall of 1929, combined with other channel improvements, has shortened the low-water channel of the river about

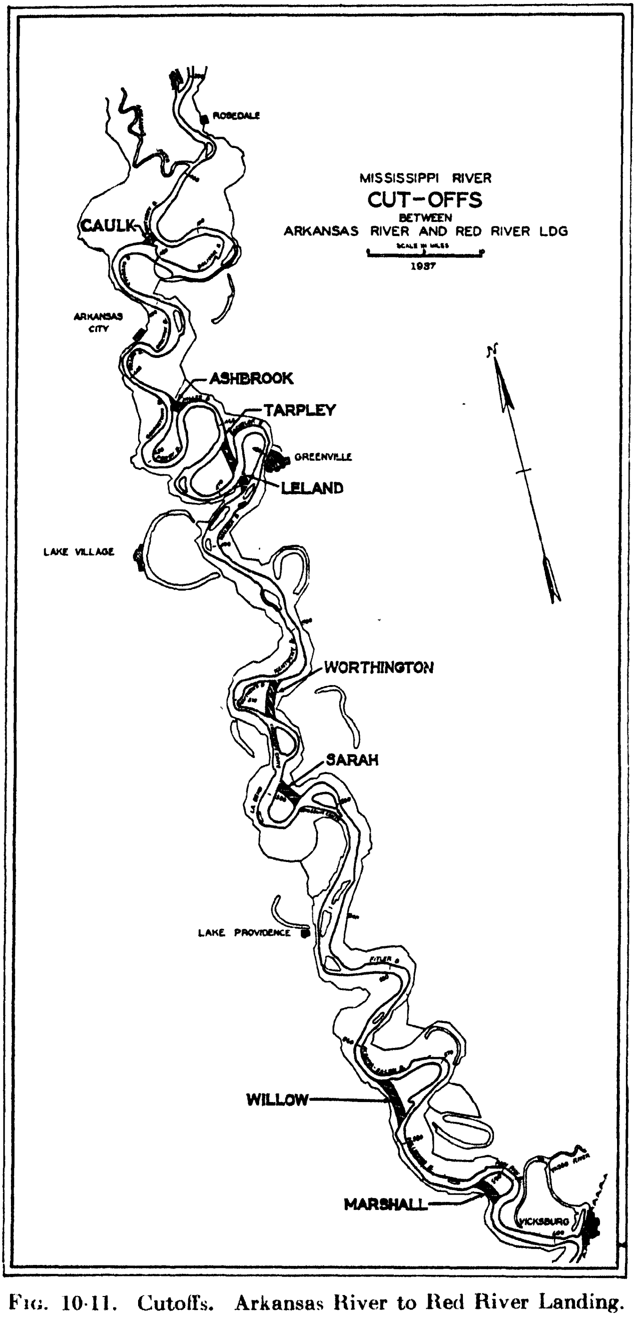


FIG. 10·11. Cutoffs. Arkansas River to Red River Landing.

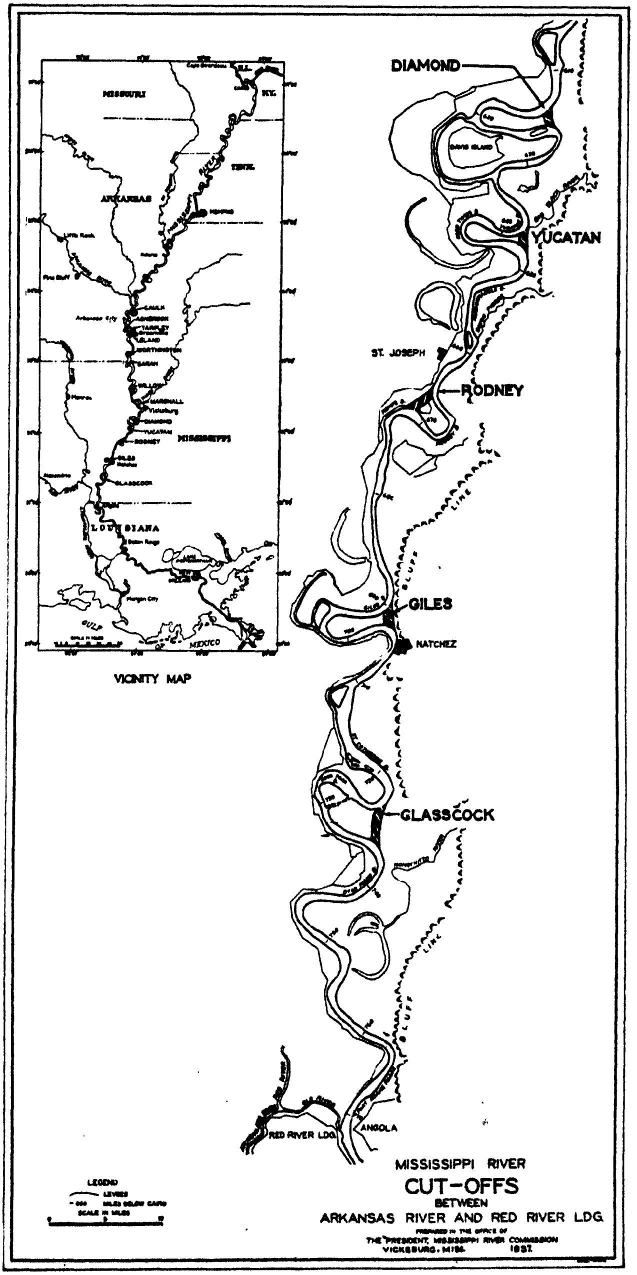


FIG. 10·11 (*Continued*).

170 miles, of which about 135 miles is between the Head of Caulk cutoff and the foot of Glasscock cutoff. The 1943 discharge measurements indicate that stages about bank-full have been lowered:

7 ft at Vicksburg.
13 ft at Arkansas City.
2 ft at Helena.
About 2 ft at Memphis.

In Fig. 10·11 is shown for this stretch of river the location and character of cutoffs. It also shows clearly the meandering tendency of the river channel.

FIG. 10·12. Mississippi River. Ashbrook, Tarpley, and Leland cutoffs.

The flood-carrying capacity of the central portion of the river has been increased by these cutoffs in amounts varying from around 100,000 cfs to as much as 700,000 to 800,000 cfs. Low-water stage has been lowered by the cutoffs, but the amount is generally considered less than the high-water lowerings. A number of troublesome bars have been eliminated by cutoffs, and low-water depths over the remaining bars have been adequate for navigation.

Sand fills or dikes have been used in the channel-improvement program to promote closure of secondary channels to prevent formation of sand bars in the middle of the reaches of the river and to accelerate extension of sand bars and deflect the direction of currents. In general, these works have been successful in causing the river to accomplish desired changes in channel or bank configuration. They are not to be regarded as permanent structures, although in many cases they have proved permanent. They have in most cases been constructed of waste material incidental to channel or improvement dredging in the immediate vicinity. In some

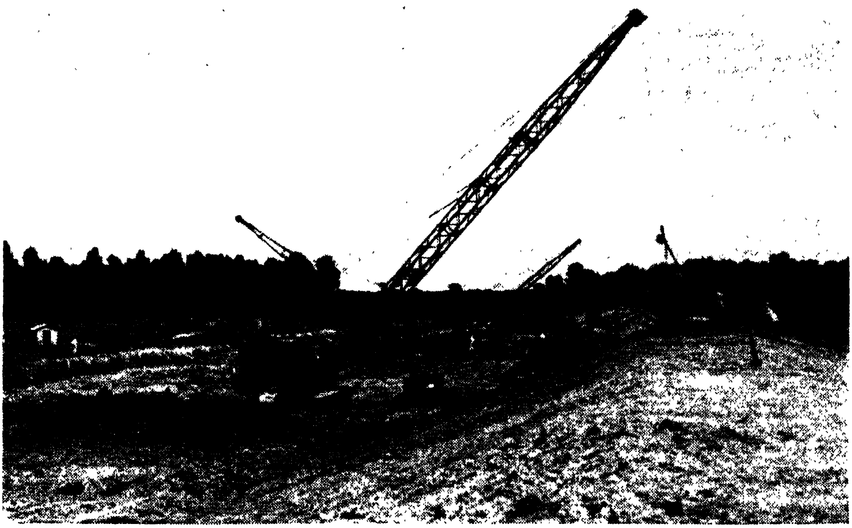


FIG. 10·13. Mississippi River. Levee construction.

localities, they were constructed from material pumped from the river for that purpose.

Levee Work. During the past 15 years, more than 839,953,000 cu yd have been placed in levees, or an average of about 56,000,000 cu yd yearly. Prior to 1928, total yardage placed by the United States was about 328,000,000 cu yd.

The average price per cubic yard for levee work, during the fiscal year ending June 30, 1943, was 30 cents in the Memphis district, 24.5 cents in the Vicksburg district, and 25 cents in the New Orleans district, covering both contract and hired-labor work and including overhead charges for the entire period from 1928–1943. The average price for the entire river has been 22 cents per cubic yard.

Levee Construction Plant (see Fig. 10·13). During the fiscal year 1943, the principal plant used by contractors in the construction of levee work included:

1. In placing compacted fill, hauling equipment used consisted of Euclid Trac-truks, Athey tractors, and Kosh ring trucks of from 12 to 18 cu yd capacity, loaded by drag lines with buckets from $1\frac{3}{4}$ to $3\frac{1}{2}$ cu yd capacity.

2. In placing of uncompacted fill, large drag lines with buckets from 4 to 8 cu yd capacity were used; and, in some cases, where the cast was too great for one machine, two machines were used.

3. A compacted-fill contract is being constructed with self-loading scraper units from 8 to 14 cu yd capacity.

4. In excavation of the drainage ditch in Cross County, Arkansas, three 6W Monighans, one 621 Page, and one 95 Northwest drag lines were used.

During the early work in the Vicksburg district, steam-driven tower or cableway machines and long boom drag lines with buckets of 6 to 8 cu yd capacity were extensively used. Several reaches of levee were constructed by hydraulic dredges with small drag lines as auxiliary equipment for cutting drain ditches and for dressing slopes and crown of levee. Tractor and wagon units were used for hauling material from pits beyond the reach of the tower machine or boom drag line. The wagons of from 5 to 7 yd capacity were loaded by a small drag-line or a belt-type elevating grader. More recently, hydraulic dredging operations have been confined to the construction of levee foundations at lake crossings, berm construction, and borrow-pit refills. Steam-powered land plants have given way to Diesel-electric equipment of much greater capacities. Tractor and wagon units are used quite extensively on long-haul jobs, and the track truck is used successfully on long-haul work because of its speed and economy of operation. In the New Orleans district during 1943, hired-labor activities were continued, using drag lines, bulldozer tractors, and hauling equipment.

Specifications. The method of placing earth in the levee section and the kind of material are important. With the usual equipment, the embankment is built in layers 3 to 5 ft thick. Clean earth, free from foreign matter, is used, rejecting earth that tends to slough except when hydraulic methods are employed. By plowing the foundation, a proper bond is secured; and, if any undue compaction has occurred, the surface is thoroughly broken up before fill is placed upon it. Provision is made for shrinkage after completion, varying from 8 to 25 per cent, depending upon the manner of construction. Enlargement of a levee is made upon the river side to avoid places of weakness between old and new earthwork. Specifications are rigidly enforced.

Paving for Riverbanks and Levees.[1] In the New Orleans district, bank protection or paving is now being used to protect the exposed riverbank above low water and the slopes of levees where they are close to the

[1] PICKETT, A. B.: Uncompacted Mass Asphalt Paving for River Banks and Levees, *Civil Eng.*, October, 1946, pp. 451–453.

riverbank and subject to attack by wave wash and current in exposed locations. This requires a paving that will flex with its own weight, adhere tightly to the subgrade without crawling, and, if possible, be sufficiently porous to bleed ground water while heavy and be rugged enough to withstand the various forms of water attack.

A type of mass-asphalt paving was first developed and tried out for upper-bank paving at Avondale bend just above New Orleans in 1943 with a mixture of 55 per cent washed concrete gravel, 38 per cent washed sand, and 7 per cent of 40/50 penetration asphalt cement. The mass asphalt was mixed in batches at 300 to 350°F and placed, in a clamshell bucket at about 300°F. The bank subgrade irregularities were removed, and the hot mix dumped on and raked down to an average thickness of about 6 in., neither rolled nor compacted. This bank is still in excellent condition. This general method has also been applied to levee slopes in the New Orleans district with good results in a number of contracts with unit prices lower than for other types of pavement.

Total cost of levees to 1943 has been as follows:

	New work	Maintenance	Total
Costs incurred by United States:			
From government funds	$183,238,000	$14,124,000	$197,362,000
Contributed funds	1,468,000	4,000	1,472,000
Expended by state and local organizations, including prior projects	. . . . . .	. . . . . .	253,224,000
Totals	$184,706,000	$14,128,000	$452,058,000

Present Condition of Levees. The present condition of the levee system from Cape Girardeau, Mo., to Head of Passes, La., as of June 30, 1943, is as follows:

Items	Main-river levees	Tributary levees and outlets
1. Included in system when completed, miles	2,508	112
2. Built, miles	2,490	90
3. Contents, million cu yd	1,078	4
4. Built to adopted grade and section, per cent	80	75
5. Required to complete work, million cu yd	258	1.6
6. Estimated final contents, million cu yd	1,303	6.5
7. Estimated maintenance, 1944, million cu yd	16.5	0.05
8. Approximate areas protected with existing levees, sq miles	27,627	375

Reservoirs in Mississippi River Basin.[1] The benefits from further flood control that may be attained by reservoirs lie in the possibility of eliminating recourse to floodways for the escape of excess floodwaters or in reducing the frequency with which the floodways must be used; and in affording greater safety to the main-river levees by reducing the height to which floodwaters rise against them and by reducing the area flooded by backwater through the opening left for outflow of tributary streams. It is impossible to provide reservoirs that will prevent the flooding of all lands without levee protection.

In 1935, a comprehensive study[2] was made to determine the aggregate effect of reservoir systems on river control. As a result of these investigations the so-called "Plan II" was recommended to serve as a general guide for construction of reservoirs for flood-control protection. The outline and estimated cost of this plan are as follows:

Reservoirs and capacities	Number	Million acre-ft
Total in Ohio Basin	81	25.3
Total above Cairo, Ill.	106	52.9
Total above Arkansas City, Ark.	132	68.1
Total all reservoirs	151	98.7

Cost:	
Construction	$ 605,376,000
Flowage	304,828,000
Railroads and highways	215,603,000
Total	$1,125,807,000
Annual cost, operation, and maintenance	$7,784,800

As compared with the total cost of about $1,126,000,000 for Plan II, the computed benefits were found to be only about one-half of the cost. The recommendation was made that the system of reservoirs in Plan II be accepted as the general plan toward which future Federal flood construction will be aimed.

It is thus seen that no specific plan for reservoirs in the Mississippi Basin has yet been worked out. The procedure has been to study each river system, *viz.*, the Ohio, Missouri, Upper Mississippi, the group of westerly tributaries, including the White, Arkansas, etc., and determine the best reservoir sites for the flood control of these tributaries as a beginning of the

[1] H.R. Doc. 259, 74th Cong., 1st Sess.
[2] H.R. Doc. 308, 69th Cong., 1st Sess.

problem. It is not known as yet what final groups of reservoirs will be adopted.

This has resulted in a tentative classification of (1) reservoirs constructed or initiated and (2) those proposed, or most desirable, at this time, the latter being added to from time to time as investigations proceed.

Effect of Headwater Reservoirs.[1] The approximate present status of the reservoir program in the Mississippi River Basin is shown in Fig. 10·14 supplemented by Table 10·2. Included are 43 reservoirs that have been constructed or initiated with about 45 million acre-ft of storage capacity (of which about 20 million acre-ft is at Fort Peck Reservoir in the upper Missouri River) and 90 more reservoirs that are proposed, with about 82 million acre-ft of storage.

This makes a grand total of 133 reservoirs and about 127 million acre-ft of storage.

These reservoirs will, with the exception of the group on the Tennessee River, be operated for so-called "local protection," that is, to reduce flood-peak discharges so that resulting outflow will not exceed safe stages in tributary valleys in which reservoirs are located. Mississippi River flood stages at and below Cairo will benefit incidentally except in the case of very remote developments such as those above Pittsburgh and in the Muskingum Basin.

The 1937 flood is one in which practically all the Ohio River tributaries contributed an appreciable amount of water. It serves as an example in which reservoirs on the Ohio River Basin, especially in the lower portion of the basin, would manifest maximum effects upon flood flows at Cairo.[2] The 1937 flood is one in which the middle and lower tributaries of the Ohio contributed relatively high discharges. The project or design flood combines high flows from the tributaries of the middle and upper Ohio River and moderate flows from the two lower tributaries with high flows from the Upper Mississippi and Missouri Rivers. The March-April, 1913, flood in the Ohio Basin was a part of this combination. In the storm producing this flood, the axis lay north of the Ohio River and extended from northern Ohio to southern Illinois. Had the axis of this storm been rotated clockwise about 10 to 15 deg, all that would have been needed to produce the design flood would have been the recurrence of slightly greater depths of precipitation in the western end of the storm. It is therefore believed that the changes required are meteorological possibilities. Below Cairo, the project flood assumes 1927 floods in the tributaries. The period between the date of the rainfall above Cairo and the arrival of the upriver flood crest at Arkansas City is sufficient to permit a storm to occur over the Arkansas

[1] Com. Doc. 1, 75th Cong., 1st Sess., 1937.

[2] H.R. Doc. 359, 77th Cong., 1st Sess., 1941, pp. 26–28.

Table 10·2 Mississippi River—All Tributary Storage Reservoirs, Lower Mississippi River

River basin	Constructed or initiated		Proposed		Total		Cost	
	No.	Capacity, thousand acre-ft	No.	Capacity, thousand acre-ft	No.	Capacity, thousand acre-ft	Total	Per acre-ft
St. Francis	1	585	...	......	1	585	$ 5,500,000	$9.40
White	2	1,123	6	6,345	8	7,468	151,564,000	20.40
Arkansas	11	6,135	7	3,030	18	9,165	188,098,000	20.50
Red	3	6,000	...	......	3	6,000	57,500,000	9.60
Yazoo	3	2,557	2	1,957	5	4,514	48,000,000	10.60
Totals	20	16,400	15	11,332	35	27,732	$450,662,000	$16.30
Missouri River	1	19,500	8	42,150	9.	61,650	490,000,000	8.00
Upper Mississippi River*	...	....	10	1,900	10	1,900	21,000,000	11.00
Ohio River	22	8,735	57	26,833	79	35,568	552,400,000	15.50
Grand total Mississippi River Basin	43	44,635	90	82,215	133	126,850	$1,514,062,000	$12.00

* As proposed in H. R. Doc. 1.

River and White River basins at such a time that the resultant peak floods could synchronize with the main-river flood. Hence floods equal to 1927 could result if the rainfall were of the required intensity.

The stage reduction on the Cairo gauge with the Bird's Point–New Madrid floodway operating are given in the following table for the two groups of reservoirs described above:

Discharge and Stage Reductions at Cairo Due to Reservoirs

Flood	Reservoirs constructed or initiated		Reservoirs proposed	
	Discharge, cfs	Stage, ft	Discharge, cfs	Stage, ft
1937	216,000	2.2	306,000	3.1
Project flood	129,000	1.3	169,000	1.7

The reservoir effects on the middle section were determined by calculations carried to Arkansas City and Vicksburg for the project flood, with results as follows:

Discharge and Stage Reductions at Arkansas City and Vicksburg Due to Reservoirs

	Reservoirs constructed or initiated		Reservoirs proposed	
	Discharge, cfs	Stage reduction, ft	Discharge, cfs	Stage reduction, ft
Kansas City, Ark.	172,000	2.3	195,000	2.6
Vicksburg, Miss.	96,000	1.2	112,000	1.4

The foregoing reservoir effects in restraining floods are practically available only from the first of the group of reservoir systems. The second, larger system, although it is authorized, cannot be counted upon for some time to come.

In the Tennessee River group, the large-capacity Kentucky Reservoir just above the mouth is specifically designed to regulate floodwaters from the entire Tennessee River with the object of reducing flood stages at Cairo as effectively as possible, assisted somewhat by the operation of the Lower Tennessee River. The Kentucky Reservoir and the other three lower reservoirs on the Tennessee River will have a flood-storage capacity of 5,000,000 acre-ft.

Preliminary studies in the Kentucky Reservoir control on Cairo stages indicate that a reduction of from 2 to 2½ ft can be expected. The accompanying table gives data in this regard for numerous floods between 1912 and 1937.[1]

Reduction of Cairo Stages in Major Winter Floods by Kentucky Reservoir

Date of crest	Actual peak stage, ft	Estimated peak for confined flood, ft	Reduction, ft	Maximum reservoir level at the dam
Apr. 6, 1912	53.94	....	2.4	373.9
Apr. 7, 1913	54.69	62.5	2.3	357.0
Mar. 27, 1922	53.60	....	2.5	357.4
Apr. 25, 1922	53.50	....	2.4	365.6
Apr. 20, 1927	56.40	58.5	2.5	359.0
Mar. 19, 1929	51.80	....	2.5	
Apr. 5, 1929	51.50	....	2.2	368.5
May 18, 1929	52.70	....	2.5	373.9
Apr. 10, 1936	52.74	....	2.5	370.8
Feb. 4, 1937	59.51	62.5–63.0	2.0	375.0

Reservoirs in the Alluvial Valley—Lower Mississippi River.[2] As a part of the flood regulation of the St. Francis and Yazoo River basins, five reservoirs have been planned, of which three have been constructed, as shown in Table 10·3. They will afford a total storage capacity of about 4½ million acre-ft at a cost estimated at $53,500,000, or about $11.80 per acre-ft, a low unit cost. These five reservoirs will control 5,635 sq in. of drainage area with 800 acre-ft per sq mile or about 15 in. of storage, a relatively high degree of storage, which results in low unit costs, as noted above.

Model Studies—Vicksburg Experiment Station[3]

Model of Mississippi River Basin. In conjunction with the selection of number and capacity of reservoirs, a model study of reservoir operation was ordered made by the Chief of Engineers, U.S. Army, in 1942, at the Vicksburg Experiment Station, which it is hoped will be of great value in a more accurate solution of this problem. This, when completed, will be

[1] H.R. Doc. 359, 77th Cong., 1st Sess., p. 25.

[2] *Report of Chief of Engineers*, 1943, pp. 1846–1850.

[3] Matthes, Gerard H.: *Tech. Rev.*, February, 1945, p. 7.

Table 10·3 Reservoirs in Alluvial Valley—Lower Mississippi River

No.	Reservoir or site	River	Drainage area, sq miles	Max. height of dam, ft	Storage capacity, acre-ft	Acre-ft per sq mile	Cost		
							Total	Per acre-ft	Per sq mile of drainage area
				Constructed					
	St. Francis Basin:								
1	Wappapello (1938–1941)	St. Francis, Mo.	1,301	77	585,000	450	$ 5,500,000	$9.40	$ 4,200
	Yazoo Basin:								
2	Arkabutla (1940–1943)	Coldwater, Miss.	1,000	65	494,000	494	10,000,000	20.30	4,100
3	Sardis (1936–1940)	Little Tallahatchi, Miss.	1,545	117	1,478,000	960	13,000,000	8.80	8,400
					2,557,000				
				Proposed					
1	New Grenada	Yalobasha, Miss.	1,320	84	1,337,000	1,010	$17,000,000	$12.70	$12,800
2	Enid	Yocona, Miss.	560	87	620,000	1,100	8,000,000	12.90	14,300
Totals			5,735	.....	4,514,000	800	$53,500,000	$11.80	$ 9,500

the largest hydraulic model yet built. The model will include the entire watershed of the Mississippi, of 1,244,000 sq miles and its principal tributaries.

It will be built to a scale of 1 ft = 2,000 ft horizontally and 1 ft = 100 ft vertically and will require an area of about 200 acres and a vertical range in elevation of ground survace of 50 ft, with a total length of model streams of 8 miles.

All existing and proposed flood-control reservoirs totaling about 200, as well as levees, dikes, flood walls, floodways, and other pertinent works, will be reproduced. The Lower Mississippi River, which has a width of about 4,000 ft and a bank-full depth ranging from 50 to 150 ft, will be scaled down in the model to a width of 24 in. and a depth of from 6 to 18 in.

Using data obtained in nature, floods of known magnitude will be placed in the model and their courses recorded by sensitive electrical recording devices, expected to number about 1,700. Water for the model stream will be controlled by mechanical measuring equipment, electrically operated from central control points. More than 1,000 gpm of water will be required to operate the model, since the amount will represent at the scale of the model a maximum flood of record in the Lower Mississippi River. The model rivers will be molded in concrete with the intervening ridges and valleys covered with sod.

The purpose of the model is to provide a visual means of studying the many problems that arise in the operation of so large a number of reservoirs as those already built or contemplated. These reservoirs will aggregate a billion and a half dollars in cost when all are completed. In 1944–1945, grading and drainage work was carried on, with about 1,300 German prisoners of war for labor, working with wheelbarrows and shovels, or operating trucks and tractors. They cleared the site and constructed temporary roads, bridges, and drainage ditches. The actual model construction is going slowly until needed materials become available. Upon completion, a road around the model, 3 miles long, and viewing towers will be provided to observe the model in operation.

Model of Lower Mississippi River.[1] In 1938, a model was constructed at the Vicksburg Experiment Station of the lower Mississippi River Basin between Helena (a little below Memphis) and near the Gulf, representing an area of about 16,000 sq miles, 400 miles long, on a scale of 2,000 ft = 1 ft horizontally and 100 ft = 1 ft vertically (see Fig. 10·15). It was laid out as a plane with a horizontal control of polyconic projection of geographic coordinates. Sheet-metal templates were used to develop topographic features, with spaces between filled with fine sand and clay.

[1] FIELDS, CAPT. K. E.: The Mississippi River Flood Control Model, *Proc. Hydrology Conference*, State College, Pa., June 30–July 2, 1941, p. 213.

Scaled ratios used were

Velocity	1 to 10
Discharge.	1 to 2,000,000
Time.	1 to 200, or 1 day (prototype) = 7.2 min (model)

This model is adaptable to all the possible methods of flood control upon the Mississippi River, including cutoffs, channel enlargements, levee alignments and grades, floodways, tail-water storage areas, and backwater

FIG. 10·15. Model of Lower Mississippi River Basin, air view.

reservoir operation. It has proved very useful in numerous studies, including determining the effects of leveling off the Yazoo backwater area upon downriver flood heights; also in recent determination of necessary flood capacity at the Morganza floodway to handle safely the 1945 flood rise on the Lower Mississippi.

Note that its scale is the same as that of the much larger model of the entire river basin now being constructed, as previously described.

Cost of Lower Mississippi River Work. As of 1945, there has been *expended* upon the lower Mississippi work the following:

Previous project	$125,486,000
Current project ac/o Act of 1928 as amended . .	862,982,000
Total	$988,468,000

Federal authorizations include:

1928	$325,000,000
1936	313,000,000
1938	40,000,000
1944	200,000,000
1946	100,000,000
	$978,000,000

Essential Features—Flood Control of Lower Mississippi

The essential features of the present plans for flood control of the Lower Mississippi may be summarized as follows:

1. *Levees* to prevent overflow, along each side of the river except at the entrance to tributary backwater area and where high banks serve to confine flow.
2. *Cutoffs* to increase the flood-carrying capacity of the river resulting in lower stages.
3. *Revetments* where necessary to stabilize the channel.
4. *Emergency storage areas* or portions of the backwater areas leveed off and protected from all but extreme floods.
5. *Floodways* to carry water in excess of the safe capacity of leveed channels, including the new Madrid floodway near Cairo, the Morganza-Atchafalaya floodway near Angola, and the Bonnet Carre spillway to Lake Pontchartrain just above New Orleans.
6. *Reservoirs*, one upon the headwaters of the St. Francis and five upon those of the Yazoo to assist in the control of these tributary streams.
7. *Reservoir systems* in the Ohio, Missouri, Upper Mississippi, Arkansas, White, and Red River basins primarily for flood control upon these rivers, but which incidentally lessen flood peaks upon the Lower Mississippi. The number and capacity of these reservoirs and their effect are not yet definitely determined.

CHAPTER 11

OHIO, MISSOURI, AND UPPER MISSISSIPPI RIVERS

Ohio River Basin[1]

The Ohio River is formed at Pittsburgh by the confluence of the Allegheny (11,705 sq miles) and Monongahela (7,340 sq miles) rivers. It occupies portions of 14 states and flows in a generally southwesterly direction to join the Mississippi River at Cairo, Ill. Its total drainage area is 204,000 sq miles; length about 981 miles; average river slope 0.4 ft per mile, and estimated maximum and minimum discharges at the mouth of 1,700,000 and 22,000 cfs, respectively. Its larger tributaries in order downstream and their areas in square miles are as follows:

Muskingum	8,040
Kanawha	12,200
Scioto	6,510
Miami	5,406
Kentucky	6,935
Green	9,430
Wabash	33,100
Cumberland	18,000
Tennessee	40,600

It composes about 16 per cent of the total area of the Mississippi River Basin, or about 6½ per cent of the entire area of the United States with, however, a population of about 14½ per cent of that of the entire country, and 13 per cent of its railroad mileage.

Agriculture is one of the principal pursuits; and the basin is one of the most highly developed industrial regions of the world and is rich in mineral resources, coal, and many other products.

Navigation. The development of navigation facilitates upon the Ohio River covers a period of over 125 years, beginning with the Louisville and Portland Canal Co., chartered in 1825, and followed later by Congressional action for dredging and channel improvements. In 1874, supervision of navigation was taken over by the United States, and traffic charges were abolished.

The Ohio River navigation system includes, in addition to the main river, 11 of its longer tributaries with a total of over 2,500 miles of improved waterways in the system. In 1925, the total commerce upon the Ohio

[1] Ohio River, H.R. Doc. 306, 74th Cong., 1st Sess., 1936; also Com. Doc. 1, 75th Cong., 1st Sess., 1937.

River proper was nearly 16 million tons and on the entire system about 48 million tons.

The present main-river project was authorized in 1910, calling for the construction of 54 locks and dams from Pittsburgh to the mouth to provide a navigable depth of 9 ft and channel width of 500 ft. Of these dams, 51 were actually constructed, with locks 110 by 600 ft, and a total lift of about 426 ft with lifts in general from about 6 to 9 ft—the chief exception being No. 41 at Louisville with a lift of 37 ft. All but two of the dams are movable, of "bear-trap" construction. The project is essentially complete, and traffic can proceed under practically any river condition. Navigation facilities are adequate for existing commerce and for any demand that can now be foreseen. The total cost to 1943 of the dams and locks has been about $134,000,000 for new work and $62,000,000 for maintenance. Channel work, including diking, dredging, etc., has required about $14,000,000 for new work and $23,000,000 for maintenance.

Above Pittsburgh on the Monongahela River, there are 13 locks and dams built between 1897 and 1925 at a cost of about $16,000,000, affording slack-water navigation about 127 miles to Fairmont, W. Va. with depths of 9 to 7 ft. On the Allegheny, there are 8 locks and dams affording slack water to East Brady, Pa., a distance of 72 miles. These were built between 1927 and 1938 at a cost of about $18,000,000.

The Yougheogeny River, principal tributary of the Monongahela, has under construction two locks and dams to provide a 9-ft channel from its mouth to West Newton, Pa. about 19 miles, at an estimated cost of about $4,000,000.

Below Pittsburgh, the Kanawha, with four dams and locks (one on Ohio River) provides a 9-ft channel for 90 miles at a cost of about $25,000,000.

The total cost to 1943 of the Ohio River navigation works, including the more important tributaries, except the Tennessee and Cumberland (see page 292) is therefore about 200 million dollars for new work and 100 million for maintenance, or a total of about 300 million dollars.

River traffic between Pittsburgh and Cairo has been as follows:

Year	Vessel traffic, tons	Rafted, tons	Total traffic, tons	Number of passengers
1937	23,348,000	8,800	23,357,000	1,728,539
1940	29,546,000	2,900	29,549,000	1,957,308
1943	36,607,000	3,200	36,610,000	1,191,350

The Tennessee River, the largest tributary of the Ohio River, has been comprehensively developed in the interests of power, navigation, and flood control by the Tennessee River Authority during the past decade, as described elsewhere (see page 291).

Ohio River Flood Control—Federal Projects. Flood control at Pittsburgh has been under consideration for many years. In 1912, the Pittsburgh Flood Commission made an elaborate report[1] advocating a proposed 17-reservoir system on tributaries above Pittsburgh, controlling about 10,000 sq miles of drainage area with 1,360,000 acre-ft (about 2 in) of storage capacity at an estimated cost of $20,000,000, which included a river wall at Pittsburgh. This project was not carried out.

Federal flood-control measures began in 1935 with the Tygart Reservoir, on the Tygart River near Grafton, W.Va., completed in 1937 with a storage capacity of about 300,000 acre-ft upon a drainage area of 1,183 sq miles at a cost of about $18,500,000. In addition to flood-control use, this reservoir aids the navigation low-water flow of the Monongahela.

The Flood Control Act of 1936 authorized the construction of nine reservoirs in the Allegheny-Monongahela basin for the protection of Pittsburgh and five reservoirs in the Kanawha and Licking River basins.

The Flood Control Act of 1938 approved, "with such modifications thereof . . . as may be desirable," a plan including 49 additional reservoirs, at an estimated total cost of $312,000,000, a total storage capacity of about 29 million acre-ft, controlling about 40,600 sq miles of drainage area, together with some 235 levee and wall projects along the Ohio River and its tributaries, at a total estimated cost of about $300,000,000. This act authorized for initial expenditure $75,000,000 for reservoirs and $50,000,000 for local flood-protection works.

A summary of the list of reservoirs under consideration for flood control in the Ohio River Basin as of 1941 is given in Table 11·1. This shows data for both constructed and authorized projects and also for proposed projects "which appear most necessary and desirable at this time."

This revised schedule (1941) of the comprehensive reservoir system includes 79 reservoirs, of which 22 are constructed or in process of construction, with about 22 million acre-ft of flood-control storage capacity, and about 37 million acre-ft of total storage capacity, controlling about 58,900 sq miles of drainage area, at a total estimated cost of about $544,000,000.

Local flood-protection works at cities and towns along the Ohio River are also planned as follows for the several districts, as of 1941:

District	Estimated Cost of Local Protection Works
Pittsburgh	$ 81,765,000
Huntington	44,437,000
Cincinnati	55,257,000
Louisville	34,488,000
Nashville	50,300,000
Total	$266,247,000

[1] Report of Pittsburgh Flood Commission, 1912.

Table 11·1 Ohio River Reservoir Projects, 1941

District	River	No. of reservoirs	Drainage area controlled, sq miles	Capacity, thousand acre-ft		Cost	
				Flood control	Total	Total	Per acre-ft capacity
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
A. Completed, under construction, or funds allotted							
Pittsburgh	Monongahela	2	1,618	528	544	$ 27,500,000	$51.00
	Allegheny	4	1,379	380	397	22,200,000	56.00
Huntington	Muskingum	14	4,267*	1,326	1,539	$ 43,100,000	$28.00
	Kanawha	1	4,565	607	610	14,857,000	24.40
Nashville	Cumberland	1	5,810	1,895	5,782	$ 35,000,000	$ 6.10
Subtotals		22	17,639	4,736	8,872	$142,657,000	$16.10
B. Projects most necessary and desirable							
Pittsburgh	Monongahela	1	366	61	63	$ 4,919,000	$78.20
	Allegheny	4	4,189	1,369	1,600	70,814,000	44.30
	Beaver	4	1,029	318	331	19,887,000	60.10
Huntington	Little Kanawha	3	576	303	304	$ 8,918,000	$29.40
	Muskingum	3	1,268	408	433	13,731,000	31.80
	Kanawha	6	0*	991	1,697	61,866,000	36.50
	Big Sandy	4	852	346	371	13,760,000	37.10
	Twelve Pole Creek	1	138	86	86	2,794,000	32.50
	Guyandot	1	270	140	140	3,478,000	25.00
	Hocking	1	90	42	42	3,562,000	84.80
Cincinnati	Licking	2	2,264*	1,144	1,170	$ 8,603,000	$ 7.40
	Scioto	5	1,822	441	445	17,714,000	40.00
	Kentucky	3	4,440*	1,511	1,538	17,293,000	11.20
	Little Miami	2	579	195	212	8,045,000	38.00
	Great Miami	2	1,080	415	450	19,190,000	42.60
Louisville	Green	5	6,630*	4,511	9,000	$ 42,542,000	$ 4.70
	Wabash	5	9,904	2,990	3,075	30,207,000	9.80
Nashville	Cumberland	5	5,805	2,152	5,845	$ 53,800,000	$ 9.20
Subtotals		57	41,302	17,423	26,802	$401,123,000	$23.10 (flood control) $15.00 (total)
Totals		79	58,941	22,159	35,674	$543,780,000	$24.50 (flood control) $15.30 (total)

* Not including areas controlled by other reservoirs above.

The total cost of the 79 reservoir projects and local flood-protection work in the Ohio Basin will thus be about $810,000,000.

Table 11·1 does not include data for reservoirs of the Tennessee Valley Authority in the Tennessee River Basin, which include 24 reservoirs, totaling about 15 million acre-ft of capacity, at an (allocated) cost of about $78,000,000. These reservoirs are a part of the Ohio River reservoir system (see page 281).

The stage reduction along the Ohio River from the operation of the comprehensive reservoir system is stated to be about as follows:[1]

Location	Flood of	Crest stage, ft	Reduction, ft
Pittsburgh	March, 1913	33.6	8–9
Wheeling	March, 1936	55.5	8.5
Cincinnati	January, 1937	80.0	3.3–4.3
Louisville	January, 1937	57.1	2.2
Evansville	January, 1937	53.7	2.0
Paducah	January, 1937	60.8	2.4
Cairo	January, 1937	59.5	1.8*

* Reduction with New Madrid floodway in operation.

These stage reductions were prepared for the 74-reservoir system proposed or under way in 1937, as noted previously, but would apply approximately to the 79-reservoir system as of 1941.

A list of reservoirs for the Pittsburgh district as of 1945 is given in Table 11·2. This includes costs of the eight reservoirs constructed to date and a revised list and data for six reservoirs as then proposed with costs as far as available. As will be noted, these differ from the Pittsburgh district group of 1941 in Table 11·1, the revised group having a greater total storage capacity and cost.

Tygart River Reservoir and Dam near Grafton, W. Va. This dam site is located on the Tygart River about 23 miles above its mouth, near the town of Grafton, W. Va., and about 150 miles south of Pittsburgh.

The dam was authorized in 1934 and is fully completed. It is 207 ft high above the river bed at the uncontrolled spillway section, which is flanked by abutment sections 23 ft higher. Total length is 1,896 ft. The drainage area is 1,183 sq miles and the total storage 289,600 acre-ft, of which all but 11,000 acre-ft is available for flood control and during periods of low water makes up the deficiency in the Monongahela River. Two penstocks are contained in the dam for future hydroelectric power.

[1] Comprehensive Flood Control Plan for Ohio and Lower Mississippi Rivers, Committee on Flood Control, H.R. Com. Doc. 1, 75th Cong., 1st Sess., p. 12.

Table 11·2 Ohio River—Reservoirs in Pittsburgh District

Reservoir	Stream	Basin	Drainage area, sq miles	Effective storage capacity, acre-ft		Total, including dead storage	Cost	
				Flood control	Low water or power		Total	Per acre-ft
Constructed (8)								
Tionesta	Tionesta Creek	Allegheny	483	125,600	....	133,400	$ 5,200,000	$38.80
Mahoning	Mahoning Creek	Allegheny	341	69,700	....	74,200	6,500,000	87.50
Crooked Creek	Crooked Creek	Allegheny	278	89,500	....	94,000	4,300,000	45.60
Loyalhanna	Loyalhanna	Allegheny	291	93,300	....	95,300	5,400,000	56.60
Tygart	Tygart	Monongahela	1,183	278,400	....	289,600	18,500,000	64.00
Youghiogheny	Youghiogheny	Monongahela	435	150,000	100,000	254,000	9,600,000	37.60
Berlin	Mahoning River	Beaver	275	11,500–116,500	105,000	118,300	7,300,000	66.00
Mosquito	Mosquito Creek	Beaver	97	33,000	69,000	107,000	4,600,000	43.00
Subtotal			3,383	851,000	274,000	1,165,800	$61,400,000	$53.00
Proposed (6)								
Conemaugh	Conemaugh River	Allegheny	1,351	270,000	....	274,000	$21,400,000	$78.00
Allegheny	Allegheny River	Allegheny	2,190	910,000	195,000	1,125,000	30,600,000	27.20
Mill Creek	Clarion River	Allegheny	833	222,000	544,000	856,000		
Rowlesburg	Cheat River	Monongahela	941	251,000	492,000	890,000	22,700,000	20.20
Shenango	Shenango River	Beaver	434	113,000	....	127,000		
Eagle Creek	Eagle Creek	Beaver	95	33,000	59,000	99,000		
Subtotal			5,844	1,799,000	1,290,000	3,371,000		
Totals			9,227	2,650,000	1,564,000	4,536,800		

The estimated cost (1940) was $18,500,000 with $45,000 for yearly maintenance and operation. In Fig. 11·1 is a general view of the Tygart Dam looking upstream.

Loyalhanna Reservoir, Pennsylvania. This reservoir is located on the Loyalhanna Creek about 4 miles above the junction of the Conemaugh River and about 29 miles east of Pittsburgh. The dam is of concrete with a gate-controlled center spillway flanked by abutment sections and an

FIG. 11·1. Tygart Reservoir and Dam near Grafton, West Virginia.

earth embankment section terminating in the left abutment. In Fig. 11·2 is a view of the dam from downstream, showing the five-gate spillway, each gate about 32 ft wide by 29 ft high, and traveling crane control. The dam is 113 ft high above stream bed and 960 ft along the top and provides total storage of 95,000 acre-ft, of which all but 2,000 acre-ft are available for flood prevention in Pittsburgh and the upper Ohio Valley. One penstock is contained in the structure for future hydroelectric power. The project was authorized in 1936 and is completed. Estimated cost (1943) was $5,400,000.

Miami River Conservancy District (1917-1923). The flood of March, 1913, centering in Ohio (see page 106) with great loss of life and great property damage, stimulated interest in measures for flood control. The first tangible result was the formation of the Miami Conservancy

District and the planning and construction of five large reservoirs of the retarding-basin type, built between 1917 and 1923, as well as river-channel works for local river protection in Dayton, Hamilton, and other municipalities (see Fig. 11·3 and Fig. 11·4).

The Miami Conservancy work is a splendid example of comprehensive and thorough investigation and sound conclusions as to the best methods of procedure in the solution of the flood problem. A by-product of this

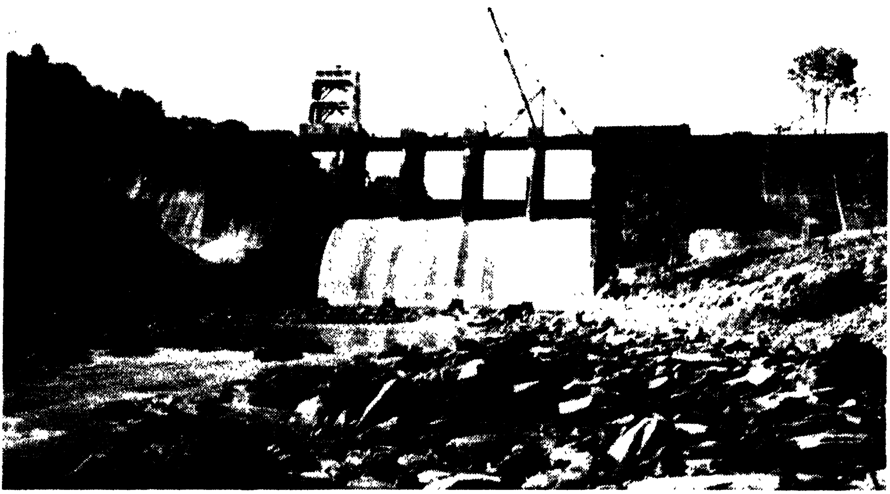


FIG. 11·2. Loyalhanna Dam.

work has been a series of technical reports in five volumes dealing with the history of the project, storm rainfall, flow in open channels, hydraulics of the project, as well as costs of the different features. These reports are a most valuable contribution to the subject of flood control and have been widely used.

Reservoirs. Details of reservoir characteristics and cost are given in Table 11·3. As will be noted from Table 11·3, the total storage capacity of the project drainage area above Hamilton of 3,500 sq miles is 843,000 acre-ft, or 241 acre-ft per sq mile, or 4.5 in. over the drainage area. This is fairly typical of usual storage requirements for retarding-basin-type reservoirs in the North and East. These reservoirs all had earth dams and concrete spillways and were constructed with free conduit outlets, without any gates. The cost of $25.60 per acre-ft of storage was relatively low.

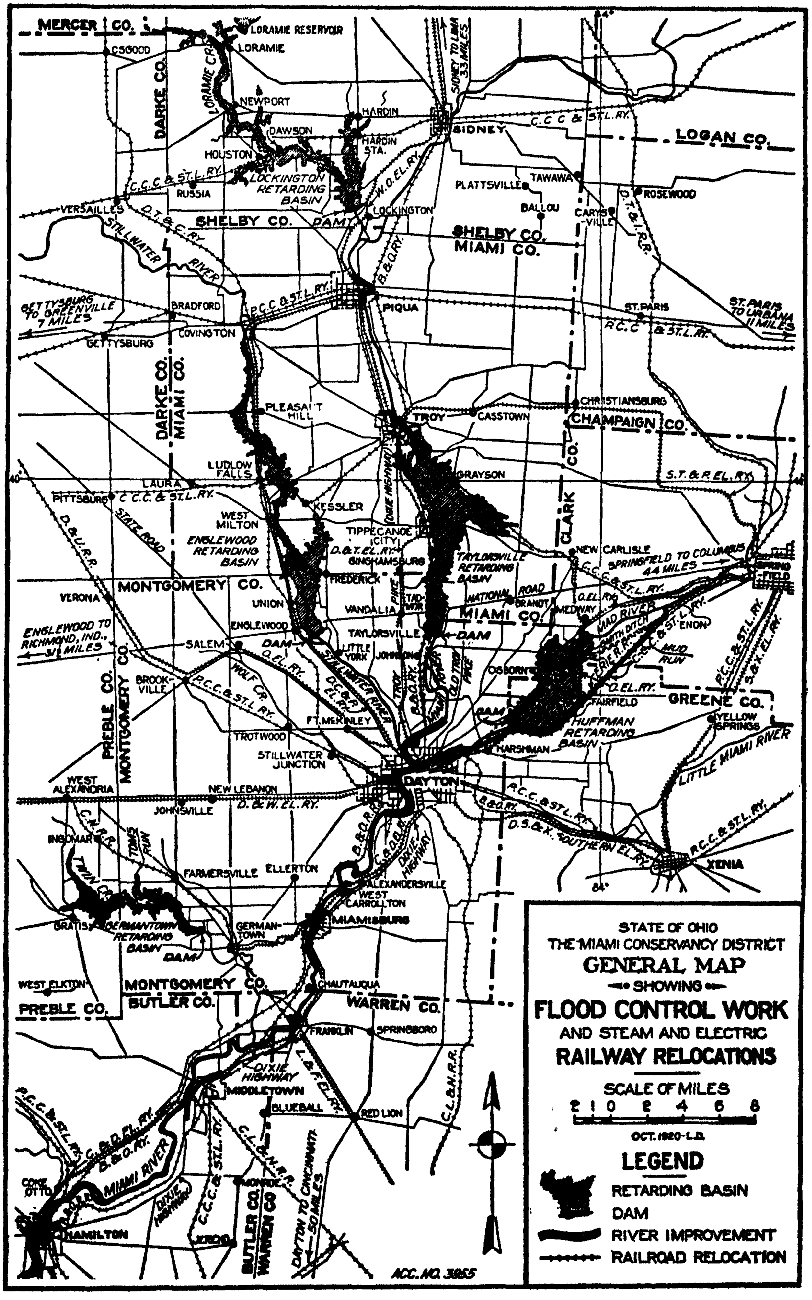
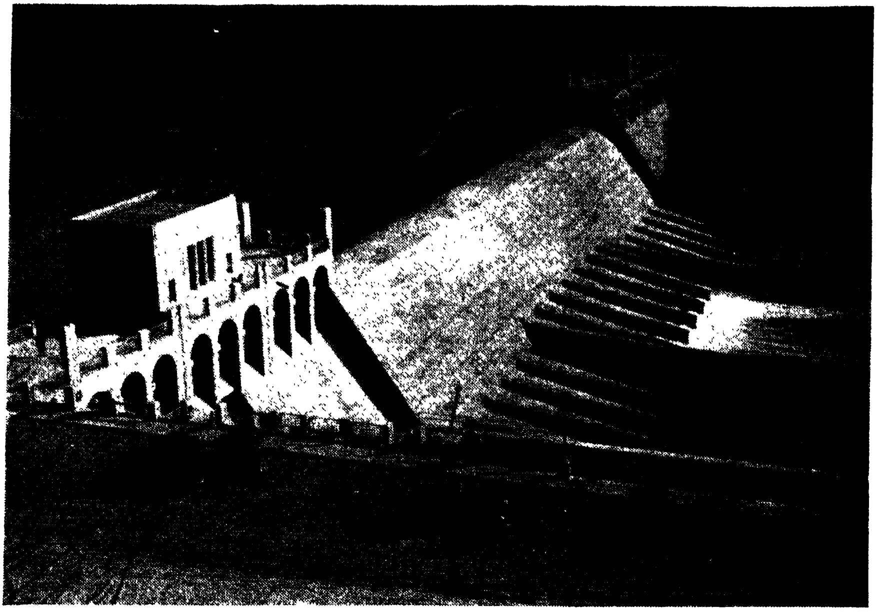


Fig. 11·3. Miami Conservancy District, general map.



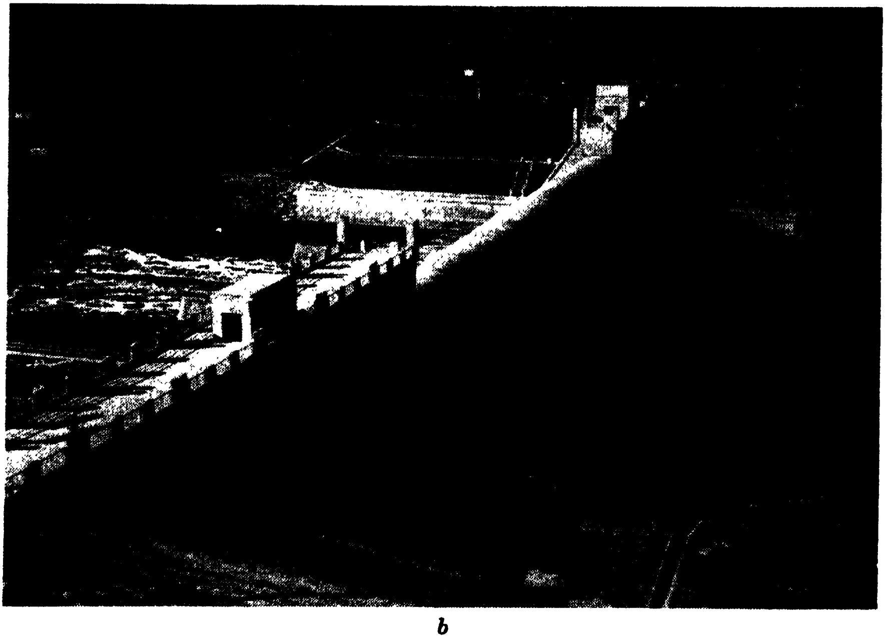


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FIG. 11·4. Miami Conservancy District, Dover Dam. *a*, From right end downstream. *b*, From left end upstream.

Table 11·3 Reservoirs of Miami Conservancy District

Reservoir	River	Drainage area, sq miles	Reservoir Capacity			Dams		Cost of reservoir (except land and overhead)	
			Acre-ft		In. depth on drainage area	Max. height, ft	Length, ft	Total	Per acre-ft
			Total	Per sq mile					
Lockington	Loramie Creek	255	69,000	271	5.0	74	6,400	$ 1,324,800	$19.30
Taylorsville	Miami	878*	186,000	212	4.0	78	3,000	3,492,900	18.70
Englewood	Stillwater	651	313,000	480	9.0	125	4,700	3,792,800	12.10
Huffman	Mad	671	169,000	251	4.7	73	2,300	5,809,200	35.00
Total above Dayton	Miami	2,500	737,000	295	5.5				
Germantown	Twin Creek	270	106,000	393	7.3	110	1,300	1,378,800	13.00
Total above Hamilton	Miami	3,500	843,000	241	4.5			$15,798,500	$18.60

* Not including area above Lockington Reservoir.

NOTE: In addition to the above cost, there was a total item of real-estate damages, etc., of $5,238,400; also administrative and general expense, totaling $1,182,200, of which about one-half is chargeable to reservoirs. This adds a total of about $5,829,500, making a total cost of $21,628,000 or 25.60 per acre-foot. Local river improvements cost about $8,100,000, making the total cost of the Miami project $29,728,000, or about $30,000,000.

The cost index (1917–1923) was approximately 200 per cent of 1913 costs; as compared with a present (1945) cost of about 300 per cent.

The cost of local protection works brought the total cost of the Miami Conservancy works up to about $30,000,000, or about $8,500 per square mile, for the drainage area above Hamilton. This is only about 40 per cent of the losses in the maximum flood of 1913 (see Table, page 156) and reflects an economical solution of the flood problem.

It has been found that land within the confines of reservoir flowage yields much the same return—three-fourths or more—as that outside flowage limits. The operation of these occasionally flowed lands is upon a cooperative basis between the farmer and the district—the former supplying labor, equipment, and half the seed, and the latter supplying half the seed and also attending to product sales, which are divided equally. On the Germantown and Lockington sites, nearly all the land has been sold back to the farmer. Of a total of 30,000 acre-ft acquired for the whole project, some 17,000 acres have been sold back.

This initial Miami flood-control project is an outstanding example of efficient and economical cooperative planning for construction and operation and reflects great credit upon its executives and engineers.

Pueblo Conservancy District (1924). The severe flood of June, 1921, on the Arkansas River in Colorado, in the extreme westerly portion of the Mississippi River Basin (see page 109), resulted in the formation of a conservancy district and construction of flood-protection works at and near the city of Pueblo, Colo., where many lives were lost in 1921 and damages incurred of about $19,000,000. This flood-control project closely followed and patterned that of the Miami Conservancy District in Ohio and is accordingly here described.

These works include a barrier dam forming a retarding-basin-type reservoir on the river above Pueblo at Rock Canyon and a new river channel through the city of Pueblo formed partly by excavation and partly by a levee faced with concrete, with a capacity of 100,000 cfs.

The reservoir has a capacity of about 24,000 acre-ft and a drainage area of 4,800 sq miles. Under extreme floods, about 100,000 cfs will pass the dams as compared with a possible total peak of about 160,000 cfs. The barrier dam has a 500-ft concrete spillway about 50 ft high with earth wings. Instead of outlet conduits, an opening 75 ft wide is left in the dam at the river channel, and another opening about 50 ft wide is left at the location of the Denver and Rio Grande railroad, through the dam. These openings are designed to act as restricted channels with a maximum flow of 100,000 cfs. At this point, the railroad is about 17 ft above ordinary river level; and, since the main opening in the dam will carry all but large floods, interruption in railroad service will seldom occur and will not be frequent enough to warrant relocation at a higher level.

The cost of the Pueblo project is about $4,500,000, of which only about 10 per cent was included in the dam. The project is therefore essentially river-channel improvement with necessary railroad and bridge changes, etc. It appears to be an effective and economical solution of the flood problem.

The Muskingum Valley Conservancy District (1934-1938). Following some time after the Miami River Flood Control Project in Ohio, and that at Pueblo, Colo., the Muskingum River Project was initiated in 1933. This valley, in southeastern Ohio, has been subject to damaging floods ever since its settlement. The great flood of 1913, though not in general so severe as in the Miami Valley, caused direct damages estimated at over $13,000,000 in the Muskingum. The August, 1935, flood caused a damage of about $6,000,000, mostly to agricultural crops.

After the 1913 flood, several communities made studies of methods of relieving flood conditions, but the usual result of such studies was that complete control of the floods would cost more than the communities could bear. An organization called the Muskingum-Tuscarawas Improvement Association made surveys and investigations and formed a plan for flood control and water conservation for the entire valley, estimated to cost $50,000,000.

The Muskingum Valley Conservancy District was formed in 1933, under the Ohio conservancy law. This organization prepared a comprehensive plan of flood control and conservation reservoirs, supplemented by channel improvement and local protection works. It also included reforestation and soil-erosion control. The total estimated cost of this plan was $35,000,-000. The actual cost of the final works was $43,500,000, of which about $500,000 was for levees.

In view of the flood and navigation benefits in the lower Ohio and Mississippi rivers, the district was granted a total of $27,190,000 by the Public Works Administration, the planning and construction of the projects to be handled by the U.S. Corps of Engineers. The state contributed the expense of highway relocation of $8,920,000 and also $2,000,000 toward the cost of lands and rights of way. This left about $7,000,000 to be raised by a levy on the properties benefited by the project.

Construction was started late in 1934, and the 14 dams in the original plan have been completed. Three additional dams and reservoirs are now contemplated with a total of 433,000 acre-ft of storage capacity, estimated to cost about $43,000,000.

Characteristics of the reservoirs and costs are given in Table 11·4. The

Table 11·4 Muskingum Valley Conservancy District Reservoirs

Name	Stream	Drainage area, sq miles	Dam		Storage capacity, acre-ft		Flood-control capacity, in., on drainage area	Cost		
			Length, ft	Max. height, ft	Flood control	Total		Total	Per acre-ft	Per sq mile drainage area
1. Atwood	Indian Fork	70	3,700	65	26,100	49,700	7.0	$ 1,399,920	$28	$20,000
2. Beach City	Sugar Creek	300	5,600	60	70,000	71,700	4.4	4,043,185	56	13,000
3. Bolivar	Sandy Creek	502	6,300	80	149,600	149,600	5.6	5,828,139	39	12,000
4. Charles Mill	Black Fork	216	1,390	50	80,600	88,000	7.0	2,091,515	24	10,000
5. Clendening	Brushy Fork	70	950	60	26,100	54,000	7.0	1,413,179	26	20,000
6. Dover	Tuscarawas River	777*	820	80	202,000	203,000	5.0	7,591,222	37	10,000
7. Leesville	McGuire Creek	48	1,695	65	17,900	37,400	7.0	1,068,779	29	22,000
8. Mohawk	Walhonding River	817*	2,330	115	285,000	285,000	6.5	6,253,071	22	8,000
9. Mohicanville	Lake Fork	269	1,220	46	102,000	102,000	7.0	1,115,973	11	4,000
10. Piedmont	Stillwater Creek	84	1,750	55	31,400	65,000	7.0	1,681,790	26	20,000
11. Pleasant Hill	Clear Fork	199	775	130	74,200	87,700	7.0	1,988,935	23	10,000
12. Senecaville	Seneca Fork	121	2,350	50	45,000	88,500	7.0	2,058,848	23	17,000
13. Tappan	Little Stillwater Creek	71	1,550	52	26,500	61,600	7.0	3,425,004	56	48,000
14. Wills Creek	Wills Creek	723*	2,500	60	190,000	196,000	4.9	3,126,125	16	4,000
Total or mean		4,267	. . . . .	. . . . .	1,326,400	1,539,200	5.8	$43,085,685	$28	$10,000

* Not including areas controlled by upstream reservoirs.

Dover Dam is of concrete masonry and is shown in Fig. 11·3*a*; all the other dams constructed are of the earth-fill type. They are all gate-controlled, and 11 have conservation pools. Of the total area of 8,038 sq miles in the Muskingum Valley, the 14 reservoirs control 4,267 sq miles. The total capacity is 1,539,200 acre-ft, of which 1,326,400 is for flood control and the remainder for conservation. Nine of the reservoirs can store a runoff of 7 in. from their respective drainage areas, and the whole group can store an average of 5.8 in. over the entire controlled area. The operation and maintenance of the system are in the hands of the Muskingum Conservancy District, but the method of operation is subject to the approval of the U.S. Corps of Engineers.

References

"Working Together in the Muskingum Valley," The Muskingum Watershed Conservancy District. (Unpublished.)

Civil Eng., January, 1936, p. 1.

Eng. News-Record, June 21, 1934, p. 799.

Eng. News-Record, Nov. 14, 1935, p. 689.

"Pertinent Data Concerning Project Works," U.S. Engineers Office, Zanesville, Ohio.

Reservoir Operation above Pittsburgh and Wheeling for Flood Control.[1] Reservoir construction for flood control upon the upper Ohio River has progressed sufficiently to show results. At the beginning of 1945, four reservoirs were in operation in the Allegheny Basin; two in the Monongahela, and two in the Beaver (effective below Pittsburgh).

In Table 11·5 are outlined the effects at Pittsburgh and Wheeling that each of the existing reservoirs has as *observed* in the flood of December, 1942, and also what this effect *would have been* in the floods of 1936 and 1937 if these reservoirs had then been operated for flood control. Similar figures are also given for the six proposed reservoirs (see Table 11·5) and finally the combined effects of all existing and proposed reservoirs.

It is evident that, even with this complete group of reservoirs in operation, floods would not be completely eliminated, although all floods except those of great magnitude would be controlled, and the latter would be reduced so as to be of less consequence. Thus the flood stage of 1936 at Pittsburgh would be reduced by the 14-reservoir operation in Table 11·5 from 46.0 ft to 32.2 ft; at Wheeling from a stage of 55.2 ft to 41.3 ft.

Since further reservoir developments are difficult, it is likely that to eliminate all flood damages completely may require local flood-protection works where this is found necessary.

[1] SCHULEEN, E. P.: Control of Floods at Pittsburgh, *Civil Eng.*, October, 1945, pp. 455–458.

Table 11·5 Flood Reductions, in Feet, at Pittsburgh and Wheeling due to Reservoirs

Reservoir	Pittsburgh, Pa.			Wheeling Lock (12), W. Va.		
	Dec. 31, 1942	Jan. 26, 1937	Mar. 18, 1936	Dec. 31, 1942	Jan. 26, 1937	Mar. 19, 1936
Existing:						
Tionesta Creek	0.7	0.7	0.6	0.8	0.7	0.6
Mahoning Creek	0.5	0.3	0.9	0.6	0.3	0.9
Crooked Creek	0.5	0.2	0.7	0.6	0.3	0.7
Loyalhanna Creek	0.4	0.3	1.3	0.5	0.3	1.3
Tygart River	0.5	0.8	1.2	0.6	0.8	1.3
Youghiogheny River	0.2	0.3	0.8	0.2	0.4	0.8
Berlin	...	.	...	0.2	0.3	0.0
Mosquito Creek	...	..	.	0.1	0.1	0.0
Subtotal	2.8	2.6	5.5	3.6	3.2	5.6
Proposed:						
Conemaugh River	2.2	0.6	4.6	2.6	0.6	4.6
Allegheny River	1.2	1.0	0.6	1.5	1.2	0.6
Mill Creek (Clarion River)	0.9	0.8	1.9	1.1	0.8	1.9
Rowlesburg (Cheat River)	1.1	0.7	1.2	1.3	0.8	1.2
Shenango River	...	...	...	0.5	0.6	0.0
Eagle Creek	...	..	...	0.1	0.1	0.0
Subtotal	5.4	3.1	8.3	7.1	4.1	8.3
Total	8.2	5.7	13.8	10.7	7.3	13.9

Tygart Reservoir Operation.[1] The Tygart Reservoir on the tributary of the Monongahela, as previously noted, was constructed in 1937. Its operation since that time, as a multiple-use reservoir, is of interest.

In 1930, the upper reaches of the canalized Monongahela almost went dry, several of the pools losing their entire head owing to inadequate stream flow, estimated at as low as 5 cu ft per sec. Only by using some 65,000 acre-ft stored in a private Cheat River Reservoir was there sufficient flow for leakage and lockage in this most heavily traveled river in the country, which normally carries some 20-odd million tons of commerce annually. Also, the water supply for industrial and domestic uses and for sanitation in this highly industralized and thickly populated valley would have been inadequate.

[1] BOWDEN, N. W.: Multiple Reservoir Operation, *Civil Eng.*, May, 1941, pp. 2–3.

The Tygart multiple-use reservoir on a tributary of the Monongahela, about 150 miles above Pittsburgh, not only fills these needs but serves other useful purposes. With a drainage area of about 1,180 sq miles, it has a controlled capacity of about 290,000 acre-ft; all except about 11,000 acre-ft retained for a conservation pool, is available for flood control from Dec. 15 to Apr. 1 each year. During this period, floods are temporarily held in the reservoir and then released as rapidly as it is safe to restore the flood-storage space. Beginning early in April, after which time damaging floods are less likely to occur, filling is started (Fig. 11·5) with a view to impound-

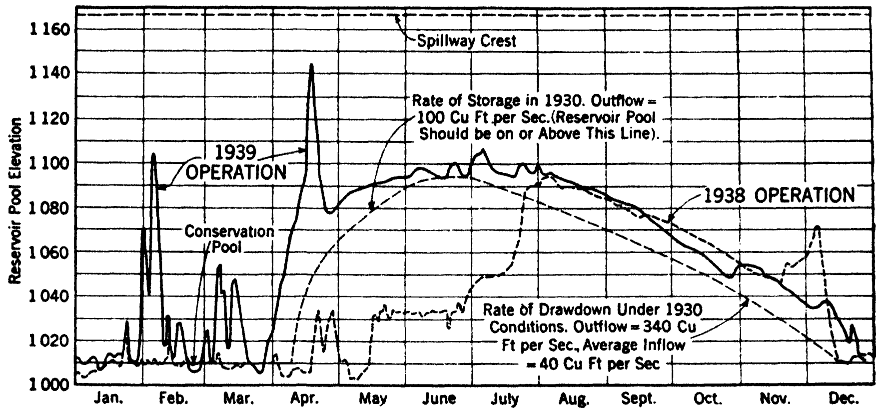


FIG. 11·5. Tygart Reservoir operation, 1938–1939.

ing 100,000 acre-ft by June 15 for release during the low-flow season in the late summer and fall. The remaining storage space of some 180,000 acre-ft is always available for flood control, as is an additional 87,000 acre-ft of partly controlled storage between the spillway crest and the top of dam.

Depending upon flows in the Monongahela River, the rate of discharge is varied, care being taken to maintain sufficient storage to ensure a minimum outflow of 340 cfs to the end of the low-water season. Following an operating schedule based on flows in the extreme low-flow year of 1930, the level is kept above the filling and emptying guide for that year (Fig. 11·5). The late impounding of storage in 1938, the first year of operation, was due to delay in adjusting certain flowage items.

Since its completion and up to early in 1940, the reservoir has reduced the crests of five floods on the Ohio River at Pittsburgh an average of about 1 ft and on the Monongahela River an average of about 3 ft, with resulting benefits estimated at about $3,000,000; it has also maintained adequate flows during the low-water seasons. In 1939, it successfully met a drought situation almost as severe as that of 1930. At times, it supplied about

60 per cent of the total discharge of the Monongahela, thus maintaining traffic and greatly improving municipal and industrial water supply. At the end of the low-flow season, there still remained in the reservoir sufficient water to permit further releases for a considerable period.

Missouri River[1]

The Missouri River has its source in southwestern Montana and flows for 2,460 miles through seven states, emptying into the Mississippi River

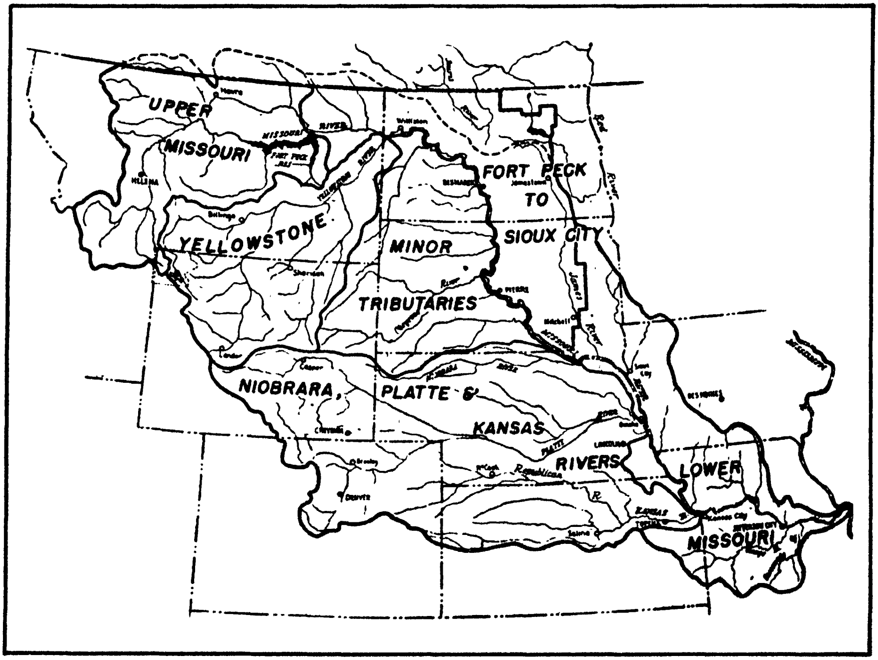


FIG. 11·6. Missouri River drainage area.

17 miles above St. Louis. It drains 529,350 sq miles consisting largely of plains but including also easterly slopes of the Rocky Mountains and other rugged areas. It has probably a more diverse terrain than any other major river system in the United States. About 60 per cent of the drainage (314,617 sq miles) is upstream from Sioux City, Iowa, 760 miles above the river mouth. The principal tributaries below Sioux City are the Platte and Kansas Rivers from the west and the Grand, Osage, and Gasconade Rivers in Missouri (see Fig. 11·6).

[1] H.R. Doc. 475, 78th Cong., 2d Sess., 1944; also Sen. Doc. 191, 78th Cong., 2d Sess., 1944; also *Civil Eng.*, October, 1944, p. 413.

The average annual precipitation ranges from 26 in. at Sioux City to 40 in. at the river mouth. The soils are fertile, and agriculture predominates. In addition to Sioux City, Kansas City, and the cities of Omaha, Nebr., and Council Bluffs, Iowa, are important industrially and have important railroad facilities.

Congress has authorized improvement of the Missouri River for navigation to secure a minimum low-water depth of 6 ft between the mouth and Sioux City by means of bank revetment, dikes, and stabilization of the waterways by dredging. The estimated cost of new work for this navigation project, which includes a length of about 761 miles, is $172,000,000, with a yearly maintenance charge of $4,500,000.

In 1933, work began upon the Fort Peck Reservoir project upon the Missouri River in eastern Montana, about 11 miles above the mouth of the Milk River, for the improvement of navigation below Sioux City. The drainage area at this point is 57,725 sq miles. The dam is about 250 ft high and 21,000 ft long, of earth (hydraulic fill), requiring about 125 million cu yd (the largest earth dam yet constructed). It has a concrete channel spillway about 1 mile long, controlled by 16 Stoney gates each 25 ft high by 40 ft wide. The reservoir has an area of about 380 sq miles and a maximum capacity of 19,400,000 acre-ft, or about 6.4 in. depth above the site. Normal operating capacity is 18,400,000 acre-ft. Power is developed by 50,000 kw of hydraulic turbines in two units. The project was completed in 1944. Its total cost was about $124,000,000.

There are two types of floods on the Missouri River, known as the March and June floods from the time of their general occurrence. The March floods result from melting snow in the plains area above Sioux City and are usually accompanied by a small amount of precipitation. The June floods result from snow thaws in the headwater mountains accompanied by heavy rainfall. Between Sioux City and the mouth, 10 floods, severe in character, have occurred during the past 100 years. About 1,800,000 acres of land, largely cultivated and productive, are subject to inundation at extreme river stages; and important areas in the principal cities are included in the flood plain. In March-June, 1943, severe floods occurred that overtopped or caused failure of nearly all the levees below Sioux City. These floods caused direct damages estimated at $40,000,000 and indirect damages of about $8,000,000, and 11 levees were lost. See Chap. 20 for discussion of floods which occurred in 1947.

Various plans for river works as well as reservoirs have been approved from time to time by Congress, but the work has been interrupted during the war, and relatively little progress has been made. The plans for flood-control works proposed in 1944 by the Chief of Engineers include the following:

1. Nine authorized reservoirs with about 12 million acre-ft of storage capacity estimated to cost about $156,000,000. These include Kanopolis upon the Smoky Hill River near Ellsworth, Kan. (450,000 acre-ft, $9,000,000); Harlan County on the Republican River, Nebr. (850,000 acre-ft, $31,000,000); Turtle Creek or the Big Blue River near Manhattan, Kan. (1,470,000 acre-ft, $28,500,000); Osceola near Osceola, Mo. (4,500,000 acre-ft, $28,500,000); Chilicothe on the Grand River, Missouri ($1,470,000 acre-ft, $28,500,000); and four smaller reservoirs.

2. Seven proposed multiple-purpose reservoirs above Sioux City as follows:

Project	Location	Height of dam, ft	Storage capacity, acre-ft	Cost
Garrison	Missouri River, Garrison, N. D.	195	17,000,000	$180,000,000
Oak Creek	Missouri River, Mowbridge, S. D.	135	6,000,000	60,000,000
Oahe	Missouri River, Pierre, S. D.	135	6,000,000	50,000,000
Fort Randall	Missouri River, Wheeler, S. D.	165	6,000,000	75,000,000
Gavins Point	Missouri River, Yankton, S. D.	35	200,000	15,000,000
Lower Canyon	Yellowstone, Livingston, Mont.	305	2,300,000	35,000,000
Boysen	Big Horn, Thermopolis, Wyo.	217	3,500,000	20,000,000
Total 7 reservoirs			41,000,000	$335,000,000

The group of proposed reservoirs agreed upon by the U.S. Engineers and Bureau of Reclamation in 1944 as essential for flood control below Sioux City includes Garrison, Oahe (much enlarged), Big Bend, Fort Randall, and Gavins Point reservoirs with a total storage capacity of 42 million acre-ft.

3. Five authorized reservoirs in the Republican River Basin totaling 367,000 acre-ft of capacity and estimated to cost $25,000,000.

4. The total cost of the complete plan is estimated at $566,000,000 for reservoirs and $95,000,000 for levees and appurtenant works (with revised plans at the Kansas Cities), or a grand total of $661,000,000. Total storage capacity including Fort Peck Reservoir would be about 72,000,000 acre-ft, or about 4.3 in. for the drainage area above Sioux City.

The cost of flood control as planned in the Missouri River Basin will be very high. The maximum flood damage in the basin did not exceed about $60,000,000, whereas the proposed cost of flood-relief works totals $660,000,000, a ratio of cost to maximum damage of about 11 to 1. This would seem to indicate that a much less costly solution of the flood problem should be worked out, if possible.

A multiple-purpose development of the Missouri River has been planned and made the subject of a report by the Bureau of Reclamation of the Department of Interior.[1] This plan is for the conservation and control of the water resources of the entire Missouri River Basin, including navigation, flood control, irrigation, and power. The plans for flood control below Sioux City, as previously described, are a part of this general project. In the upper reaches of the river and its tributaries, additional reservoirs are planned that will be of some service on the lower river with reference to flood control and navigation; but, more particularly, they are to enlarge the supply of water available for irrigation and to develop power that will be used for pumping water in connection with irrigation and with surplus power sold for domestic, municipal, and industrial use. As a matter of convenience in making studies for this report, the river drainage area has been subdivided, as shown in Fig. 11·6.

Some of the details of the proposed development in the various subdivisions of the basin are shown in Table 11·6.

Population. For the whole basin, population is about 6,840,000, with 30 per cent or more in the two subdivisions of Niobrara-Platte and the lower Missouri. The present amount of irrigation (not shown in Table 11·6) is about 1,157,000 acres. It is proposed to irrigate a total of about 5,700,000 acres, all of which is located above Sioux City.

Reservoirs. A total of about 75 million acre-ft storage capacity is planned in 99 reservoirs, of which 9 reservoirs with about 30 million acre-ft of capacity are those previously described in the lower Missouri section.

Power. About 758,000 kw of capacity in 17 power plants is planned to yield about 3,809,000 kw yearly of firm power. This includes the power development of 35,000 kw at Fort Peck Reservoir, which has already been constructed. The 17 power plants are estimated to cost about $288,000,000, or $378 per kw capacity, and to produce nearly 4 million kwhr yearly of firm power.

The following developments are contemplated in the revision of main-river storage-reservoir projects during 1945-1946.[2]

Garrison Reservoir will include a very large rolled-fill dam with an estimated 75 million cu yd of embankment, with a crest length of over 2

[1] Sen. Doc. 191, 78th Cong., 2d Sess.

[2] Garrison Dam on the Missouri River to Make Rolled Fill History, *Eng. News-Record*, June 13, 1946, p. 110.

Table 11·6 Missouri River Multiple-purpose Project—Details

Area	Population	Proposed irrigation, acres	Power			Reservoirs	
			No. plants	Kw capacity	Kwhr yearly	No.	acre-ft capacity
Yellowstone	212,530	806,000	9	312,000	1,509,000,000	27	4,285,000
Upper Missouri	247,950	679,000	3	79,000	418,000,000	19	3,360,000
Minor tributaries	262,747	224,000	.....	...	....	15	1,237,000
Niobrara, etc.	2,219,225	1,305,864	1	2,000	7,000,000	22	5,650,000
Fort Peck–Sioux City	526,751	2,292,000	4	325,000	1,621,000,000	7	31,180,000
Lower Missouri	3,370,828	....	..	...	...	9	30,000,000
Totals	6,840,034	5,306,864	17	718,000	3,555,000,000	99	75,712,000

miles, and 210 ft high. Its water area will be 390,000 acres, extending 200 miles upstream, and storing about 23 million acre-ft of water.

Oahe Reservoir will store about 20 million acre-ft, making a total of about 49 million acre-ft in the five main-river projects exclusive of Fort Peck Reservoir, as agreed upon in 1944.

Including Fort Peck, the complete multiple-purpose program for the Missouri River Basin will have 106 reservoir projects with a total storage of 105 million acre-ft.

The estimates of benefits and costs are shown in Table 11·7.[1]

Table 11·7 Missouri River Project—Bureau of Reclamation

Summary of Benefits	Annual
Irrigation	$ 130,000,000
Power	17,141,000
Flood control	16,500,000
Navigation	4,165,000
Municipal water	500,000
Total	$ 168,306,000
Annual Costs	
Operation, maintenance, repairs, and replacements:	
Irrigation	$ 7,725,000
Power	4,316,000
Flood control and navigation	4,500,000
Amortization of entire cost of project at 3 per cent in 50 years	48,872,000
Total annual cost	$ 65,413,000
Ratio of annual costs to annual benefits	1:2.57
Repayments and Returns	
Total estimated cost	$1,257,645,700
Allocation to	
Flood control	$ 419,300,700
Navigation	97,245,000
Subtotal	$ 516,545,700
Balance repayable	741,100,000
Repayments from	
Irrigation (40 annual payments)	$ 298,000,000
Power (50 annual payments)*	423,100,000
Municipal (40 annual payments)	20,000,000
Total	$ 741,100,000

* In addition to the repayments indicated, power revenues will also be sufficient to collect the interest charges on the costs allocated to power.

[1] Sen. Doc. 191, p. 27. 78th Cong., 2d Sess.

It will be noted that, in this Missouri River project, irrigation benefits appear to be the principal basis for the assumed justification of the project and make up over 75 per cent of the total yearly benefits.

1. Power benefits are

$$\frac{\$17,141,000}{3,800,000,000} = 0.45 \text{ cents per kilowatt-hour}$$

Allocated power benefits include yearly

		$17,141,000
Operation, maintenance, etc.		4,316,000
Repayment at 3 per cent, 50 years =	$423,000,000	3,800,000
		$25,257,000

$25,257,000 ÷ 3,800,000,000 = 0.67 cent per kilowatt-hour.

From this, the power sale price will be 1.12 cents per kilowatt-hour.

2. Irrigation benefits will represent

$$\frac{\$130,000,000}{5,347,000} = \$24.60 \text{ per acre yearly}$$

3. Flood-control benefits will represent

$$\frac{\$16,500,000}{0.05} = \$330,000,000$$

which is much out of line with past flood damages.

Allocations of the total estimated cost and the yearly benefits are as follows:

Items	Allocated costs		Allocated yearly benefits	
	Dollars	Per cent	Dollars	Per cent
Irrigation	298,000,000	24	130,000,000	77
Power	423,100,000	34	17,141,000	10
Flood control	419,301,000	33	16,500,000	10
Navigation	97,245,000	8	4,165,000	3
Municipal	20,000,000	1	500,000	0
Total	1,257,646,000	100	168,306,000	100

This project depends chiefly for its economic justification upon a successful increase in use of irrigated land of from 1,157,000 to 5,700,000 acres, or to five times the present amount. It appears to be highly speculative in nature as well as involving high cost.

Upper Mississippi River

This includes the portion of the Mississippi River between the Missouri River and the headwaters of the Mississippi in northern Minnesota—a distance of about 1,200 miles with a total drainage area of about 172,000 sq miles and a mean yearly precipitation of about 31 in.

The Federal project for this portion of the Mississippi provides for a channel of 9 ft depth and adequate width. There are 27 locks and dams. At Keokuk, Iowa, the lock provides for a lift of 38 ft, built in 1913. At St. Paul, the lift is 36 ft, built in 1917. Most of the other dams and locks were built between 1930 and 1940 with lifts varying from 6 to 23 ft. At St. Anthonys Falls in Minneapolis, the upper lock, with a lift of about 50 ft at the dam of the Northern States Power Co., is not yet constructed; and the lower lock, with a lift of 26 ft, is not yet completed. The cost of the existing project, including locks and dams and other miscellaneous work, has been $170,000,000.

Headwater reservoirs, six in number, were constructed under the Act of 1880. These were all natural lakes, raised in level, as noted in Table 11·8.

Table 11·8 Headwater Reservoirs in the Mississippi River

Reservoir	River	Drainage area, sq miles	Storage capacity		Cost	
			acre-ft	Acre-ft per sq mile	Total	Per acre-ft
Winnibigoshish	Mississippi	1,442	1,000,000	700	$ 387,470	$0.39
Leech Lake	Leech	1,163	760,000	650	256,185	0.34
Pohegama	Mississippi	660	120,000	180	211,030	1.75
Sandy Lake	Sandy	421	72,000	170	231,020	3.22
Pine River	Pine	562	178,000	320	230,320	1.30
Gull Lake	Gull	287	70,000	250	86,826	1.23
Total or mean		4,535	2,200,000	490	$1,402,851	$0.64
		Surveys and flowage rights			160,939	0.07
					$1,563,790	$0.71

Operating and care for this group of reservoirs has cost about $967,000 over a period of 41 years prior to 1936, or about $24,000 per year.

These are very low unit cost reservoirs. Their storage capacity is about 9 in. of depth upon the 4,535 sq miles of controlled drainage, which affords fairly good regulation. They control the main river only slightly, however, since, at the lower end of this section, for illustration, storage is only about ¼ in. depth over the 172,000 sq miles of drainage area.

Federal flood-control projects include:

Rock Island District Reservoirs:		
Central City, Wapsipinicon River, Iowa	$ 3,000,000	
Coralville, Iowa River	6,000,000	
Rochester, Iowa River	3,950,000	
Howell, Des Moines River	13,550,000	
		$26,500,000
St. Paul District Reservoirs:		
Lacqui Parle, Minnesota River	$ 8,000,000	
Black River, Black River	6,500,000	
		$14,500,000
Total		$41,000,000

Little work has been done, as yet, on these projects.

CHAPTER 12

WHITE, RED, ARKANSAS, AND OUACHITA RIVERS[1]

White River[2]

The White River rises in northwestern Arkansas and flows generally southeasterly 720 miles to join the Mississippi River 45 miles above Arkansas City. It has a total drainage area of 28,000 sq miles, with a population of 750,000. Farming is the principal occupation, with raising of fruits, cotton, hay, and livestock. Its natural resources include forest products, coal, manganese ore, glass sand, natural gas, clay, tripoli, and lead and zinc ores. Navigation improvements are provided for channel maintenance between the mouth and Batesville, mile 301, and for canalization by these locks and dams between Batesville and Guion, mile 331.

Floods. Frequent floods occur and cause considerable damage in the fertile lowlands along the lower river. High stages of short duration are caused by intense local storms in the upper basin; and major floods with extremely high stages, lasting for many days, result from continued general rains and contribute largely to floods of the Mississippi River.

Flood Losses. In the central basin, 73 floods have occurred in 49 years of record, 6 of which were major floods. Area of land subject to inundation is 6,262,000 acres, fixed property is valued at $35,000,000, and crop values average over $8,000,000 yearly. In the major flood of 1927, direct losses were about $16,000,000, including about $12,000,000 for crops and farm losses. Average yearly direct flood losses are estimated at about $2,500,000 (two-thirds of which is in the Mississippi backwater area), with perhaps 25 per cent more indirect losses.

Improvements authorized by Congress for flood control include five levee projects in the main valley and six reservoirs on tributaries, including Clearwater on the Black River and Norfork on the North Fork River, which are under construction (and the latter practically completed), and Bull Shoals Dam on the main river, now under way (see Fig. 12·1).

[1] The flood-control reservoirs upon this group of four rivers are chiefly for multiple-purpose use, and their features are discussed in some detail to show the essentials of such reservoirs. (See Fig. 12·1.)

[2] White River, H.R. Doc. 917, 76th Cong., 2d Sess., 1940; also Norfolk Dam, H.R. Doc. 290, 77th Cong., 1st Sess., 1941.

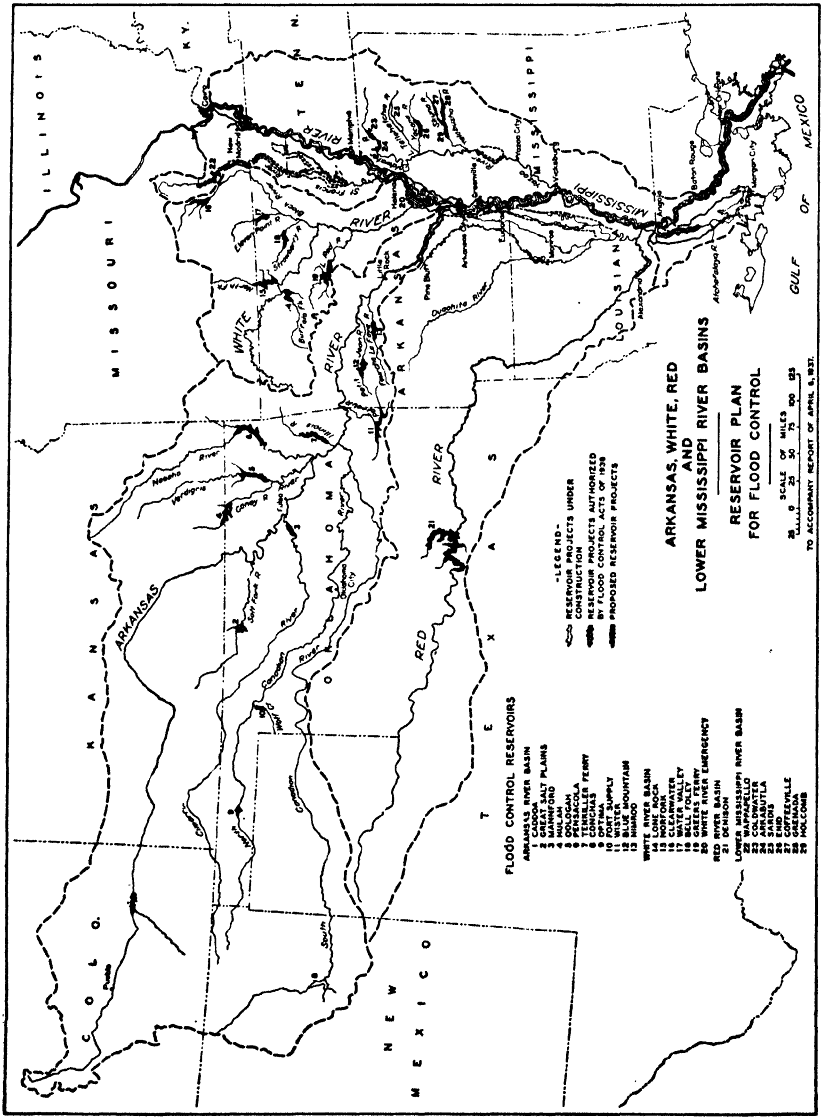


FIG. 12·1. Reservoir plan for Arkansas, White, Red, and Lower Mississippi basins.

Norfork Reservoir. The Norfork Reservoir is located on the North Fork River about 5 miles above its confluence with the White River, with a drainage area of 1,780 sq miles. Storage capacities, etc., are as follows:

Elevation	Level	Storage capacity, acre-ft	In. depth	Water area, acres
580	Total—top of flood-control pool	1,983,000	20.6	30,700
552	Flood control	732,000	7.6	22,000
	Power drawdown	707,000	7.3	
510	Dead storage	544,000	5.7	

The dam is 230 ft in maximum height, with top at El. 590, and is 200 ft thick at the base and 2,600 ft long. Spillway discharge will be controlled by 12 radial gates, each 28 ft high and 40 ft long with sills at El. 652. A roadway runs across the top of the dam at El. 590 with a concrete bridge over the spillway. Eleven conduits for the release of water, each 4 ft wide by 9 ft high, controlled by slide gates, run through the base of the spillway.

Four 18-ft-diameter steel-lined penstocks run through the dam at the right of the spillway to the powerhouse just below the dam (see Fig. 12·2) with provision for two units—with an initial installation of one vertical unit of 47,000 hp at 131 ft head operating at 128.6 rpm, direct-connected with a 35,000-kw generator at 13,800 volts, which will be stepped up for transmission at 154 kv. Tail-water elevation averages El. 378. Space is provided to extend the powerhouse for two more units, so that ultimate capacity may be about 140,000 kw.

The *estimated cost* of the project, revised in 1941, is $24,400,000 for construction and $3,100,000 for lands and damages, or a total of $27,500,000. A flood-control reservoir without power development is estimated to cost $14,000,000. Hence cost chargeable to power is $13,500,000.[1]

Yearly charges for power would be as follows:

Investment charges:	
Increased first cost over flood control	$13,500,000
Interest during construction at 6 per cent.	810,000
	$14,310,000
Yearly cost:	
Interest on $14,310,000 at 3½ per cent	$501,000
Amortization	141,000
Operation and maintenance (over flood control only). . .	70,000
	$712,000

[1] Norfork Dam, H.R. Doc. 290, 77th Cong., 1st Sess., p. 27, par. 69.

Estimate of Average Yearly Power Value.[1] The estimated average total electrical energy at the dam available for transmission amounts to 148,000,-000 kwhr per year. Of this amount, 108,000,000 kwhr are prime (dependable) energy. The remaining 40,000,000 kwhr are classed as "dump" energy, which would be available intermittently, mostly during short flood periods; and there would be periods of a few consecutive years when no

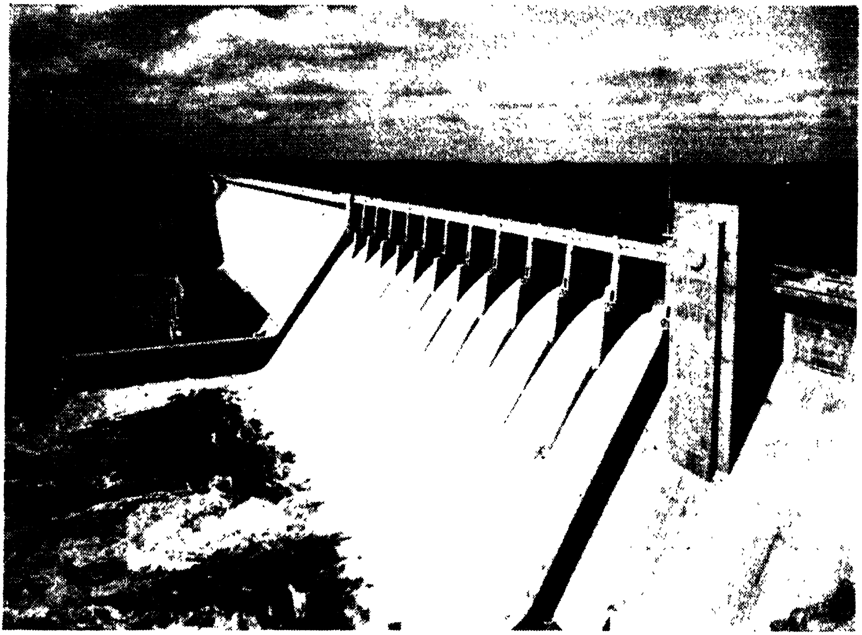


FIG. 12·2. Norfork Dam and Reservoir, White River Basin.

"dump" energy would be available. The proposed initial installation of 60,000 kw would permit the transmission of 58,000 kw of prime capacity for serving peak loads.

The report of the Federal Power Commission gives unit-capacity and energy values. The unit-capacity value is based upon the annual cost of supplying equivalent dependable capacity for serving loads by private concerns, exclusive of the cost of those elements of production which vary with the amount of energy generated, by alternate new fuel-burning plants. The unit-capacity value thus given is $15.40 per kilowatt. The energy value given is 1.11 mills per kilowatt-hour and is based upon the cost of those elements of production by fuel-burning plants, which vary with the amount of energy generated. Transmission losses for the Norfork

[1] *Ibid.*, pp. 27–31, par. 70–83; p. 31, par. 84–85.

power would be greater than for power generated at alternate fuel-burning plants, because the latter could be located at or near load centers. The study by the commission indicates that these losses would average 5,000 kw of capacity and 10,000,000 kwhr per year of energy more then corresponding losses for equivalent power generated in fuel-burning plants. Thus, the prime capacity for serving peak loads and the average annual energy that would be available at or near load centers from the proposed Norfork project amount to 53,000 kw and 138,000,000 kwhr. respectively. Applying the unit capacity and energy values to these amounts, the gross average annual power value for the proposed project is $969,000.

The Federal Power Commission made studies of the relative system production costs with and without the proposed initial Norfork installation. These studies showed that certain economies would result from the operation of an alternate fuel-burning plant and that the gross average annual value of the Norfork power, as measured by equivalent fuel-burning plant cost, would have to be reduced. After the results of the studies and the several factors that would affect the production costs of the area were considered, it was concluded by the commission that a fair average of the annual reduction that should be charged against the initial installation at the Norfork Dam would be about $75,000. The estimated net average annual value of the Norfork power at or near load centers amounts to $894,000.

Justification of Power Facilities in the Norfork Dam. A comparison of the estimated annual power charges with the estimated net average annual value is indicated in the following tabulation:

Estimated annual charges for power development	$712,000
Estimated net average annual power value	$894,000
Ratio of annual power charges to net average annual power value	1 : 1.26

It is apparent that power facilities in the Norfork Dam are economically justified for serving peak loads when based upon the initial installation of 60,000 kw recommended by the Federal Power Commi sion and upon the unit-power values determined by that agency.

Conclusions.[1] The topography and foundation conditions in the North Fork River Valley are suitable for the construction of a dam for flood control or of a higher dam for the dual purpose of flood control and hydroelectric-power generation. The volume and character of the flows in the North Fork River are such as to warrant the consideration of power generation in any plan of developing the stream. According to the report of the Federal Commission, there would be a market for the power that could be

[1] *Ibid.*, p. 31, par. 84–85.

generated at the Norfork site as soon as the dam could be completed. A reservoir for flood control only would not result in the best development of the site, would not utilize the water resources of the stream, and would not result in the greatest rate of return on the investment. Based upon the power values established by the Federal Power Commission and the district engineer's estimated costs of producing the power, the generation of power in conjunction with flood control at the Norfork Dam site is economically justified.

In addition to the direct monetary return to the Federal treasury that would result from the sale of power, the power facilities of the proposed dual-purpose dam would result in other actual and potential benefits. The large permanent power pool would be of greater benefit from a recreational and wild-life standpoint than a small conservation pool in a flood-control reservoir. The regulated flow downstream from the dam occasioned by

Table 12·1 Engineering Features for Table Rock and Bull Shoals Reservoirs, Flood Control and Water Power Combined

Dams	Table Rock	Bull Shoals
Location of dam, miles above mouth of White River	528	420
Drainage area, sq miles	4,000	5,905
Crest length of dam, ft	4,600	2,235
Maximum height of dam above stream bed, ft	206	223
Spillway design flood, cfs	440,000	585,000
Spillway capacity, cfs	380,000	535,000
Maximum peak discharge of record, cfs	186,000	220,000
Top of dam, elevation, ft, mean sea level	905	675
Top of spillway gates, elevation, ft, mean sea level	887	658
Spillway crest, elevation, ft, mean sea level	858.5	628.5
Flood-control pool, elevation, ft, mean sea level	887	658
Power pool, elevation, ft, mean sea level	867	575
Area inundated (flood-control pool elevation), acres	30,000	40,000
Gross storage, acre-ft	1,685,000	2,750,000
Flood-control storage, acre-ft	485,000	2,155,000
Power storage, acre-ft	1,200,000	595,000
Power drawdown storage, acre-ft	780,000	144,000
Power drawdown, ft	50	11
Net maximum head, ft	157	115
Net average head, ft	144	113
Net mimimum head, ft	107	104
Regulated flow, cfs	2,700	3,350
Prime capacity, kw	59,300	71,500
Installed capacity, kw	105,000	80,000
Number of installed generating units	3	4
Prime energy output, kwhr	156,000,000	188,000,000
Total energy output, kwhr	307,000,000	312,000,000

Table 12·2 Estimated Costs for Table Rock and Bull Shoals Reservoirs, Flood Control and Water Power Combined

(Estimated costs, annual charges, and annual benefits for Bull Shoals Reservoirs based upon the regulated flows that would result from the Table Rock Reservoir)

Item	Table Rock	Bull Shoals	Total for the 2 reservoirs
Cost of dam and appurtenances and reservoir:			
Dam (including reservoir clearing, camp, access railroads and highways, and foundation exploration and treatment	$20,447,000	$25,240,000	$45,687,000
Powerhouse and equipment	6,700,000	6,650,000	13,350,000
Power-transmission facilities to existing load-distribution centers	3,400,000	4,387,000	7,787,000
Lands	1,200,000	1,470,000	2,670,000
Highway relocations	2,700,000	140,000	2,840,000
Cemetery relocations	40,000	18,000	58,000
Damage to villages	6,000	94,500	100,500
Damage to miscellaneous structures	7,000	500	7,500
Total construction cost (estimated appropriation of public funds necessary for the execution of the project)	$34,500,000	$38,000,000	$72,500,000
Federal investment:			
Total construction cost	$34,500,000	$38,000,000	$72,500,000
Interest during construction	1,811,300	1,995,000	3,806,300
Total	$36,311,300	$39,995,000	$76,306,300
Present value of Federal properties	1,200	300	1,500
Total Federal investment	$36,312,500	$39,995,300	$76,307,800
Annual charges: interest, amortization, maintenance, and operation	1,642,200	1,815,100	3,457,300
Annual benefits:			
Prevented direct flood losses in White River Basin:			
Present conditions	$ 60,100	$ 266,900	$ 327,000
Future developments	19,000	84,200	103,200
Prevented indirect flood losses owing to floods in White River Basin	19,800	87,800	107,600
Enhancement in property values in White River valley	7,700	34,000	41,700
Mississippi River benefits	220,000	980,000	1,200,000
Annual flood benefits	$ 326,600	$ 1,452,900	$ 1,779,500
Power value	1,415,600	1,403,400	2,819,000
Total annual benefits	$ 1,742,200	$ 2,856,300	$ 4,598,500
Ratio of annual charges to annual benefits	1:1.06	1:1.57	1:1.33

the power pool could be expected to improve navigation conditions and would increase power outputs at any potential downstream hydroelectric plants. The increased low-water flow downstream from the structure might prove beneficial in the future in many other ways, including the possibility of rice irrigation from the White River in the lower part of the basin. The power facilities would be of great benefit to the general development of the area and would conserve natural resources, particularly gas, oil, and coal. Since a large part of the benefits would transcend state boundaries, the power facilities as well as the flood-control features in the proposed Norfork Dam are of national importance. The dual-purpose development of the Norfork site is essential to a comprehensive plan of developing the water resources of the White River Basin.

Table Rock and Bull Shoals Projects.[1] These two projects are located on the White River near the Missouri-Arkansas state line and have the features described in Table 12·1 (see Fig. 12·3)

The estimated construction costs, including overhead and contingencies, investment costs, annual charges, and annual benefits for the proposed developments are shown in Table 12·2.

The Buffalo and North Fork rivers, as well as the upper White River, have been substantial and important contributors to major flood flows that have caused extensive damage in the alluvial valley of the White River. Owing to the large amount of flood runoff that originates in the upper White River Basin, control of the Buffalo, North Fork, and upper White Rivers is essential in any comprehensive reservoir plan for the reduction

Table 12·3 Natural and Modified Peak Stages and Discharges at Lock and Dam No. 3, White River, Arkansas

(Drainage area 10,500 sq miles, channel capacity 73,000 cfs, flood stage 14.4 ft)

Flood year	Natural		Modified by Lone Rock and Norfork Reservoirs		Modified by Table Rock and Bull Shoals Reservoirs		Modified by the 4 reservoirs	
	Stage, ft	cfs	Stage, ft	cfs	Stage, ft	cfs	Stage, ft	cfs
1927	35.3	324,800	27.2	228,900	25.5	209,300	16.5	105,300
1928	26.0	215,100	23.4	185,000	21.5	163,400	12.3	56,400
1929	16.8	108,800	16.4	104,200	12.9	63,700	12.2	55,200
1933	23.6	187,300	19.0	134,200	17.0	111,100	12.2	55,200
1935	28.6	245,100	21.4	162,100	18.7	130,700	12.9	63,700
1937	17.0	111,100	15.2	90,300	14.9	86,800	12.8	62,500
1938	29.0	249,700	21.4	162,100	20.6	152,900	15.6	95,000

[1] White River—Ark. and Mo., H.R. Doc. 917, 76th Cong., 3d Sess., 1940.

of flood stages on the White and Lower Mississippi Rivers. The Lone Rock and Norfork Dam sites are near the mouths of the Buffalo and North Fork Rivers, respectively. The Bull Shoals Dam site is on the main stem of the White River upstream from the mouths of these two tributaries.

Flood-routing studies have been made to determine the effect of the various proposed reservoirs in the upper White River Basin upon different floods, with results as given in Table 12·3.

Revision—Table Rock and Bull Shoals Reservoirs. A revision of plans for these two reservoirs upon the basis of higher dams was made in 1944, as shown in Table 12·4.[1]

Table 12·4 Revised Data for Table Rock and Bull Shoals Reservoirs (1946)

Item	Table Rock	Bull Shoals
Drainage area, sq miles	4,020	6,036
Height of dam above stream bed, ft	255	263
Crest length of dam, ft	6,388	2,349
Elevation, top of flood-control pool	931	695
Elevation, top of power pool	920	654
Total storage, acre-ft	3,348,000	5,408,000
Flood-control storage, acre-ft	522,000	2,360,000
Power-storage, acre-ft	2,826,000	3,048,000
Flood-control pool, acres	50,700	71,200
Power pool, acres	44,400	45,400
Estimated cost	$37,000,000	$47,000,000

As will be noted from Tables 12·1 and 12·3, the present plans (1945) provide for a higher dam at both sites and corresponding increases in storage capacity and cost. It will be noted, however, that little change is made in flood-control storage but that power storage is increased for the two plants by a total of about 4 million acre-ft, or over three times as much. The foregoing estimates and procedure have been given in detail as illustrative of the modification of flood-control projects by further study and the use of possible multiple-purpose reservoirs.

Other Reservoirs in White River Basin. In addition to the Norfolk, Table Rock, and Bull Shoals projects, there are five other approved reservoirs, as shown in Table 12·5 (see[2] Fig. 12·3). This group of approved reservoirs will provide flood-control storage of about 7,382,000 acre-ft upon the White River and will control about 13,000 sq miles with a storage of nearly 11 in. depth, or about 47 per cent of the total drainage area of 28,000 sq miles.

[1] Letter from district engineer, Feb. 28, 1946.
[2] *Ibid.*

Table 12·5 White River Basin—Approved Reservoirs

Reservoirs	Drainage area, sq miles	Total storage capacity, acre-ft	Flood-control storage, acre-ft	Estimated cost (1946)
Clearwater	898	413,000	391,000	$ 8,500,000
Water Valley	1,152	1,563,000	1,537,000	7,600,000
Bell Foley	520	245,000	227,000	5,363,000
Lone Rock	1,331	687,000	657,000	11,422,000
Greers Ferry	1,147	981,000	956,000	7,179,000
Totals, 5 reservoirs	5,048	3,889,000	3,768,000	$ 40,064,000
Norfork, Table Rock, and Bull Shoals	7,842*	10,739,000	3,614,000	$111,500,000
Totals, 8 reservoirs	12,890	14,628,000	7,382,000	$151,564,000

* At Bull Shoals and Norfork.

The location of the eight approved reservoirs, including Norfork, which is completed, are shown in Fig. 12·3, which shows the White River drainage area and the location of all flood-control projects therein. The group of *other reservoirs* considered as shown in Fig. 12·3, includes 13 additional projects, mostly multiple-purpose power and flood control, which aggregate about 3.6 million acre-ft more of flood-control storage and about 7.4 million acre-ft of power storage, or 11 million acre-ft total storage. No estimates of reservoir cost are available for these sites. In Fig. 12·3 are also shown the location of levees—existing, proposed, and considered, for local flood protection—particularly upon the Black and Lower White Rivers.

Denison Reservoir and Red River, Tex.[1]

The Red River rises near the easterly boundary of New Mexico, flows generally eastward some 850 miles across the Texas Panhandle, and between Alabama and Texas to Fulton in southwestern Arkansas, thence south and southeast 463 miles, entering the Mississippi through the Old River at Red River Landing, 771 miles below Cairo. It is the last large tributary of the Mississippi, entering from the west (see Fig. 12·1).

It has a total drainage area of 91,430 sq miles. The Denison Reservoir Dam is upon the Red River 288 miles by river above Fulton, and 5 miles northeast of Denison, Tex. The drainage area at the dam site is 38,291 sq miles, or about 42 per cent of the total for the main river (see Fig. 12·4).

[1] Denison Reservoir, Tex., H.R. Doc. 541, 75th Cong., 3d Sess., 1938.

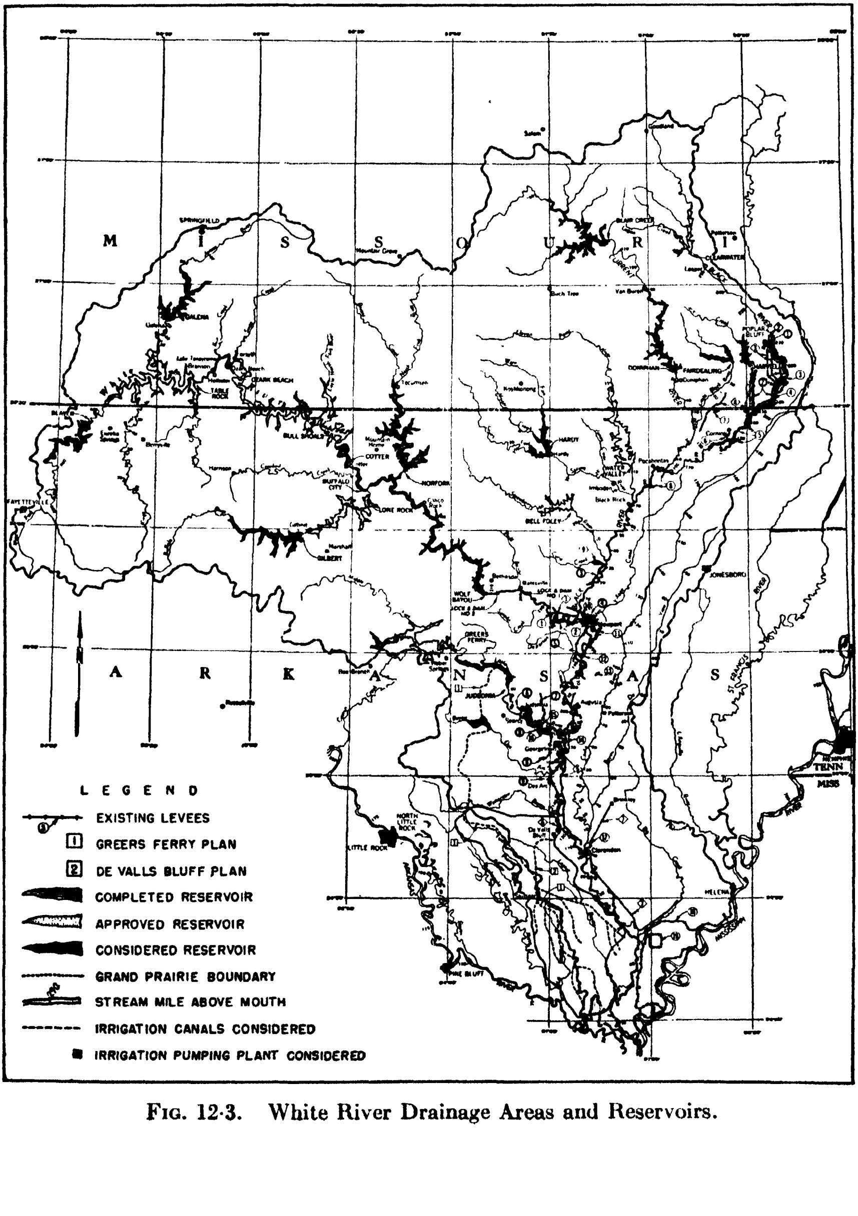


FIG. 12·3. White River Drainage Areas and Reservoirs.

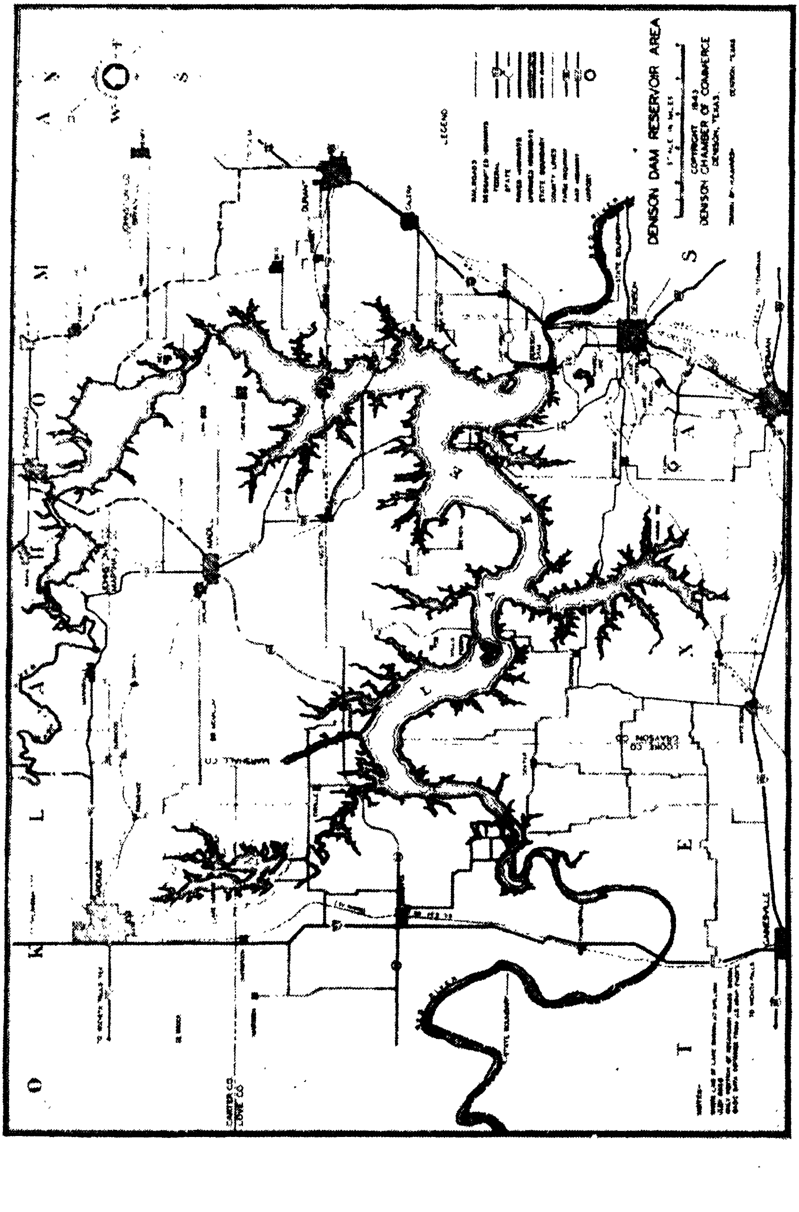


Fig. 12·4. Denison Reservoir.

The upper basin is semiarid, and the headwater streams flow intermittently in wide beds between low and unstable banks, with progressively decreasing slopes. In the central part, the watershed is less arid. The flood plain is wider, and the river slope decreases to 1 ft per mile. The low-water channel, although more stable than in the upper regions, is still poorly defined and subject to continual shifting with fluctuations in stream flow; and, during floods, the river creates numerous cutoffs. At Alexandria, La., 122 miles above the mouth, the river enters the backwater region of the Mississippi, and its channel becomes fairly stable. The mean annual precipitation increases from 20 in. near the headwaters to 37 in. at Denison and 50 in. near the mouth. The runoff at Denison during the 31 years of record has varied from 85 to 416,000 cfs, averaging 6,000 cfs, or only about 0.16 csm, reflecting its semiarid character.

The basin is predominantly rural in character, the areas affected by the proposed Denison Reservoir being devoted chiefly to the production of cotton, corn, hay, wheat, oats, potatoes, and dairy products. The more important natural resources of the region include oil, natural gas, coal, lignite, and lumber. The five leading industries are the refining of petroleum; lumbering and allied pursuits; the manufacture of cottonseed oil, cake, and meal; printing and publishing; and the processing of meats.

The more important cities on the river are Alexandria, with 23,000 inhabitants; Shreveport, La., 307 miles above the mouth, with a population of 76,600; Fulton, with 593 inhabitants; and Denison, with 13,800. The total population within 200 miles of the Denison Reservoir site is reported to exceed 5,000,000, the larger centers being the Dallas–Fort Worth industrial area 80 miles to the southward, with 400,000 inhabitants; Oklahoma City, 135 miles northwest, with 185,000 inhabitants; and Tulsa, 170 miles to the north, with a population of 141,000. Excellent rail and highway facilities are available, and the region is traversed by a network of gas transmission and distribution lines serving nearly every community of appreciable size.

Navigation. Prior to the advent of railways in the latter half of the nineteenth century, the Red River formed the principal artery of transportation into its basin. Commerce on the waterway is now limited to the section below Alexandria, only about 60 miles from the Mississippi. It consists chiefly of movements of forest products, petroleum, construction materials, and general merchandise between Mississippi River points and the Ouachita, which enters the Red River from the north through the Black River some 35 miles above the Mississippi. The initial project for the improvement of the Red River, authorized in 1828, provided for the removal of a vast accumulation of drift that obstructed the channel for a distance of 75 miles near Shreveport. The project was subsequently modified from time to time, the latest projects, adopted in 1892 and 1912, providing for bank clearing, snagging, dredging of shoals, construction of levees, and other channel improvements below Fulton; the removal of snags, logs, and drift; the clearing of timber from caving banks; a small amount of dredging; and the closure of certain by-channels from Fulton to the mouth of the Washita River a few miles above Denison. No work has been done above Alexandria since 1916. Expenditures to June 30, 1936, have totaled about $4,020,000. Channel depths of 5 to 6 ft are generally available below Fulton during the 6-month period January to June of each year; but, above Alexandria, the channel is narrow and tortuous, and currents are too swift for the economical movement of towed barges. The Supreme Court of the United States in 1922 held that the evidence in the case of a suit between Oklahoma and Texas established the fact that no part of the Red River forming the boundary between these states has the legal status of a navigable stream.

Investigations at the time of the so-called "308 Reports"[1] including study of a system of reservoirs for stream-flow regulations, with locks and dams and channel reclassification and regulating works, indicated that costs of improvement would be far in excess of probable benefits.

Floods. Records are available for only three floods on the Red River occurring in 1908, 1915, and 1935, although it is known that floods of considerable magnitude occurred in 1843, 1876, and 1892. The 1843 flood was the largest of these three, with an estimated peak discharge at Denison of roughly 600,000 cfs and a runoff of 4 million acre-ft (about 2.0 in.).

For the three floods of record and the anticipated "maximum flood," data are as follows:

Floods at Denison Dam Site

Flood	Rainfall, in.	Runoff		Peak discharge	
		In.	Acre-ft	Cfs	Csm
1908	5.03	1.30	2,712,000	470,000	12.3
1915	3.71	1.04	2,140,000	240,000	6.2
1935	4.10	0.78	1,588,000	138,000	3.6
"Probable maximum"	7.83	4.40	8,978,000	1,250,000	32.7

The study of probable maximum flood was based upon some 70 storms over and around the Red River Basin as well as examination of available flow records. The greatest storm would produce an average of 7.8 in. over the entire basin in about 3 days. Runoff is taken as 56 per cent of the rainfall,

[1] H.R. Doc. 378, 74th Cong.

or about double that for the 1908 flood, assuming a previous soaking of the soil. This results in a volume of runoff of nearly 9 million acre-ft, or 4.4 in. depth, or about three times that for 1908.

Flooded Areas. Bottom lands in the Red River Valley are subject to overflow practically throughout the length of the river. Above Denison Dam site, however, flooded areas are small and relatively unimportant. Below Denison, the flood plain is large in area, and flood losses of agricultural crops are high. In addition to flooding of lands in the main valley, some flooding occurs in the principal tributary valleys. This will be alleviated to some extent by the reservoir.

Between Denison and the Texas-Arkansas line, 153,000 acres of cleared land were inundated by the 1935 flood. All but 10 per cent of this, or 138,000 acres, would have been protected by the proposed Denison Reservoir. If the 1908 flood were to recur under present conditions, about 233,000 acres would be flooded in this reach, which could be reduced to about 15,000 acres with the reservoir in operation. If the estimated maximum probable flood were to occur now, about 263,000 acres would be flooded, of which 110,000 acres would be protected if the reservoir were operating.

Between the Texas-Arkansas line and Alexandria, there are numerous levee districts with levees in various stages of repair (or disrepair). Certain new levees have also been authorized below Shreveport. For the purpose of estimating benefits to be obtained from the Denison Reservoir, it was assumed that existing levees, with a few exceptions, were complete to their proposed grade and section and that authorized work was completed. On this basis, the proposed Denison Reservoir would protect in this reach 35,000 acres of cleared land from a 1935 flood, 179,000 acres from a 1908 flood, and 486,000 acres from the estimated maximum probable flood.

Flood Damages. These are chiefly confined to agricultural lands and crops. As an essential part of the evaluation of flood damages in the Red River Valley, a survey was made of the agricultural land between Denison, Tex., and Alexandria, La., to determine the average production of the different crops grown in the flooded areas. This was done in conjunction with the measurements of areas that are subject to flooding by floods of various magnitudes. As a result of this survey, it was determined that a fair average for cotton production in the valley above the Texas-Arkansas line was 200 lb per acre, and a conservative value for the lower valley was 250 lb per acre, where protection against minor floods exists. Cotton is by far the principal crop of the area, with corn, alfalfa, mixed hay, and vegetables furnishing the bulk of the remaining agricultural products. An estimate of the proportion and value of these crops combined with an estimate of the value of the cotton and cottonseed resulted in an estimated annual flood damage to crops above of $1,007,535. Preventing these crop losses would be the principal flood-control benefit to be obtained from the proposed reservoir.

Other damages would be prevented by reducing flood magnitudes especially of great floods, including loss of buildings, bridges, and livestock, reduced crops, land erosion, and silt deposition.

Yearly flood losses, as estimated, would be

Crop losses	$1,008,000
Other losses	609,000
Direct losses	$1,617,000
Intangibles	150,000
Total yearly flood losses	$1,767,000

Reservoir Storage and Project. Based upon detailed studies, it was determined that a total storage capacity of about 5.8 million acre-ft (or about 3 in. depth on the drainage area) of storage would be required for effective flood control, if built for the combined purposes of flood control and power development. This storage capacity in detail is as follows:

	Million Acre-ft
Silt and dead storage	1.2
Power storage	1.8
Flood storage	2.8
	5.8

Flood storage of about 0.8 million total acre-ft is also available in the surcharge of 5 ft above spillway level.

At El. 640, spillway crest level, the water area will be 146,000 acres (220 sq miles); at El. 617, normal pool level, this area will be 95,000 acres (140 sq miles) (see Fig. 12·4, also Frontispiece).

The project, as constructed, includes a large rolled-fill earth dam with about 18.5 million cu yd of earth embankment. It is 15,200 ft long, 40 ft wide on top, and rises 165 ft above the river bed, with the top of the dam at El. 670 (mean Gulf level). On the Oklahoma side is a dike extension 5,800 ft long, and on the Texas side a spillway section 2,000 ft long at El. 640 (see Fig. 12·4). The spillway has an estimated peak outflow capacity of 750,000 cfs with a surcharge of 22 ft or to El. 662. This corresponds to a peak flow of about 1,350,000 cfs routed through the reservoir.

The ultimate installation at the power plant will be five 35,000-kw generating units, or 175,000 kw under a normal head of about 102 ft. Two of these units will constitute the initial installation. One unit was installed and began operation in March, 1945, and the second unit is expected to be completed by the middle of 1948. The ultimate plant energy output is estimated at 274 million kwhr for the average year and 110 million kwhr for the minimum year.

In Fig. 12·5 is shown a view of the powerhouse, tailrace, and outlet channel, and stilling basin and overflow weir. A portion of the surge tank 57 ft in diameter and 135 ft high shows back of the powerhouse and the earth dam in the background.

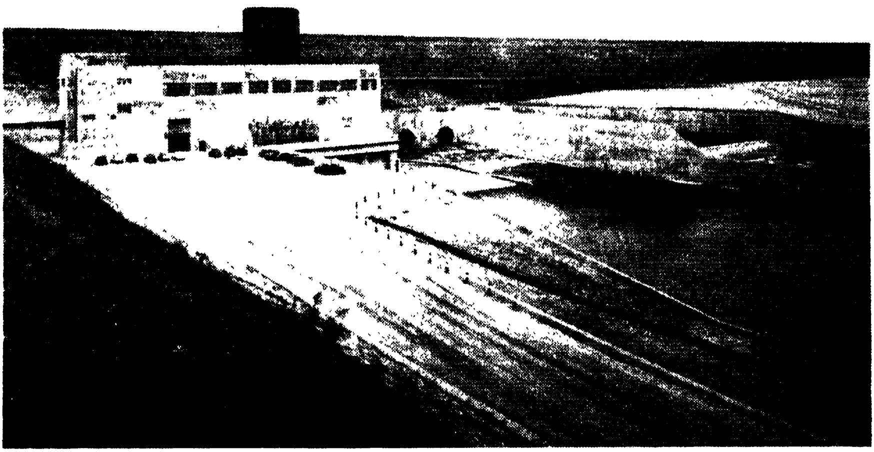


FIG. 12·5. Denison Dam. Powerhouse, tailrace, and outlet channel.

Sixteen head gates of the Broome type, each 9 by 19 ft with individual hoists, control the 8- to 20-ft-diameter steel-lined conduits (two gates for each conduit). Of these conduits, five will ultimately serve as penstocks for the power development. The remaining three conduits, together with a single stilling basin divided into two parts by an overflow weir, constitute the outlet works.

The cost of the 16 Broome head gates (1940) was $475,000, or about $174 per square foot, including operating equipment and cost of installation. The estimated cost of the completed initial project is about $56,500,000. The present (1946) estimate of the total ultimate installation cost is about $65,000,000.

Arkansas River

The Arkansas River has a total length of about 1,450 miles. It rises in the Rocky Mountains near Leadville, Colo., and flows in a general southeasterly direction through Colorado, Kansas, Oklahoma, and Arkansas to join the Mississippi River near Arkansas City, Ark. (437 miles below Cairo). It falls from about El. 11,500 ft at its source to El. 116 ft at its mouth. It is considered navigable from its mouth to the confluence of Grand (Neosho) River, mile 465, about 67 miles downstream from Tulsa, Okla. (see Fig. 12·1).

It has a total drainage area of 160,500 sq miles and 74,730 sq miles at Tulsa. Other principal tributaries besides the Grand (Neosho) are the Verdigris running approximately parallel to and westerly from the former and the Cimarron and Canadian rivers entering from the west near Tulsa and below the Grand (Neosho), respectively. See Fig. 12·1, which shows the Arkansas, White, and Red Rivers as well as the lower Mississippi with flood-control reservoirs as planned in 1937[1] for these rivers.

The flood-control project at Pueblo, Colo., in the extreme upper river, constructed in 1924, is described on page 238.

Three out of four authorized flood-control reservoir projects in the White River Basin above Tulsa are now (1946) partly completed. These include the following, the first three now constructed:

White River Basin above Tulsa—Reservoir Projects

Reservoir	River	Location	Storage capacity, acre-ft	Cost total
John Martin (Caddos)	Arkansas	Southwest Colorado	270,000	$14,800,000
Great Salt Plains . .	Salt Fork	North Central Oklahoma	317,000	4,485,000
Fort Supply.	Wolf Creek	Woodward, Okla.	118,000	7,710,000
Hulah	Caney	Huld, Okla.	295,000	8,840,000
Totals.			1,000,000	$35,835,000

Reservoirs in the Arkansas River Basin in the Tulsa district, some of which are partly constructed, are listed in Table 12·6. Three of the larger projects in Table 12·6 will now be discussed.

Canton Reservoir. Located upon North Canadian River about 373 miles above its mouth and near Canton, Okla., this reservoir will have an

[1] Comprehensive Flood Control Plan for Ohio and Lower Mississippi Rivers, Com. Doc. 1, 75th Cong., 1st Sess.

Table 12·6 Flood-control Reservoirs in Tulsa, Okla., District

Reservoir	River	Distance above mouth, miles	Reservoir capacity, acre-ft	Estimated cost		
				Construction	Lands, damages, and relocations	Total
Canton	North Canadian	393	390,000	$13,220,000	$ 650,000	$13,870,000
Mannford	Cimarron	20	600,000	6,100,000	1,800,000	7,900,000
Toronto	Verdigris	271	195,000	4,815,000	3,287,000	8,102,000
Fall River	Fall River	54	263,000	8,335,000	1,130,000	9,465,000
Oologah	Verdigris	87	815,000	4,745,000	3,600,000	8,345,000
Tenkiller Ferry	Illinois	13	650,000	8,090,000	1,700,000	9,790,000
Wister	Poteau	61	430,000	4,900,000	1,690,000	6,630,000
Neodesha	Verdigris	223	90,000	3,785,000	1,372,000	5,157,000
Elk City (Table Mound)	Elk	9	250,000	6,836,000	2,479,000	9,315,000
Markham Ferry	Grand (Neosho)	46	452,000	12,153,000	3,257,000	15,410,000
Fort Gibson	Grand (Neosho)	7.7	1,287,000	16,250,000	5,185,000	21,435,000

earth dam 68 ft high with 240,000 acre-ft of flood-control storage and 150,000 acre-ft of conservation and siltation storage. It will have a concrete spillway with 16 by 40 to 25 ft gates. The cost will be $13,870,000, or $35 per acre-foot.

Markham Ferry and Fort Gibson Reservoirs. These reservoirs are upon the Grand (Neosho) River.[1] The Neosho River, known as the Grand River in its lower reaches, rises in east-central Kansas, flows southeast 297 miles to the Oklahoma line, thence south across Oklahoma 163 miles, joining the Arkansas River 465 miles above its mouth. Its drainage area is 12,660 sq miles.

The basin is devoted largely to farming, stock raising, and dairying; and in the Ozark sections fruits are important. This section has the world's greatest production of lead and zinc. Industrial and manufacturing activities are also important in the basin.

Markham Ferry Reservoir (Mile 46). This is a combined flood-control and power reservoir. The dam is of concrete, with earth embankment, 90 ft high and 10,600 ft long with roadway over the top, providing 239,000 acre-ft of flood-control storage and 213,000 acre-ft of power storage. The capacity of the power plant will be 40,000 kw. The output will average 160 million kwhr yearly. The estimated cost wi l be $15,410,000, of which $3,300,000 is for lands and damages. This project is in the preliminary stage of construction.

Fort Gibson Reservoir (Mile 8). This is a combined flood-control and power project. The dam is of concrete, 110 ft high topped by a roadway, and 2,900 ft long, providing 922,000 acre-ft of flood-control storage and 365,000 acre-ft of power storage. The capacity of the power plant is 45,000 kw. The cost will be $21,435,000, of which $5,185,000 is for lands and damages. The project is in the preliminary stage of construction.

Pensacola Reservoir. Not shown in Table 12·6, this reservoir at mile 77 is for combined flood control and power and is a Federal Works Project. The dam, completed in 1940, is of concrete and provides 960,000 acre-ft of flood-control storage and 1,140,000 acre-ft of power storage. The capacity of the power plant is 63,000 kw. Power production in 1944 was 357 million kwhr; in 1943, 313 million kwhr. The firm power of the plant is 225 million kwhr yearly. The cost will be about $25,000,000, of which $3,000,000 is for lands and damages.

Little Rock, Ark., District. In this district on the Arkansas River, the Blue Mountain and Nimrod Reservoirs are well toward completion (see Fig. 12·1).

[1] H.R. Doc. 107, 76th Cong., 1st Sess., 1939; also *Report of Chief of Engineers*, 1944, p. 909.

Blue Mountain Reservoir. This is on the Petit Jean River about 75 miles above its confluence with the Arkansas River, near Pavis, Ark. An earth-fill dam 115 ft high and 2,800 ft long will afford 233,000 acre-ft of flood-control storage and 25,000 acre-ft for a permanent recreational and wild-life refuge pool. The cost will be $3,700,000, of which $580,000 is for lands and damages.

Nimrod Reservoir. This is on the Fourche La Fave River, 60 miles above its confluence with the Arkansas River near Danville, Ark. A 97-ft concrete dam will provide 305,000 acre-ft of flood-control storage and 30,000 acre-ft for a recreational and wild-life pool. The cost will be $3,500,000, of which $900,000 is for lands and damages.

Summary of Storage Capacity—Arkansas River Basin

No. of reservoirs	Subdivision	Total flood-control storage capacity, acre-ft	Total cost	Cost per acre-ft
5	Upper Arkansas	1,390,000	$ 49,700,000	
3	Grand (Neosho)	2,121,000	60,476,000	
9	Tulsa district except Grand (Neosho)	3,683,000	76,574,000	
2	Little Rock district	538,000	7,200,000	
19	Total	7,732,000	$193,950,000	$25

Total flood-control storage for the Arkansas River Basin is thus about 7,700,000 acre-ft, which is about 0.9 in. upon the total drainage area of 160,500 sq miles.

Grand (Neosho) River Flood Control and Power.[1] This river, in addition to flooding extensive areas in its own basin, is one of the principal flood-producing tributaries of the Arkansas River, and control of its floods would be of benefit on both the Arkansas and Mississippi rivers. Below the mouth of Grand River, the flood plain of the Arkansas River is about 1,044,000 acres (1,630 sq miles), of which about one-third is in the back-water of the Mississippi. Average yearly flood losses in the area below Grand (Neosho) River are estimated at about $600,000, of which $80,000 is in the Mississippi backwater.

The most suitable plan appears to be a combined development for flood control and power by the construction of Pensacola, Markham Ferry, and

[1] Pensacola, Markham Ferry, and Fort Gibson Reservoirs on Grand (Neosho) River, Okla. H.R. Doc. 107, 76th Cong., 1st Sess., 1939.

Fort Gibson reservoirs, allocating top storage in each reservoir to flood control. Storage and power are given in the following table:

Reservoir	Storage capacity			Installed capacity, kw	Firm capacity, kw	Annual energy output, million kwhr
	Allocated to power development, acre-ft	Allocated to flood control, acre-ft	Gross, acre-ft			
Pensacola	1,140,000	960,000	2,100,000	63,000	58,000	265
Markham Ferry	213,000	239,000	452,000	40,000	40,000	160
Fort Gibson	365,000	486,000	851,000	40,000	40,000	170
Total	1,718,000	1,685,000	3,403,000	143,000	138,000	595

The total cost of the three reservoirs is estimated at $61,850,000 (revised to 1942), with $17,300,000 charged to flood control and $44,550,000 to power. Annual charges at 4½ per cent are $2,780,000, and annual benefits are estimated at $3,450,000.

It is estimated[1] that, in the flood of 1927, with the three reservoirs in use, the peak flow of the Grand River at Fort Gibson would have been reduced from about 175,000 to about 50,000 cfs; in the Arkansas River below the Grand River, the peak flow of 378,000 cfs would have been reduced to 287,000 cfs.

Ouachita River[2]

The Ouachita River rises in western Arkansas, flows east 158 miles to near Malvern, Ark., and turns generally southeast to join the Red River 35 miles above its mouth. It is 605 miles long and drains an area of 24,300 sq miles. The basin above Malvern is rugged and mountainous and generally timberland except in the valleys. Below Malvern, the river enters an area of low rolling hills extending to within 286 miles of the river mouth and bordered by wide bottom lands only partly cleared. Below Moro Bay, the Ouachita enters the alluvial valley of the Mississippi River, with numerous swamps, lakes, and bayous, with about 22 per cent cleared and 78 per cent cutover timberland. The lower 57 miles of the river's course is known as the Black River.

The Ouachita Basin had a population of about 800,000 in 1930. The

[1] *Ibid.*, ¶ 86–87.

[2] H.R. Doc. 69, 76th Cong., 1st Sess., 1938; also H.R. Doc. 647, 78th Cong., 2d Sess., 1944.

principal agricultural products are cotton and corn. There are a number of producing oil fields in southern Arkansas and northern Louisiana, and some natural gas.

Two hydroelectric developments of importance have been constructed, the *Remmel plant* of 9,000 kw capacity a little above Malvern and, just above it, the *Carpenter plant* of 56,000 kw capacity, both built under a license of the Federal Power Commission issued in 1923. The license provides for the future construction of a third plant and reservoir at the Blakely Mountain site at the head of the Carpenter pool.

The Ouachita Basin is well provided with rail and highway facilities, and pipe lines extend from oil and gas fields to market centers. An authorized navigation project provides a channel 6.5 ft deep to Camden, Ark., 351 miles from the mouth, by six locks and dams and by dredging and by removal of channel obstructions to Arkadelphia 417 miles above the river mouth. Expenditures on this project to June 30, 1942, were $4,575,000 for new work, $3,162,000 for maintenance, and $2,941,000 for operation and care.

Floods. A flood-control project authorized by the Flood Control Act of June 28, 1938, provides for Federal participation in the interest of flood control and navigation to an amount not exceeding $2,000,000 in the cost of construction by private interests of a multiple-purpose reservoir at the Blakely Mountain site in accordance with plans and estimates duly approved by the Secretary of War and the Chief of Engineers, subject to the provision that the sum of money expended in this participation shall not exceed the estimated value of the flood control to be achieved or exceed a just and reasonable proportion of the total cost of the multiple-use reservoir as allocated according to the proportionate storage capacity reserved or utilized for flood control. The authorization further provides that the reservoir shall be operated and maintained at the expense of the private parties who construct the project, in accordance with regulations, with respect to navigation and flood control, approved by the Secretary of War and the Chief of Engineers. This development has not been undertaken.

The average annual precipitation on the Blakeley Mountain Reservoir Basin is 52 in., with maximum and minimum recorded annual values of 75 and 33 in., respectively. The heaviest rainfall generally occurs during April and May, although flood-producing storms may occur at any season of the year. The mean monthly flow at the Blakeley Mountain site was 1,630 sec-ft during the 50-year period from 1893 to 1942, with a maximum annual runoff of 2,705,000 acre-ft and a minimum of 388,000 acre-ft. Six damaging floods occurred in the Ouachita River Basin during the 14-year period 1927–1940, inclusive, of which the dual flood of April, 1927, was the most severe and produced the maximum recorded stage of 29.2 ft at Arkadelphia and peak stages of 42.4 and 41.8 ft at Hot Springs and Camden, respectively. About 573,400 acres of bottom land between Remmel Dam, river mile 452, and lock and dam No. 3, river mile 134, were flooded, with resulting total damages for the six floods of $1,260,000 principally to agricultural interests and transportation facilities. Approximately 90,000 acres of the flooded area, including 27,000 acres of cultivated land, are located along the Ouachita River between Remmel Dam and the mouth of the Little Missouri River, river mile 380, and would be benefited by the Blakeley Mountain Reservoir. The estimated average annual direct and indirect flood damages on this area are $115,000 and $29,000, respectively, a total of $144,000, comprising $56,000 from damage to crops, $74,000 from other agricultural damages, and $14,000 from nonagricultural damages.

Blakeley Mountain Project.[1] Various plans for development of the Blakeley Mountain site have been considered for flood control only, for power production only, and for flood control and power production combined. The construction cost of a reservoir with a storage capacity of 560,000 acre-ft for flood control only would be $6,027,000. The annual cost would be $287,600 and probable annual benefits $138,500, which indicates that development for flood control alone is not warranted. The most desirable plan for development consists of a dual-purpose project for flood control and power production by means of an earth-rock-fill dam located at river mile 490, 10 miles northwesterly from Hot Springs, Ark., and at the head of Lake Hamilton, the pool formed by Carpenter Dam.

Dam. As proposed, the dam would be 217 ft high and 1,114 ft long, with a detached uncontrolled spillway located in a saddle on the right side of the river about 1½ miles from the dam. It would have a crest elevation of 602 and would create a reservoir of 1,915,000 acre-ft capacity at the spillway crest level, El. 572. The 525,000 acre-ft of storage in the 16 ft between the spillway crest and El. 556 would be reserved for flood control, while the 905,000 acre-ft between El. 556 and 513 would be utilized for power production, thereby leaving 485,000 acre-ft of dead storage below El. 513. Surcharge storage between the spillway crest and the maximum allowable water level, El. 595.5, would be 1,020,000 acre-ft.

Power Plant. The power plant, designed for an ultimate installed capacity of 57,000 kw, would be located a short distance below the dam on the left side of the river. A 24-ft-diameter, concrete-and-steel-lined tunnel through the left abutment, with a suitable intake gate arrangement at the upper end, would serve as the combined intake works for power and release of water for flood control. The main tunnel, with a surge tank at the downstream end, would supply water to the turbines; and a left-hand wye branch near the downstream end would constitute a by-pass for release of water

[1] H.R. Doc. 647, 78th Cong., 2d Sess., 1941.

through four 96-in. diffusion valves for flood-control purposes. The tunnel would also serve as a diversion works during construction by means of a short, temporary, low-level connecting tunnel at the upstream end, which would be plugged with concrete after it had served its purpose.

Power Installation. The proposed initial power installation is 38,000 kw and, under the average net head of 149 ft, would produce on the average 83,000,000 kwhr of primary and 30,200,000 kwhr of usable secondary energy annually at the site. The proposed project would provide a regulated flow of 1,060 sec-ft, which would increase the annual output of the existing Carpenter and Remmel plants by 4,000,000 kwhr.

With 525,000 acre-ft of storage capacity reserved for flood-control purposes, a flood such as that which occurred in April, 1927, could be reduced to bank-full stage at Malvern, Ark. Peak-stage reductions would amount to 10.7 ft at Malvern, 4.3 ft at Arkadelphia, and 1.8 ft at Camden. Annual flood damages prevented, on the 95,000 acres of bottom land between Remmel Dam and the mouth of the Little Missouri River, which is subject to flooding, are estimated to be $60,600 direct and $21,600 indirect, or a total of $82,200, which will be increased to an average of $94,500 during the life of the project by the normal future development of the region. In addition to the flood damages prevented, estimated annual benefits to the amount of $35,400 will accrue from the increased value of land due to flood control, which gives total average annual tangible benefits from flood control of $129,900. Intangible benefits arising from regulation of stream flow would be realized from improved water supplies for existing industrial plants; from pollution abatement because of dilution of industrial waste from paper mills and oil fields, and of the effluent from sewage-disposal plants; and from improved recreational facilities. The estimated construction cost of the dual-purpose project with an initial installation of 38,000 kw of generating capacity in two units and provision for the installation of one additional generating unit is $11,080,000, including $450,000 for transmission connections to existing facilities. The estimated total annual cost is $571,600.

The power used in this region is supplied principally by steam-electric plants, using natural gas for fuel, which have an estimated capacity value of $13 per kilowatt of prime capacity and an energy value of 1.2 mills per kilowatt-hour of output. Energy generated by the interconnected systems comprising the Arkansas Power & Light Co., the Louisiana Power & Light Co., and the Mississippi Power & Light Co., serving this region, ranged from 189.5 million kwhr in 1926 to 937.2 in 1939, and the peak demand ranged from 47,400 to 193,000 kw during the same period. Assuming that the power demand will increase at rates prevailing prior to the war, it is estimated that the annual requirements for the years 1948, 1956, and 1968 will be 1,500, 1,850, and 2,300 million kwhr, respectively, with corresponding peak demands of 300,000, 375,000, and 460,000 kw. The existing hydroelectric plants in the basin are now used only for peak-load purposes, and the output from the authorized Norfork and Bull Shoals projects in the neighboring White River Basin will also be available for peak-load purposes. Even under these conditions, the output from the proposed Blakely Mountain plant could be utilized for peak-load purposes in the early future.

The proposed project would have a total energy value of $135,000 at a unit value of 1.2 mills per kilowatt-hour of output and a capacity value of $494,000 at a unit value of $13 for the 38,000 kw of prime capacity. The increase in prime peak capacity and average annual output of the existing Carpenter and Remmel plants would have a total net power value of $94,600 after the annual value of the losses due to the original filling of the Blakeley Mountain pool was deducted. The total annual power value of the Blakeley Mountain project is therefore $721,400, which, when added to the flood-control benefits of $129,900, gives total annual benefits of $854,300 and an over-all annual benefits-to-cost ratio of 1.49 to 1.0. The annual cost chargeable to flood control is $106,000 and to power, $165,000, which gives a unit cost of production of 4.1 mills for the 113,200,000 kwhr of total average annual output.

The Blakeley Mountain project was recommended in 1943 and authorized by the Flood Control Act of 1944 at an estimated cost of $11,080,000 for construction and $61,000 yearly for maintenance and operation.

Trend toward Multiple-purpose Reservoirs. It will be noted that flood control upon the group of rivers discussed in this chapter is being effected by multiple-purpose reservoirs where power development is a dominating feature and storage capacity to a high degree is contemplated in some of the more recent projects on the White River.

As noted in Table 12·5, page 262, the three reservoirs, Norfork, Table Rock, and Bull Shoals, will have a total storage capacity of 10.7 million acre-ft of which 7.4 million acre-ft is power storage on the drainage area of 7,842 sq miles above Bull Shoals and Norfork sites, or a total storage of 25.6 in. including 17.6 in. for power and 8.0 in. for flood control.

The total cost of the group of eight reservoirs upon the White River in Table 12·5 is about $112,000,000. The three reservoirs, Norfork, Table Rock, and Bull Shoals, however, will provide yearly power value totaling about $3,800,000 for about 770 million kwhr (or about 0.5 cent per kilowatt-hour), which at 4.5 per cent would correspond to a capital amount of $85,000,000. The net project cost, allowing for power value, would therefore be $152,000,000 − $85,000,000 = $67,000,000. For the flood-control storage of 7.38 million acre-ft, this would be about $9 per acre-foot for storage, which is a low cost.

The cost for flood control upon the White River (28,000 sq miles) would be \$67,000,000/28,000 = \$2,400 per square mile, which is relatively low (see Table 9·3). It should also be kept in mind that these White River reservoirs are of help in relieving Lower Mississippi River floods.

Without power development, the eight-reservoir system upon the White River would cost about \$110,000,000, or \$15 per acre-foot of storage. The cost of flood control would be about \$4,000 per square mile. The relative economy of the multiple-purpose reservoir system for flood control is thus obvious. Similar advantages in multiple-purpose development for flood control will also be found to obtain for Denison Reservoir upon the Red River and for the group of three reservoirs upon the Grand (Neosho) River. There appears to be a distinct trend in the Southwest toward the use of multiple-purpose reservoirs for flood control.

CHAPTER 13

TENNESSEE VALLEY AUTHORITY

The Tennessee Valley Authority was created in 1933 and stands as the first attempt in this country to control and utilize a large river for the benefit of the surrounding region. Its purpose, as stated in the original Act of 1933, was "Maintaining and operating the property now owned by the United States in the vicinity of Muscle Shoals in Alabama in the interests of the national defense and for agricultural and industrial developments and to improve navigation in the Tennessee River and to control the destructive floodwaters in the Tennessee River and Mississippi River Basin."

It was created as a corporation under a board of three directors appointed by the President, the President designating the chairman. All other officers, agents, and employees are designated and selected by the board. Power development was authorized in amendments during 1935, and authority was given to acquire real estate for powerhouses, transmission lines, and other structures.

During the First World War, the large Wilson power dam had been built across the Tennessee at Muscle Shoals, together with two nitrate factories that had been subsequently of little or no use. The Tennessee Valley Act turned the Wilson Dam over to the TVA and authorized this agency to sell the power generated there and at its other dams as well as to create an interconnection and control of the power sold. The board authorized the modernizing of its plant at Muscle Shoals to improve its production of fertilizer and engaged in the study of the efficiency of various fertilizers and their manufacture.

The purposes of the act were further broadened to bring about in the Tennessee drainage basin the maximum amount of flood control and navigation; the maximum amount of generation of electric power; the proper use of marginal land and reforestation suitable for this and the economic well-being of the people living in the river basin.

Tennessee River Basin. The Tennessee River is formed by the junction of the Holston and French Broad Rivers just above Knoxville. It discharges into the Ohio River near Paducah, Ky., a distance of about 650 miles, with a fall of about 510 ft in this distance.

The drainage area of the Tennessee River Basin at its entrance to the Ohio is 40,600 sq miles (an area about equal to that of the state of Ohio) and includes parts of seven states. Land elevations range from more than 6,500 ft at the crests of the Great Smoky Mountains and the Blue Ridge in North Carolina and the eastern part of the basin to a little over 300 ft along the river's lower course. The eastern mountainous area is a region of forests, steep-sloping pasture lands, and small farms in contrast to the relatively flat cotton-plantation country in northern Alabama and the low rolling hills of western Tennessee and Kentucky.

Large areas of crystalline rocks occur in the headwaters, but the chief characteristic rock is limestone of relatively high solubility. Coal beds occur in the plateau regions of Tennessee and northern Alabama. About one-half of the basin is forested. The soil is fine-textured, absorbs water slowly, has rapid runoff, and erodes freely unless protected by adequate ground cover. Much soil erosion and gullying therefore occur on the cleared and farm lands.

The mean annual precipitation is about 52 in. for the basin, varying for individual years from 40 to 60 in. Portions of the eastern mountainous areas have high rainfalls with yearly means reaching 80 in. Gulf storms entering from the west and hurricanes coming north from the Atlantic Coast give intense precipitation at times.

Runoff varies from about 19 to 32 in., as shown in Table 13·1, and averages about 22 in. for the entire basin.

Table 13·1 Tennessee River Basin—Drainage Areas and Yearly Mean Runoff

River	Point	Drainage area, sq miles	Yearly mean runoff, in.
Tennessee	Knoxville	8,934	19.7
	Loudoun	12,220	20.4
	Chattanooga	21,400	23.6
	Florence	30,810	22.4
	Johnsonville	38,520	22.0
Holston	Strawberry Plains	3,626	15.1
Wautaga	Elizabethton	692	21.3
French Broad	Dandridge	4,446	19.7
Clinch	Norris Dam	2,913	17.5
Powell	Arthur	685	22.6
Little Tennessee	McGhee	2,443	32.2
Hiwassee	Murphy	404	31.2
	Reliance	1,223	26.1

TVA Multiple-purpose Projects (see Fig. 13·1). The first storage and power project undertaken by the TVA was at the Norris Dam on Clinch River, for which immediate construction was specified in the Act of 1933. This was put in operation in 1936. Construction on the main river

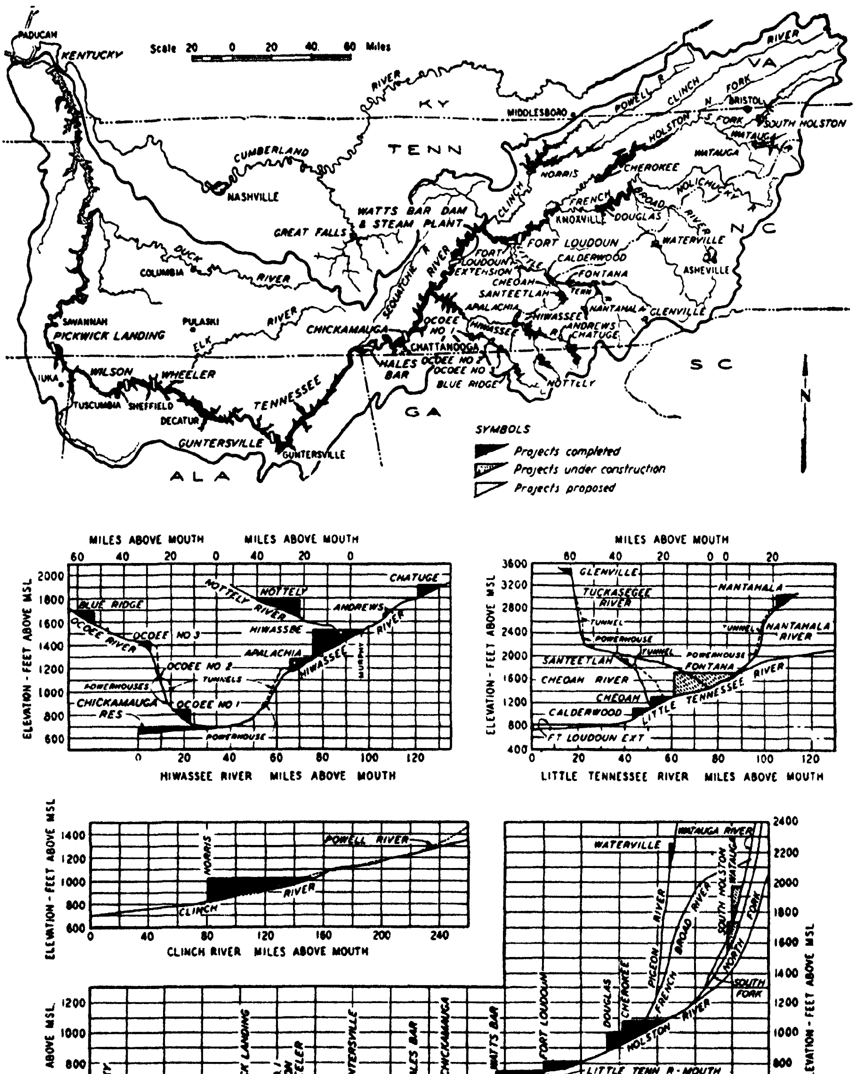


FIG. 13·1. TVA multiple-purpose projects.

of Wheeler Dam was ordered in 1933 and completed in 1936, followed by the Pickwick Landing and Guntersville main-river projects in 1938 and 1939, respectively.

The coming of the Second World War greatly stimulated project construction, and the present comprehensive development was framed along the following lines (see Table 13·2).

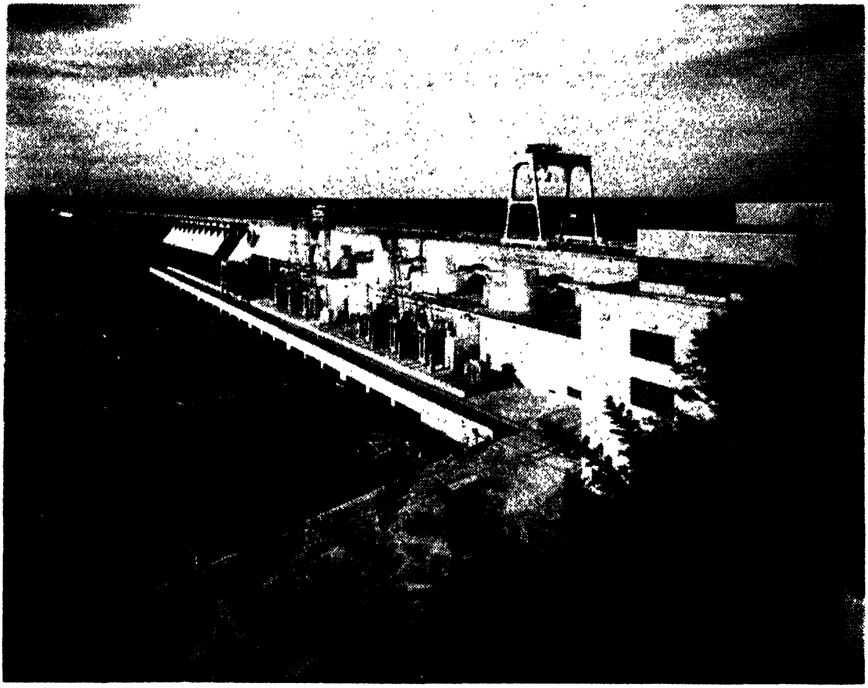


FIG. 13·2. Wheeler Dam, TVA.

1. Nine main-river projects, including a low-head major storage development—the Kentucky project near the mouth of the Tennessee, for controlling discharges into the Ohio and Lower Mississippi. The other eight projects have limited storage operated to reduce flood crests, and they provide the 9-ft navigation channel, with locks, and hydroelectric power stations to utilize the 500-ft drop in the river between Knoxville and Paducah (see profile in Fig. 13·1 and also Fig. 13·2).

These nine projects will utilize a total head of 458 ft, with about 853,000 kw (ultimately 1,600,000 kw) of hydroelectric capacity and a total usable storage capacity of about 7 million acre-ft.

2. Five main tributary projects with major storage capacity to control the five principal headwater tributaries, *viz.*, the Clinch, Holston, French

Broad, Little Tennessee, and Hiwassee Rivers. These projects afford substantial power generation and are the basic means of regulation for the multiple-purpose operation of the system. (See Fig. 13·3).

These five projects will utilize a total head of 885 ft, with about 218,000 kw (ultimately 658,000 kw) of hydroelectric capacity and a total usable storage capacity of about 6 million acre-ft.

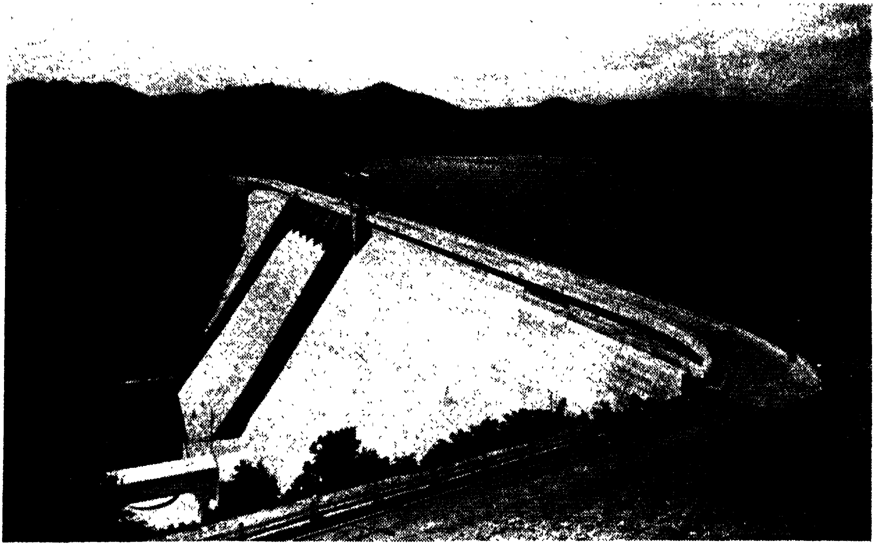


FIG. 13·3. Hiwassee Dam and Reservoir, TVA.

3. Ten secondary tributary projects with holdover storage for the main system and flood protection for the upper valley regions. The steep river slopes provide for economical power generation either directly at the storage dam or by the use of a tunnel penstock.

These 10 projects will utilize a total head of 1,687 ft, with about 87,000 kw (ultimately 324,000 kw) of hydroelectric capacity and a total usable storage capacity of about 2 million acre-ft.

4. The entire hydroelectric power and storage included in the three foregoing divisions will have

3,030 ft of developed head.
1,159,000 kw capacity—present.
2,588,000 kw capacity—ultimately.
15,000,000 acre-ft of storage capacity.

5. Ten steam power plants, operating upon base load during dry years and to carry peaks at times of flood when the main river plants operate

Table 13·2 TVA—Power and Storage Projects

	Head	Capacity, kw	Ultimate capacity, kw	Storage capacity, acre-ft
I. Main River				
1. Kentucky	48	......	160,000	4,570,000
2. Pickwick	43	144,000	216,000	418,400
3. Wilson	94	284,800	436,000	43,000
4. Wheeler	48	129,600	259,200	429,000
5. Guntersville	36	72,900	97,200	282,000
6. Hales Bar	36	51,100	51,100	
7. Chickamauga	36	81,000	108,000	377,000
8. Watts Bar	52	90,000	150,000	433,000
9. Fort Loudoun	65	...	128,000	138,500
Subtotal	458	853,400	1,605,500	6,690,900
II. Main Tributaries				
1. Norris	165	100,800	100,800	2,020,000
2. Cherokee	100	60,000	120,000	1,413,000
3. Douglas	100	......	120,000	1,260,000
4. Fontana	330	......	202,500	1,160,000
5. Hiwassee	190	57,600	115,200	365,000
Subtotal	885	218,400	658,500	6,218,000
III. Secondary Tributaries				
1. South Holston	190	....	75,000	644,000
2. Watauga	225	....	60,000	627,000
3. Apalachia	360	....	75,000	36,700
4. Chatuge	.....	....	....	226,500
5. Nottely	.....	....	....	189,200
6. Ocoee No. 1	110	18,000	18,000	25,800
7. Ocoee No. 2	250	19,900	19,900	
8. Ocoee No. 3	280	....	27,000	9,090
9. Blue Ridge	147	20,000	20,000	183,000
10. Great Falls	125	29,370	29,370	49,600
Subtotal	1,687	87,270	324,270	1,990,890
Total of 24	3,030	1,159,070	2,588,270	14,899,790

under reduced head, are available with a present plant capacity of 346,000 kw and ultimate capacity of 506,000 kw. The fourth 60,000-kw unit at the Watts Bar steam plant was placed in operation in 1945, bringing the plant to its full four-unit capacity of 240,000 kw. Including these steam plants, the TVA power system has a present (1945) capacity of about 1,500,000 kw and an ultimate capacity of about 3 million kw. (See Table 13·3.)

Table 13·3 Power Output—TVA

Year	Installed capacity, kw	Power generated			Industrial power sales, million kwhr
		Million kwhr	Kw avg.	Capacity factor	
1946	2,538,900[1]	12,314	1,410,000	0.55	9,059
1945	2,513,102[1]	11,935	1,370,000	0.55	10,314
1944	1,893,000	10,117	1,150,000	0.60	9,110
1943	1,640,000	9,056	1,130,000	0.69	6,313
1942	1,375,000	6,025	695,000	0.50	4,094
1941	1,064,000	5,556	640,000	0.60	3,234
1940	967,000	4,043	465,000	0.48	2,157
1939	420,000	1,733	200,000	0.48	

[1]Includes Alcoa hydro plants, operated under TVA direction.

6. Six hydroelectric plants of the Aluminum Co. of America are located on the Little Tennessee River with the following characteristics: total head developed, 3,883 ft (or some 800 ft more than that of the TVA system itself); capacity present and ultimate, 415,000 kw; and usable storage, 355,000 acre-ft.

These include three high-head hydroelectric developments, *viz.*, Glenville, Nantahala, and Santeetlah, with heads of 1,150, 925, and 660 ft, respectively.

Including these private projects, the total power capacity in the TVA system is about 2,000,000 kw at present and 3,400,000 kw ultimately, and a total usable storage capacity of over 15 million acre-ft.

As of June 30, 1945, the useful controlled storage in the system including about 334,000 acre-ft of the Alcoa projects (operation controlled by the TVA), was about 13.5 million acre-ft. Of this, Kentucky Reservoir near the mouth of the river, completed in 1946, contains over 4 million acre-ft of storage, located effectively for flood control on the lower Ohio and Mississippi rivers.

Power Output and Sales. In Table 13·3 are given data of installed capacity, power generated, and industrial power sales for the 7-year period 1939-1946, inclusive. Prior to 1939, power generated ranged from 122 million kwhr yearly in 1935 to 700 million in 1938. After 1942, generation increased owing to war requirements for power; and, as will be noted, industrial sales of power constitute about two-thirds of the total power generated.

The production of war materials in 1943 included chiefly aluminum—2,733 million kwhr or 30 per cent of the total power generated; copper and heavy chemicals—1,242 million kwhr or 14 per cent; ammonia, ammonium nitrate, and phosphorus for munitions; and calcium carbide for synthetic rubber.

Of the 9,056 million kwhr generated in 1943, 7,844 million were from hydroelectric plants and 1,212 million from steam plants, or 87 per cent hydroelectric. The average capacity factor for 1941–1944 was about 0.60. The total generation of power at all TVA plants, and at those under TVA direction, in 1945 was about 12 billion kwhr, with a capacity of about 2,500,000 kw.

The total power sales during 1943 were 8,336 million kwhr, distributed as shown in Table 13·4, with 38 per cent from municipal and cooperative distributions and 47 per cent from large industrial customers. The price per kilowatt-hour averaged 3.78 mills for 1943 and 4.21 mills for 1942, and 3.78 and 3.85 mills per kilowatt-hour for 1945 and 1946 respectively.

Table 13·4 Sale of Power, 1942-1943

	1943 thousand kwhr	Mills per kwhr	1942 thousand kwhr	Mills per kwhr
Municipalities and cooperatives	2,999,257	4.25	2,609,620	4.27
Industrial	3,935,122	3.49	2,409,931	4.25
Electric utilities	586,125	3.12	679,489	4.03
Federal agencies	269,437	4.20	2,934	5.56
Rural (retail)	9,101	19.05	8,500	18.65
Total outside sales	7,799,042	3.80	5,710,474	4.25
Interdepartmental sales	537,024	3.55	272,895	3.32
Total sales	8,336,066	3.78	5,983,369	4.21

Local Distribution of Power.[1] At the close of the fiscal year of 1944, 84 municipal and 45 cooperative systems were purchasing power wholesale

[1] *Annual Report TVA*, 1944, pp. 49–51.

Table 13·5 Project Costs of TVA

Date of first use or availability for power	Project	Cost	Totals
	Hydroelectric multiple purpose		
1933	Wilson	$41,115,753	
1936	Norris	30,975,740	
1936	Wheeler	35,722,380	
1938	Pickwick	35,644,924	
1939	Guntersville	31,595,725	
1939	Hales Bar	15,992,182	
1940	Chickamauga	33,886,213	
1940	Hiwassee	16,907,279	
1942	Watts Bar	33,070,549	
1943	Fort Loudoun	34,201,233	$309,111,978
1942	Nottely	$ 5,278,835	
	Chatuge	6,846,797	
	Cherokee	29,811,229	
	Douglas	38,952,464	$ 80,889,325
	Power		
1943	Ocoee No. 3	$ 7,659,247	
1943	Apalachia	22,364,338	$ 30,023,585
Other projects before 1944		$13,464,676	$ 13,464,676
	Steam-electric		
1942	Watts Bar	$14,067,329	
Other plants before 1944		9,837,404	$ 23,904,733
	Other electric plants		
Transmission lines, substations, etc.		$86,666,170	$ 86,666,170
Grand total			$544,060,467

from the TVA, serving 550,000 domestic, farm, commercial, industrial, and other consumers representing 2,861,000,000 kwhr of energy. Gross revenues were about $36,500,000, or 0.8 cent per kilowatt-hour, and a combined net income after all power expenses of about $6,360,000, or 0.2 cent per kilowatt-hour. The average use per consumer for domestic service was 1,707 kwhr as compared with 1,598 kwhr in 1943. The average domestic rate per kilowatt-hour for this service was 1.88 cents in 1944 and 1.96 cents in 1943.

Project Costs and Allocations. Project costs are shown in Table 13·5 as of June 30, 1944.[1]

The allocation of costs under normal multiple-purpose use, as of June 30, 1944, is given in Table 13·6.

Table 13·6 TVA Cost Allocation

Purpose	Multiple-purpose system			Other projects and electric plant	Total
	Single-purpose investment	Joint investment			
		%	Amount		
Navigation	$ 29,135,511.52	32	$ 78,437,279.97	....	$107,572,791.49
Flood control	9,261,600.00	28	68,632,619.97	....	77,893,619.97
Power	106,488,291.78	40	98,046,599.96	$154,059,163.51	358,594,055.25
Total	$144,884,803.30	100	$245,116,499.90	$154,059,163.51	$544,060,466.71

It is to be further noted that, during the period of emergency war power demand, the cost of the group of four plants, totaling $80,889,325, is temporarily charged to power, resulting in total amounts as follows:

Purpose	Total amounts	Per cent
Navigation	$100,119,127	18
Flood control	43,852,808	8
Power	400,088,532	74
Total	$544,060,467	100

By these allocations under normal operation, power will carry about $359,000,000, or 66 per cent of cost. In the period of war emergency, this was increased to about $400,000,000, or 74 per cent of the total amount.

Construction in progress in 1944 included the following plants:

[1] *Annual Report TVA*, 1944, pp. 178–180.

Project	Cost
Kentucky	$ 99,384,692
Fontana	58,813,524
South Holston	6,371,122
Watauga	4,011,826
Total	$168,581,164

Additions to dams, steam plants, etc., bring this total amount up to $188,241,438 as of June 30, 1944. Fixed assets also include $25,875,631, chiefly for the i ems of chemical and general plants. The total of these items is $757,477,536, from which, taking reserves for depreciation, $696,-012,200 is left as representing cost of projects. This is increased by cash, balance of appropriations, inventories, etc., to total assets of $763,666,809 as of June 30, 1944.[1]

The major portion of the fixed assets thus consists of properties now or presently to be constructed or developed. Included are land, land rights, and reservoir-clearing costs of $127,400,000. These are not included, however, in computing depreciation, which is on a straight line basis, with rates derived by studying each year of remaining useful life.

Liabilities as of June 30, 1944,[2] include

Appropriations by Congress		$667,969,270	
Funded debt	$63,072,500		
Current liabilities	15,477,047	78,549,547	
Transfer of assets from War Department		36,592,269	$783,111,086
Net expense of nonincome programs		$57,566,817	
Net income from power operations		38,122,540	19,444,277
Total liabilities			$763,666,809

As will be noted, Congressional appropriations constitute about 87 per cent of the above and funded debt about 10 per cent. The net income of about $38,000,000 from power operations for the period June 16, 1933, to June 30, 1943, helps to carry the net expense of nonincome programs, which include the following:

Net expense of nonincome programs June 16, 1933, to June 30, 1943:		
Navigation operations		$ 6,164,901
Flood-control operations		4,038,873
Other programs		39,828,981
Fiscal year ending June, 30, 1944:		
Navigation operations	$1,589, 18	
Flood-control operations	917,004	
Chemical operations	412,140	
Developed activities	4,615,700	7,534,062
Total		$57,566,817

[1] *Annual Report TVA*, 1944, p. 66.
[2] *Ibid.*, p. 67.

Cost of Power. Power generated for the year ending June 30, 1944, and its cost were as follows:[1]

Operation, maintenance, and other costs	$ 6,769,267	
Provision for depreciation and amortization	3,834,653	
Net production expense:		
Before common expense	$10,603,920	
After common expense		$11,478,491
Transmission	4,503,315	
Distribution	64,829	
Customers accounting and collecting	43,695	
Sales promotion	59,001	
Payments in lieu of taxes	2,168,798	
Administrative and general expenses	2,057,237	
Other expenses	116,910	
Amortization of electric-plant-acquisition adjustments	200,000	
		9,213,785
Total operating expenses		$20,692,276
Kilowatt-hours generated		10,117,748,000
Cost per kilowatt-hour generated		0.21¢

The above does not include interest charges upon the total amount of about $357,000,000 representing capital outlay for power projects. Depreciation and amortization allowances of $3,834,653, as used, are about 1 per cent. Payments in lieu of taxes of $2,168,798, or about 0.5 per cent of the capital outlay, as required by Section 13 of the TVA Act represent payment made to the states in which power operations are carried on, where properties have been acquired, previously subject to local and state taxation. For the year 1944, this was 7 per cent of the gross proceeds, derived from the sale of power for the previous fiscal year. This percentage gradually reduces and after 1948 will be 5 per cent. Depreciation, amortization, and payments in lieu of taxes thus constituted about 1.5 per cent of the capital outlay in 1944.

Interest charges upon capital outlay would be about 2½ per cent where financed by the United States, or about $9,000,000 yearly. If interest is included, the total yearly cost of power would be about $30,000,000, or 0.30 cent per kilowatt-hour, as compared with the average rate for power sold, of about 0.4 cent per kilowatt-hour.

Navigation (see Fig. 13·4). Authority was given by the TVA Act in 1933 to construct such dams on the Tennessee River and its tributaries as are required to provide a 9-ft navigation channel from the Ohio River at Paducah to Knoxville, Tenn., about 650 miles. At that time, the controlling depths were 4 ft from Wilson Dam and 2 ft from the latter to

[1] *Annual Report TVA*, 1944, Schedule G, p. 84.

Chattanooga. Development to the full navigable channel depth of 9 ft was completed in 1945 with a standard overdraft of 2 ft, 11 ft in all, for tugboats and barges. The entire length consists of slack-water pools crested by the TVA dams.

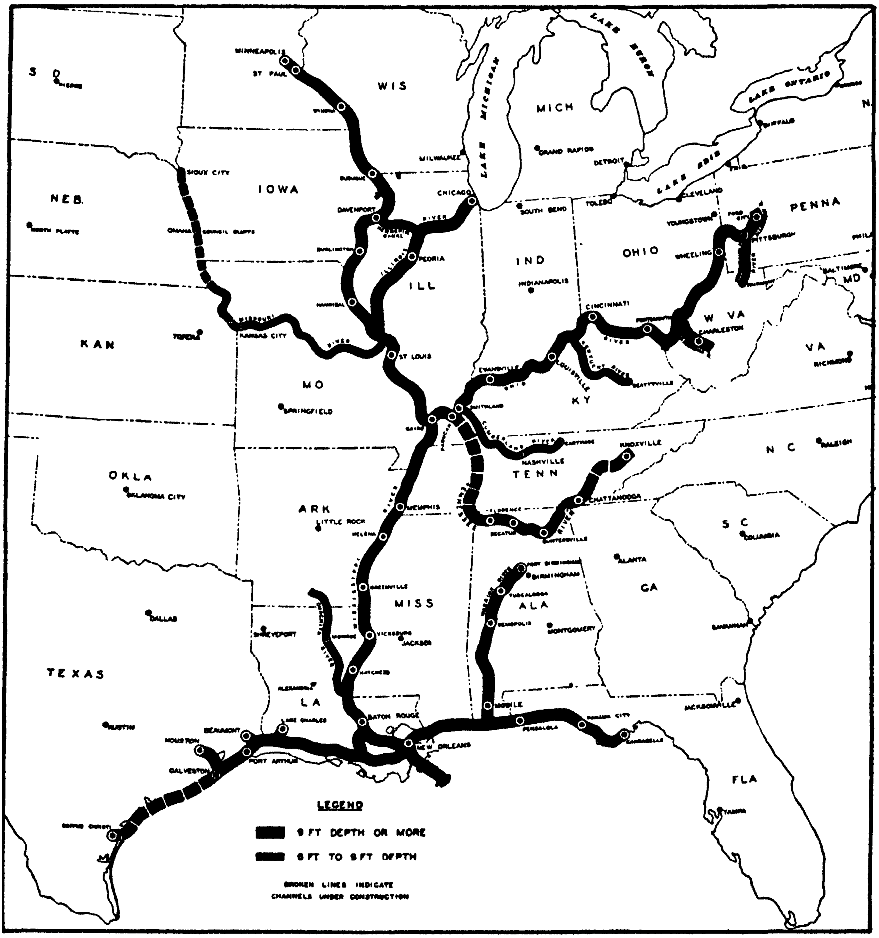


FIG. 13·4. Inland waterway system, Mississippi River Valley.

Locks at the nine dams at and below Fort Loudoun and their lifts are given below:

Freight carried during the war included large quantities of war-plant supplies and military materials. Substantial shipments of jeeps, trucks, ambulances, and weapon carriers, pig iron, limestone, machinery, coal in large quantities for the first time, grain, and petroleum products moved on

Dam	Lock chamber, ft	Maximum lift, ft
Fort Loudoun	60 × 360	80
Watts Bar	60 × 360	70
Chickamauga	60 × 360	56
Hales Bar	60 × 267	37
Guntersville	60 × 360	45
Wheeler	60 × 360	53
Wilson	60 × 300	90 (2 lifts)
Pickwick	110 × 600	63
Kentucky	110 × 600	73

the river in and subsequent to 1942. Inland shipyards were expanded and seagoing cargo and war vessels, built at these yards, traversed the river on their way to active service at sea.

Freight traffic on the river has been as follows by years:

Year	Ton-miles	Tons	Average haul, miles
1938	55,681,000	1,063,800	52
1939	91,317,000	1,115,200	82
1940	97,426,000	2,206,900	44
1941	138,396,000	3,093,700	45
1942	161,469,000	2,836,600	57
1943	206,568,000	2,869,200	72
1944	159,970,000	2,349,000	68
1945	287,527,000	2,586,500	111

The above data are based upon official data of the U.S. Engineer Corps. The years 1944 and 1945 are estimated, in 1946 freight traffic was 256,465,000 ton-miles.

The growth of river freight has been rapid, especially long-distance freight made possible by the extension of a continuous channel upstream and by the interconnection of the Tennessee with the improved inland-waterways system, spreading over the great interior of the country between the Great Lakes and the Gulf. In Fig. 13·4 is shown this interconnected inland-waterway system, which includes:

	Miles
9 ft or more	5,730
6–9 ft	2,450
4–6 ft	1,360
Total	9,540

Represented in traffic already developed are such widely separated river ports as Pittsburgh, New Orleans, Knoxville, Chattanooga, Chicago, Kansas City, St. Paul, Port Arthur (Texas), and many others.

Long-haul freight has included barge land shipments of grain, petroleum products, coal and coke, motor vehicles, fertilizer material, pig iron, forest products, iron and steel products, and limestone, with a beginning before the war of miscellaneous package freight in less-than-bargeload shipments between interior points in the Middle West and Southeast on joint barge-travel rates.

In 1940, the TVA made a river-traffic survey,[1] interviewing some 3,700 business concerns in 191 cities and towns in the TVA region. On the basis of this survey, it was estimated that about 780 million ton-miles yearly of freight could be transported on the improved channel, following its completion. This possibility includes the shipment of coal from east Tennessee and Kentucky mines to areas contiguous to the Upper Mississippi, Missouri, and Illinois rivers and would account for about 300 million ton-miles on the Tennessee. This shipment began during 1944, utilizing the public-use coal terminal at Harriman, Tenn. Public-use river terminals for the transfer and storage of general freight at five other points were completed in 1944 and are now in use. Eighteen barge lines are now in service.

The actual present and future use of the river depends upon savings in transportation costs as compared with other forms of transport. As to *existing* freight traffic, savings have not been computed but are said to be as much as $1,500,000 yearly. For the *future*, a study of this matter was made,[2] from which, based upon a total cost allocated to navigation of $150,000,000 (including $50,000,000 direct and $100,000,000 common costs), yearly costs or public aid to navigation would be $7,350,000, or 4.85 mills per ton-mile, including interest at 3.5 per cent and 2.61 mills without interest. A matured traffic (as of 1960) of 1,515,000,000 ton-miles or 7,000,000 tons with an average haul of 216 miles, is based upon the survey of 1941, adjusted upward to a moderate extent.

Annual savings are estimated at about $9,089,000, or $1.30 per ton. These are based upon the difference between anticipated waterway rates and charges that traffic would have to pay for an alternate means of transportation.

Floods. Records of flood flow have been kept at Chattanooga on the Tennessee River since 1874 and at Florence, Ala., since 1876.

In Table 13·7 are listed the 10 highest floods at these two stations, covering the period of 68 years, 1875–1942.

[1] Prospective Commerce on the Tennessee River, TVA, 1941.

[2] H.R. Doc. 159, 79th Cong., 1st Sess., 1944, Appendix C-3.

Table 13·7 Maximum Flood Discharge on the Tennessee River, 1875-1942

Date	Chattanooga, 21,400 sq miles		Date	Florence, 30,800 sq miles	
	Cfs	Csm		Cfs	Csm
Mar. 1, 1875	419,000	19.5	Mar. 19, 1897	444,000	14.4
Apr. 3, 1886	349,000	16.3	Jan. 22, 1882	397,000	12.8
Mar. 8, 1917	310,000	14.5	Mar. 8, 1875	395,000	12.8
Mar. 10, 1884	285,000	13.3	Apr. 8, 1886	372,000	12.0
Apr. 5, 1920	275,000	12.9	Dec. 29, 1926	344,000	11.2
Apr. 5, 1876	260,000	12.5	Mar. 14, 1884	325,000	10.5
Feb. 2, 1918	266,000	12.5	Mar. 12, 1917	317,000	10.3
Mar. 22, 1899	266,000	12.5	Mar. 18, 1890	315,000	10.2
Jan. 1, 1933	264,000	12.4	Apr. 6, 1936	313,000	10.2
Feb. 9, 1899	254,000	11.9	Feb. 15, 1884	310,000	10.1

Prior to 1875, in 1867, occurred a flood approximated at 460,000 cfs, or 21.4 csm, at Chattanooga—the apparent maximum in a period of about 75 years. This reached a gauge height of about 58 ft as compared with an average flood stage of about 208,000 cfs and about 30 ft gauge height.

According to estimates made by the U.S. Engineer Corps,[1] a maximum flood of 650,000 to 730,000 cfs may be expected at Chattanooga with unregulated river conditions at some future time. This approximates a thousand-year flood by the formula of Weston E. Fuller and is also consistent with Fig. 4·1, which is a chart showing maximum runoff rates in the Eastern United States prepared by the engineers of the TVA.[2] Using this, a maximum flow of 35 csm is indicated for 21,000 sq miles, or about 730,000 cfs.

Effect of Storage upon Floods. The total storage that will be available on the Tennessee River above Chattanooga is about 9 million acre-ft in 18 reservoirs (see Table 13·2). This represents an average of 420 acre-ft per square mile for the 21,400 sq miles of drainage area, or about 8 in. depth upon it. It is obvious that a high degree of control of a maximum flood is now available at and above Chattanooga. The present effective flood-control storage is about 5 million acre-ft.

Between Chattanooga and to above the Kentucky Reservoir, little additional storage is available. Thus, at Pickwick Landing (32,870 sq mi), the total usable storage upriver is about 10 million acre-ft, or about 5.6 in. depth. Above and including Kentucky Reservoir (40,200 sq mi.), storage totals about 15 million acre-ft in 24 reservoirs, or about 7 in.,

[1] HR. Doc. 479, 76th Cong., 2d Sess.

[2] "The Nottely Project," Exhibit 10 (unpublished).

which provides a substantial basis for lowering flood peaks upon the Ohio River near its mouth and upon the Lower Mississippi (see page 222).

Local Flood Protection at Chattanooga. Plans made in 1939 by the TVA and the U.S. Engineer Corps for local flood protection at Chattanooga are described elsewhere[1] (see page 378). These involve a cost of about $16,600,000, with their justification based upon an average yearly flood damage of about $1,800,000. With the TVA storage, which will be available above Chattanooga, it is expected that the "design flood" of 730,000 cfs will be reduced to about 500,000 cfs as a maximum, and the proposed levees will be based upon stage 60, or about 2 ft above the stage reached in the 1867 flood. These works have not yet been constructed (1947).

Concluding. The comprehensive development of the Tennessee River, the first of its kind in this country, as previously noted, has been carefully worked out and its engineering features well planned and executed, and it has shown what can be accomplished upon a large river with power capabilities like the Tennessee in developing and coordinating its resources, with practically unlimited funds.

Although power development was first considered as an incident of the the project, it has actually been of first importance and is contributing largely to its financing. This power development was thought by some to be of value as a "yardstick" to show a measure of fair charges for electric power, but it can hardly be said to have accomplished this purpose.

Conditions determining the generating cost of electric power under TVA auspices are widely different from those of the private utilities, with whom they are in competition. The cost of power in the case of the former includes operative depreciation and some payment in lieu of taxes but without interest charges. The utility companies, on the other hand, have to include all fixed and operating charges, including interest and taxes, and hence have a disadvantage in cost of wholesale power, as compared with the TVA. Moreover, as indicated in Table 13·6, considering the joint investment for navigation, flood control, and power, 60 per cent is allocated to the first two items and 40 per cent to power, thus further materially lessening power costs for TVA plants as compared with those for the plants of the utility companies, where no such allocation is made.

Furthermore, the cost of electric power to the ultimate consumer includes the cost of distribution, sales, and overhead charges, which much exceed the unit generating cost of power. It may better be said, therefore, that the TVA has given the power companies severe competition rather than a "yardstick" of fair rates.

[1] H.R. Doc. 91, 76th Cong., 1st Sess., 1939; also H.R. Doc. 479, 76th Cong., 2d Sess.

CHAPTER 14

CONNECTICUT AND MERRIMACK RIVERS

Connecticut River

Basin Description. The Connecticut River, the largest river system of New England, with a total drainage area of 11,260 sq miles, flows for a distance of about 300 miles between Canada and Long Island Sound. It is tidal in character to near Hartford, about 50 miles. Its basin reaches an elevation of 6,300 ft at Mount Washington in New Hampshire, and the river falls some 1,640 ft between First Connecticut Lake in its headwaters and its mouth. It is distinctly mountainous in Vermont and New Hampshire and western Massachusetts, with a wide main valley in Massachusetts and Connecticut (see Fig. 14·1).

It is an industrial area of importance, particularly in Massachusetts and Connecticut, with the manufacture of machinery, electrical goods, tools, metal products, textiles, and paper, with forest and quarrying products in the upper basin. The value of the total industrial output of the basin approximates $1,000,000,000 yearly.

The population of the basin was about 1,238,000 in 1930 with five cities of 40,000 or more population, including Hartford, Conn., 164,000 and Springfield, Mass., 150,000.

Adequate railroad and highway transportation facilities are available with commercial navigation up to Hartford. About 440,000 kw capacity of water power is developed with a yearly output in 1932 of about 1,500,000,000 kwhr, and some progress has been made in the upper river and on the Deerfield—a tributary entering in Massachusetts—toward storage development for power use.

Water Utilization. Seasonal rainfall and runoff conditions and water demands, especially for power use, are favorable to water conservation as a means of flood control. The severe floods have been of the melting-snow spring type or have occurred between September and November. In the use of water for power in New England, an empty reservoir by about Mar. 1 is contemplated so that full storage capacity in power reservoirs is available for spring floods. Reservoir draft begins about May 15 to June 1 and continues fairly steadily until the next March. By fall, sufficient storage capacity for adequate flood control is generally available. Hence the use of power reservoirs with flood control as a by-product is the most economical solution of the flood problem in New England. Experience upon the Deerfield River, where three major floods have occurred since 1927, has shown the practical effectiveness of this means of controlling floods in New England.

Runoff and Floods. The mean yearly runoff of the Connecticut River at or near Montague, Mass. (7,840 sq mi.), on a 33-year record was 13,690 cfs (1.75 csm), or about 24 in.

The three largest floods, all of which have occurred since that of 1927, show the following maximum discharges:

Station	Drainage area	November, 1927		March, 1936		September, 1938	
		Cfs	Csm	Cfs	Csm	Cfs	Csm
White River Junction	4,068	136,000	33	120,000	30	82,400	20
Montague	7,840	179,000	23	236,000	30	195,000	25
Thompsonville	9,637	190,000	20	282,000	29	236,000	25

Prior to 1927, the fourth flood in order of magnitude in a record of about 140 years was that of May, 1854, for which the approximate discharge near Montague was about 19 csm.

Flood Damages. Flood damages for the floods of 1936 and 1938 have been determined with care by the U.S. Engineer Corps. Those for the 1927 flood are known approximately. Direct flood losses for these three floods are given classified and in detail by states in Table 14·1.

The data in Table 14·1 are direct losses. Indirect losses were also determined for the floods of 1936 and may be estimated for 1927 and 1938, as shown in Table 14·2, using class ratios as of 1936.

Using the indirect damage ratios in Table 14·2 gives the total flood damages in Table 14·3.

It should be noted that the 1927 flood was essentially a tributary and upriver flood in Vermont and New Hampshire; 1936 was a main-river flood throughout its course but without, in general, outstanding tributary flows; and 1938 was essentially a tributary flood in Massachusetts and Connecticut resulting in lower main-river flows, between those of 1927 and 1936.

A plot of total flood damage (direct plus indirect) for the Connecticut River Basin against discharge in cfs at Montague, Mass. (7,850 sq miles), as an index station may be made based upon data in Table 14·3 and maximum discharges as previously given and also the approximate cfs (120,000) where damage is negligible.

Table 14·1 Connecticut River Basin—Flood Losses in Floods of 1927, 1936, and 1938

State	Direct flood damage, million dollars					
	Urban	Rural	Industrial	Railroad	Highway	Total
			Flood of November, 1927			
Vermont	1.78	0.169	1.181	2.891	4.960	10.981
New Hampshire	0.115	0.076	0.110	0.130	1.336	1.767
Massachusetts	0.505	0.275	0.507	0.575	0.295	2.157
Connecticut	0.275	0.145	0.126	0.075	0.000	0.621
Total	2.675	0.665	1 924	3.671	6.591	15.526
Per cent	17.1	4.3	12.4	23.7	42.5	100
			Flood of March, 1936			
Vermont	0.102	0.203	0.286	0.441	0.733	1.765
New Hampshire	0.060	0.156	0.517	0.167	1.368	2.268
Massachusetts	4.609	0.915	7.725	0.950	4.946	19.144
Connecticut	3.131	0.376	7.324	0.128	0.785	11.745
Total	7.902	1.650	15.852	1.686	7.832	34.922
Per cent	22.8	4.7	45.3	4.8	22.4	100
			Flood of September, 1938			
Vermont	0.407	0.231	0.267	0.065	2.839	3.809
New Hampshire	0.269	0.106	0 240	0.091	0.419	1.125
Massachusetts	2.777	0.905	4.735	1.628	5.508	15.553
Connecticut	1.686	0.681	1.772	0.095	0.875	5.109
Total	5.139	1.923	7.014	1.879	9.641	25.596
Per cent	20.0	7.6	27.4	7.3	37.7	100

Present Storage. Existing storage capacity for power use has been as follows:

Year	Storage capacity, acre-ft	Principal reservoirs
1927	340,000	Somerset, Harriman, Connecticut Lakes
1930	380,000	Comerford added
1938	480,000	Pittsburg added

Table 14·2 Indirect Flood-damage Ratios—Connecticut River Basin

Class of drainage	1936			1927		1938	
	Indirect damages as a ratio of direct	% of class	Weighted indirect damage ratio	% of class	Weighted indirect damage ratio	% of class	Weighted indirect damage ratio
Urban	1.14	22.8	0.26	17.1	0.20	20.0	0.23
Rural	0.10	4.7	0.01	4.3	0.01	7.6	0.01
Industrial	1.14	45.3	0.52	12.4	0.14	27.4	0.32
Railroad	0.70	4.8	0.03	23.7	0.16	7.3	0.05
Highway	0.50	22.4	0.11	42.5	0.22	37.7	0.19
Total	....	100	0.93	100	0.73	100	0.80

Of these reservoirs, Somerset and Harriman on the Deerfield River include 178,000 acre-ft; Connecticut Lakes 88,000; Pittsburg 100,000; and Comerford 40,000 acre-ft. All are in the upper Connecticut Basin.

The present power storage of 480,000 acre-ft represents about 1.1 in. depth in the drainage area of 8,000 sq miles on the lower river above Holyoke, Mass., and has been helpful in reducing flood effects, particularly upon the Deerfield River.

Table 14·3 Total Flood Damages—Connecticut River Basin

Flood damage	1927	1936	1938
Direct	$15,526,000	$34,922,000	$25,596,000
Indirect	11,300,000	32,000,000	20,500,000
Total	$26,826,000	$66,922,000	$46,096,000

In the lower river basin on the Swift River, Quabbin Reservoir for the water supply of the Boston metropolitan district has a capacity of about 1,275,000 acre-ft with a drainage area of 187 sq miles, or 6,800 acre-ft per square mile, or about 128 in. depth, or nearly 6 years average runoff, thus completely controlling this drainage basin (about 1½ per cent of the area above Holyoke).

Proposed Storage for Flood Control. Reservoir sites on the tributaries of the Connecticut in Vermont were selected and planned during 1928–1930.[1] Similar studies of available storage upon tributaries in New

[1] *Report of Advisory Committee of Engineers on Flood Control*, Vermont, 1930.

Hampshire were made during 1933. The U.S. Engineer Corps has made investigations and reported upon Connecticut River flood control as follows:

Date	Publication
1. February, 1935	H.R. Doc. 412, 74th Cong., 2d Sess.
2. November, 1937	H.R. Doc. 455, 75th Cong., 2d Sess.
3. March, 1940	H.R. Doc. 653, 76th Cong., 3d Sess.
4. May, 1940	H.R. Doc. 724, 76th Cong., 3d Sess.
5. 1944	H.R. Doc. 759, 77th Cong., 2d Sess.

In the first of these documents (H.R. Doc. 412), in 1935, is proposed a comprehensive plan of 33 reservoirs upon tributaries, combining flood protection with maximum benefits to power with a total storage capacity of 931,000 acre-ft and cost estimated at about $41,000,000. Ten reservoirs with 84,000 acre-ft, costing about $13,000,000, were selected as an initial plan and were authorized by Congress in the Flood Control Act of June 22, 1936.

In 1937 (H.R. Doc. 455), a comprehensive plan was submitted for the construction of 20 reservoirs (including the 10 previously authorized), estimated to cost about $35,000,000, and seven local-protection dike projects on the lower river in Massachusetts and Connecticut, at a cost of about $12,000,000. This was approved in the Flood Control Act of June 28, 1938 (see Fig. 14·1).

The further acts of 1940, 1941, 1942, and 1944 made some modification of the comprehensive plan. Appropriations have included:

1938	$11,524,000
1940	1,640,000
1941	6,000,000
	10,000,000
1942	420,000
1944	30,000,000
Total	$59,584,000

Twenty-reservoir Plan. As proposed in 1943,[1] these included 10 in Vermont, and 5 each in New Hampshire and Massachusetts, with the following summarized characteristics:

Type of dam, number:	
Concrete	2
Earth fill	18
Height, ft:	
Average	117
Range	31–300
Reservoir capacity, total acre-ft	1,235,000
Cost:	
Construction, land, and damages, total	$82,500,000
Per acre-ft	$67

[1] *Report of Chief of Engineers*, 1943, Vol. 1, Part I, p. 118.

This group included the Williamsville Reservoir on the West River in Vermont planned for power and flood control with the following characteristics:

Type	Concrete
Height, ft	300
Reservoir capacity, acre-ft	445,000
Cost:	
Construction	$25,000,000
Lands and damages	4,000,000
Total	$29,000,000
Cost per acre-ft	65

Objection to this project arose because of its excessive flowage damages; and, after considerable agitation, it was stricken from the list in the Act of 1944. It was ordered that only a flood-control dam should be built on the West River and that eight reservoirs with flood control equal to 75 per cent of that afforded by the Williamsville Reservoir should be substituted for the latter at a cost of not over $11,000,000.

With this modification allowing 150,000 acre-ft of storage in substitution for Williamsville Reservoir and a cost of $11,000,000, the 20-reservoir group would have a total capacity of about 1,000,000 acre-ft and would cost about $64,000,000, or $64 per acre-ft. Three of these reservoirs have been completed, *viz.*, Surry Mountain, Birch Hill, and Knightsville.

In Fig. 14·2 is a view of the dam at the Knightsville project. This is a hydraulic-fill dam, with a concrete spillway 400 ft long. The dam is 160 ft in maximum height and 1,600 ft long at the crest. The drainage area is 164 sq miles. The storage capacity is about 39,000 acre-ft, and the reservoir is operated as a retarding basin. The estimated cost for construction was $2,271,000; for land and damages $890,000; total cost $3,161,000, or about $82 per acre-ft and $19,300 per square mile of drainage area.

Local Flood-protection Works. In addition to the 20 flood-control reservoirs on the Connecticut River, 10 local flood-control projects consisting mostly of walls or levees and pumping stations are proposed, described in some detail in Chap. 18, pages 383–386. These will protect a total area of about 2,560 acres and are all located between Northampton, Mass., and Hartford, Conn.

These local works are planned with a 3-ft freeboard of levees and walls to take care of the maximum predicted flood of about 289,000 cfs at Holyoke (as compared with 247,000 cfs in 1936), with the 20-reservoir plan assumed in operation. This total estimated cost of local works is about $19,000,000.[1]

Total Cost. The total estimated cost of reservoirs and local works is therefore about $83,000,000, or about $10,000 per square mile of drainage

[1] *Ibid.*, p. 119.

area at Holyoke. The lack of economic justification for this plan for flood control upon the Connecticut River is discussed in Chap. 8, pages 153–155, where it is shown that an expenditure for flood control of not over about \$40,000,000 is warranted. The plan, as estimated in 1937,[1] had a cost of about \$35,000,000 for reservoirs and \$12,000,000 for local works, or a total

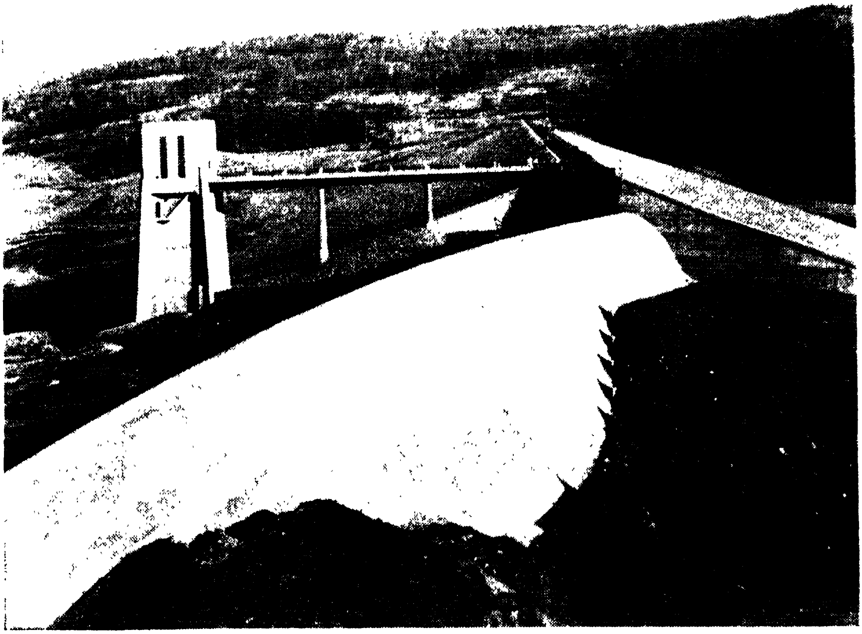


FIG. 14·2. Knightsville Dam, Connecticut River

of \$47,000,000. The present cost estimate of \$83,000,000 is over double the amount that is, in the judgment of the author, warranted and provides control by reservoirs of only about 30 per cent of the area above the lower river (see page 150). Furthermore, it reduces the maximum flood by only about 30 per cent, and it may be found necessary to provide an even greater expenditure to meet future requirements.

It is unfortunate that a more useful and less costly flood-protection scheme for the Connecticut River was not worked out. Such a scheme would have been possible in many cases through the use of power-storage–multiple-use reservoirs as planned by the Water Resources Board of the National Resources Committee in 1937,[2] developing flood control as a by-

[1] H.R. Doc. 455, 75th Cong., 2d Sess., 1937, pp. 23–25.
[2] Flood Control in N.E., *Civil Eng.*, November, 1937.

product. These effect net savings in cost, as well as improvement of low-water flow, the development of the water resources of the region, and improvement of sanitary conditions, in contrast to costly retarding-basin-type reservoirs, which are of no use except for flood control and are unsightly as well in their operation.

Of the 20 proposed reservoir projects, however, only three have yet (1947) been constructed, all of the retarding-basin type. It may not be too late to review these projects, plan developments that will be more worth while, and produce results that will add to the water resources and prosperity of the Connecticut valley by properly developing its rivers as well as reducing the menace of floods.

Merrimack River

The Merrimack River is formed by the Pemigewasset and Winnepesaukee Rivers at Franklin, N.H., flows south through central New Hampshire, and thence east, entering the Atlantic Ocean at Newburyport, Mass. It is 116 miles long and has a range and drainage area of 5,015 sq miles. The lower river is tidal for 22 miles above its mouth. The topography of the watershed ranges from rugged, mountainous areas through narrow valleys in the northern portion to gently rolling terrain in the lower valley. The Merrimack River Basin is shown in Fig. 11-1, page 297.

The population of the basin was about 810,000 in 1930. The greater part of this is concentrated in cities and towns in the highly industrialized lower valley. Manufacturing has already been of importance in the basin, initially because of the availability of good water power. The principal products are textiles, leather goods, and wood products.

The water power of the main river and tributaries has been extensively developed first for mechanical power for numerous mills and in later years for hydroelectric power. About 200,000 hp of water power is developed with storage capacity totaling 586,000 acre-ft operated chiefly for power purposes.

In the so-called "308 Report,"[1] it was concluded by the U.S. Engineers that floods were not especially important on the Merrimack River and that a system of reservoirs estimated to cost about $8,000,000, primarily for production of water power, with flood control as a by-product, would be advisable. At that time, the recent flood of importance was that of 1927, in which damages were estimated at about $2,400,000. The flood of March, 1936, changed this viewpoint materially, since a maximum flow occurred at Lawrence, Mass., of 160,000 cfs, or some 80 per cent larger than that for the 1852 flood, the greatest of previous record. The total flood damages in the Merrimack Basin for 1936 were estimated at about $34,000,000,

[1] H.R. Doc. 649, 71st Cong., 3d Sess., 1930.

and some eight lives were lost. The estimated average yearly flood losses, as made in 1938, are $1,100,000.

The Flood Control Act of June 22, 1936, authorized the construction of a system of flood-control reservoirs in the Merrimack River Basin at an estimated construction cost of $7,725,000, contingent upon a further amount of $3,500,000 for lands and damages to be provided by the states. The Act of June 22, 1938, authorized construction of a system of reservoirs at a cost of about $21,000,000, of which $13,000,000 was for construction and $8,000,000 for lands and damages.[1] Two of these reservoirs, *viz.*, Franklin Falls and Blackwater, have been constructed and are in use.

Four more reservoirs are now under consideration (1945). The proposed group, including the two constructed, will have the following total characteristics:

Drainage area controlled	1,434 sq miles
Storage capacity	329,000 acre-ft or 4.2 in.
Cost	$18,300,000

The controlled drainage area is about 28 per cent of that of the lower river and, including the 486 sq miles controlled by Lake Winnepesaukee, 1,920 sq miles, or about 38 per cent.

Franklin Falls Reservoir.[2] This is a retarding-basin-type flood-control reservoir located about 2½ miles above Franklin, N.H., on the main river, where the drainage area is 1,000 sq miles. The dam is of rolled earth fill, with dumped-rock shell and toe 1,740 ft long and 136 ft in maximum height, with about 3 million cu yd of earth and rock fill. Storage capacity is 170,000 acre-ft, or about 3.2 in. depth on the drainage area.

A 550-ft concrete spillway, with 810 sq ft of gate-controlled outlet conduits, power intake and 23-ft tunnel, required about 82,000 cu yd of concrete. The total cost, including construction, land damages, and highway relocation, was about $8,000,000, or about $47 per acre-ft of storage capacity.

Blackwater Reservoir.[3] This is located upon the Blackwater River in the towns of Webster and Salisbury, about 8 miles above its confluence with the Contoocook River, about 13 miles northwest of Concord, N.H. The drainage area is 127 sq miles. The dam is of the rolled-earth type with dumped-earth toe and dumped-rock blanket on the upstream face 750 ft long, top width of 39 ft, and maximum height of about 60 ft, with about 240,000 cu yd of earth and rock fill.

[1] H.R. Doc. 689, 75th Cong., 3d Sess., 1938.

[2] *Report of Chief of Engineers*, 1944, Vol. 1, Part 1, pp. 46–48.

[3] *Ibid.*, pp. 46, 50.

A 240-ft concrete spillway, with 55 sq ft of gate-controlled and 23 sq ft of uncontrolled outlet conduits, required about 23,000 cu yd of concrete. The storage capacity is 46,000 acre-ft, or about 7 in. depth in the drainage area. The cost was about $1,200,000, or $26 per acre-foot of storage capacity.

Comments upon Reservoirs Constructed. *Franklin.* In the judgment of the author, this should have been a power–flood-control reservoir at Livermore Falls,[1] upriver about 25 miles where the drainage area is 408 sq miles. The storage capacity there would be about 15 in. depth or 408 × 15 = 6,100 "inch-miles" as compared with 3.2 × 1,000 = 3,200 "inch-miles" for the Franklin retarding-basin type.

Although Livermore Falls has a much smaller drainage area, it would control an area of much higher unit flood runoff and for major floods have a greater flood-reducing tendency than Franklin, on the main river below. Livermore Falls would also protect the towns of Bristol and Plymouth and an area subject to severe floods, not protected by Franklin. The latter reservoir is too small for its drainage area.

Using cost and other data as given by the U.S. Engineers,[2] a comparison of the Franklin and Livermore Falls Reservoirs is made in Table 14-4.

Table 14-4 Comparison of Franklin Falls and Livermore Falls Reservoir Sites*

Item	Franklin Falls	Livermore Falls
Drainage area controlled, sq miles	1,000	408
Water area, acres	2,950	6,800
Storage capacity:		
Acre-ft	170,000	335,000
In. depth	3.2	15.4
Cost:		
Total	$7,883,000	$13,300,000
Per acre-ft	$ 46.30	$ 39.70
Yearly benefits:		
Power	$ 39,000	$ 574,300
Sanitation	83,000	100,000
Total	$ 122,000	$ 674,300
Total benefits capitalized at 8 per cent	$1,500,000	$ 8,400,000
Net cost of project (cost less benefits):		
Total	$6,400,000	$ 4,900,000
Per acre-ft	$ 37.70	$ 14.60

* Based upon data taken from *U.S. Engineer Corps Report.*

[1] Memorandum (unpublished) upon "Flood Control in the Merrimack River Basin," submitted at hearing of U.S. Engineer Corps, Concord, N.H., Dec. 21, 1938, by H. K. Barrows.

[2] H.R. Doc. 689, 75th Cong., 3d Sess., pp. 82–91; also *ibid.*, p. 5.

The comparison indicates that the net costs of Franklin and Livermore Falls storage reservoirs are $6,400,000 and $4,900,000, respectively, or $37.70 and $14.60, respectively, per acre-foot, or a saving of $1,500,000 in favor of Livermore Falls, with superior flood-reducing tendencies.

In this comparison, the benefits accorded to Franklin for power and sanitation would not result, if as at present it is being used as a retarding basin. This makes Livermore Falls of even greater relative advantage.

Blackwater. Here, too, could better have been a power–flood-control reservoir, with a saving in net cost and other benefits not now possible, since the reservoir is being operated. The Blackwater has an excellent power site with an available head of about 200 ft, which has been under consideration for development. An additional fall of 100 ft upon the Contoocook River at Penacook is available for a run-of-river plant, from a drainage area of about 700 sq miles, which, developed with the Blackwater as a storage project with about 13 in. capacity, would make an attractive power project and incidentally control the flow of the Blackwater River.

Proposed Reservoir System. Including the Franklin and Blackwater reservoirs as constructed, a group of six reservoirs is planned for the Merrimack Basin with characteristics as follows:

Total drainage area controlled	1,434 sq miles
Storage capacity	329,000 acre-ft, or 4.3 in. above Lowell, Mass. (4,100 sq miles)
Cost	$18,000,000
Cost per acre-ft	$55
Area protected	35 per cent of drainage area above Lowell

Existing storage, totaling about 586,000 acre-ft (or 2.7 in.) controls about 1,020 sq miles, or 25 per cent of the area above Lowell. Included is the Winnepesaukee River, where about 426 sq miles is completely controlled by about 168,000 acre-ft (or about 7.5 in.) of storage in Lakes Winnepesaukee and Wentworth. Thus proposed and existing storage totals 915,000 acre-ft, or about 4 in., above Lowell and would control about 60 per cent of that drainage area.

Local Flood-control Projects. Six local flood-control projects at Merrimack River cities and towns in New Hampshire and Massachusetts are planned. Some details of these are given in Chap. 18 page 386. These are partly constructed and will protect a total of about 1,155 acres of urban area at a total cost of about $5,200,000.

Flood Control—Merrimack River. The total cost of the reservoir system and local flood works on the Merrimack River will be about $26,000,000, or a yearly cost of $1,300,000, allowing 5 per cent for fixed charges, depreciation, and operation. The value of complete elimination of flood losses was stated to be $1,079,000,[1] or a ratio of benefits to cost of 0.83, indicating doubtful economy. [This ratio will be bettered slightly if the system of reservoirs now proposed (1945) is adopted at a cost of $18,300,000 and a total cost of about $23,000,000.]

This situation still further accentuates the fact that the Franklin and Blackwater reservoirs are too costly for flood control alone. As previously stated, they should have been the power–flood-control type of reservoir, which would permit a substantial net saving in cost.

Aside from the matter of cost, an opportunity has also been lost to add to the conservation development of the Merrimack and to obtain the benefits resulting therefrom. Such works should be planned for the future permanent improvement of the river, where, as in this case, it can be done and at the same time provide for flood control.

The foregoing criticism of the Franklin and Blackwater reservoirs and Merrimack River flood control was made in 1937 by the author[2] and again in 1938 at the hearing at Concord, N.H.

[1] H. R. Doc. 689, 75th Cong., 3d Sess., p. 92, par. 129.

[2] BARROWS, H. K.: Water Resources Committee Recommendations for New England, *Civil Eng.*, November, 1937, pp. 740–744.

CHAPTER 15

WILLAMETTE RIVER, BONNEVILLE, LOS ANGELES

Willamette River

The Willamette River and its tributaries drain an area in northwestern Oregon of about 11,200 sq miles, forming a part of the greater Columbia River Basin. It is roughly rectangular in shape, 150 miles long—north and south—and 75 miles wide. It ranges in elevation from about 200 to 10,000 ft in the Cascade Mountains on the east (see Fig. 15·1).

The mountain areas, comprising much of the basin, are rugged and generally forest-covered. The valley floor is a broad alluvial plain, some 3,500 sq miles in area, with winding stream courses and rich farm lands. Production of timber and wood products, and of pulp and paper are the main industries, with general manufacturing and processing of foods, particularly in the Portland area. The population of the basin was about 721,000 in 1930, Portland, with about 300,000, being the principal city. Excellent transportation and terminal facilities are available with deep-water navigation to the Portland area.

Water Utilization. Seasonal rainfall and runoff conditions and water demands are exceptionally favorable to water conservation involving multiple uses of water and of storage capacity. Floods may be expected from November through April, but major floods are normally confined to the period from late November to early February. Hence storage space reserved for flood control during the winter months can be wholly or partly filled after the flood hazard has passed, and the water can be made available for power, irrigation, and other purposes.

Runoff and Floods. The mean yearly runoff of the Willamette River at Albany (4,840 sq miles) over a 40-year period was 13,700 cfs (2.83 csm), or 38.7 in. Large floods are a result of southwest storms on snowfall in the higher elevations, accompanied by a rise in temperature and heavy precipitation on the valley flow, the storm having the same general direction of travel as the flood wave, and the unit flow tending to increase as the mouth is approached and the drainage area increases.

The three largest floods of record show discharge as given on page 310.

Flood Problem. The Willamette River and the lower reaches of its principal tributaries flow between low banks bordered by extensive flat flood plains, with bank-full capacities ranging from 10 to 30 csm. The

Station	Drainage area, sq miles	December, 1861		January, 1881		February, 1890	
		Cfs	Csm	Cfs	Csm	Cfs	Csm
Eugene	2,030	104,000	51	108,000	53	113,000	56
Albany	4,840	340,000	71	229,000	47	250,000	52
Salem	7,280	500,000	69	428,000	59	450,000	62
Portland	11,200	635,000	57	544,000	48	570,000	51

frequency of such overflow is indicated by the record at Albany, where in 47 years of record the river has been at or above bank-full stage eighty times. Areas flooded have been as follows in 1927 and 1861:

Reach	1927		1861	
	Acres	Per cent	Acres	Per cent
Above Eugene	13,600	5	19,500	6
Eugene to Albany	134,100	49	174,000	49
Albany to Salem	78,800	29	100,000	28
Salem to Oregon City	46,200	17	59,200	17
Total	272,700	100	352,700	100

The 1927 flood, a 5-year average flood, exceeded bank-full stage at Albany by 10 ft; the 1861 flood, with a 100-year frequency (although two floods of this magnitude, one in 1890, have occurred in 77 years), exceeded bank-full stage at Albany by 19 ft.

Since the floods of 1861 and 1890, the population of the Willamette Basin has increased greatly, and the present value of lands and improvements within the 1861 flood plain is estimated at nearly $100,000,000.

A recurrence of the 1861 stage would submerge the communities of Cottage Grove, Junction City, Harrisburg, and West Salem under 8 to 13 ft of water. Some 7,000 farm units, with village and suburban homes and stores, would be more or less inundated, with doubtless some loss of life and impairment of health. Damage in a flood similar to that of 1861 is estimated at over $10,000,000, while the 5-year average flood of 1927 would cause damages of about $4,000,000. Details of the flood-damage appraisal, based upon present conditions of development, are given in Table 15·1. Inasmuch as even larger floods than that of 1861 are possible, flood control is an important problem in the Willamette valley.

Coordinated Plan of Storage Development. The coordinated plan of storage development essentially includes a system of seven storage reservoirs located on major tributaries of the Willamette River. Details

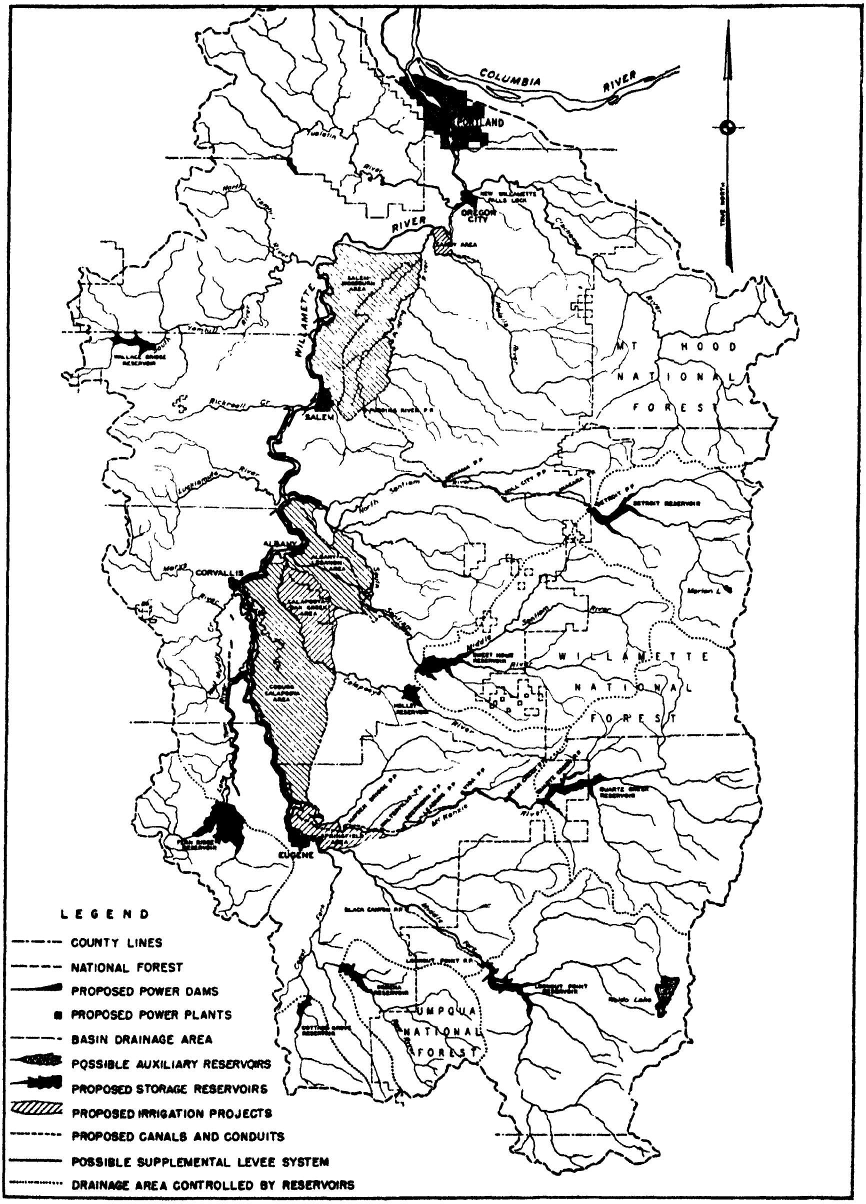


FIG. 15·1. Willamette River. Drainage area and reservoirs.

Table 15·1 Willamette River Basin—Estimates of Flood Damage

Description	1927 flood, gross overflow area 272,700 acres		1861 flood, gross overflow area 352,700 acres	
	Damage	%	Damage	%
Land surface and farm crops	$1,601,000	38.6	$ 3,284,000	30.8
Permanent land damage*	897,000	21.6	1,807,000	17.0
Farm improvements	197,000	4.7	604,000	5.7
Livestock	65,000	1.6	254,000	2.4
Suburban residential properties	104,000	2.5	157,000	1.5
Suburban business properties	175,000	4.2	208,000	2.0
Urban properties and services	339,000	8.1	2,307,000	21.6
Highways and bridges	297,000	7.1	785,000	7.4
Highway traffic interruptions	326,000	7.8	751,000	7.1
Railroad properties and services	56,000	1.3	351,000	3.3
Bank erosion†	104,000	2.5	132,000	1.2
Total	$4,161,000	100	$10,640,000	100

* Nonrecurring damage. Includes land damage due to surface erosion, scour, and wash overlay.
† Includes damages to banks, bank protection, and navigation channels.

of these proposed reservoirs are given in Table 15.2 (see also Fig. 15·1). These will provide a total usable storage capacity of 1,820,000 acre-ft at a cost estimated at $73,741,000, or $40 per acre-foot.

In addition to the reservoir system, a navigation lock is proposed at Willamette Falls, which, together with certain navigation and bank protective works, will bring the total project first cost to $78,341,000 and the yearly cost to $4,300,000.

In Fig. 15·2 is a view of the concrete spillway and a portion of the earthfill dam (which is 1,650 ft long) at the Fern Ridge site. The spillway is 264 ft long and has outlet works consisting of three gate-controlled conduits.

At the Detroit Reservoir, a powerhouse will be located on the right bank just below the dam with three 45,000-hp hydraulic turbines under a head of 275 ft and a total generator capacity of 90,000 kw. Hydroelectric power is also to be developed at the Lookout Point and Quartz Creek sites, with about 150,000 kw more of generator capacity, utilizing heads of 216 and 223 ft, respectively. Including downriver sites, a total of over 400,000 kw is planned for development, with a total head of over 1,900 ft.

Benefits from Coordinated Plan. The benefits that would accrue are estimated as follows:

1. *Flood Control.* Of the potential annual flood damage of $1,693,000, 80 per cent, or $1,343,980, would be eliminated, together with some enhancement of land and property values, making the total benefits about $1,526,000 yearly. Flood discharge at Albany will be reduced

Table 15·2 Willamette River Basin—Proposed Reservoir System (Revised 1941)

Reservoir	Stream	Drainage area, sq miles	Storage proposed, usable acre-ft	Height of dam,* ft	Cost: Total	Cost: Yearly	Cost: Per cent	Cost: Per acre-ft
Cottage Grove	Coast Fork	104	30,000	95	$ 2,360,000	$ 137,000	5.8	$79
Dorena	Row River	265	70,000	145	4,276,000	234,000	5.5	61
Lookout Point	Middle Fork	915	433,000	293	19,349,000	970,000	5.4	45
Quartz Creek	McKenzie River	899	560,000	298	15,447,000	785,000	5.1	38
Fern Ridge†	Long Tom River	252	95,000	44	2,550,000	144,000	5.65	27
Sweet Home	South Santiam River	583	310,000	147	7,644,000	425,000	5.6	25
Detroit†	North Santiam River	438	322,000	371	22,115,000	1,140,000	5.15	69
Total		3,456	1,820,000	.....	$73,741,000	$3,835,000	5.2	$49

* Height from normal tail-water elevation to normal pool elevation.
† Revised cost estimates were made in 1944 as follows:
Fern Ridge $3,971,000
Detroit 27,650,000
Revised total, seven projects $80,588,000

one-half, and flood height 9 ft for the 25-year flood; and the area flooded will be reduced to a small amount.

2. *Navigation Improvements.* A minimum flow of 5,000 cfs at Albany and 6,500 cfs at the mouth of the Santiam River, with commercial navigation to Albany, estimated at $834,000 yearly.
3. *Irrigation.* Yearly available 560,000 acre-ft of water for irrigation use or sufficient for supplemental irrigation of 355,000 acres in the basin, estimated at $519,000 yearly.

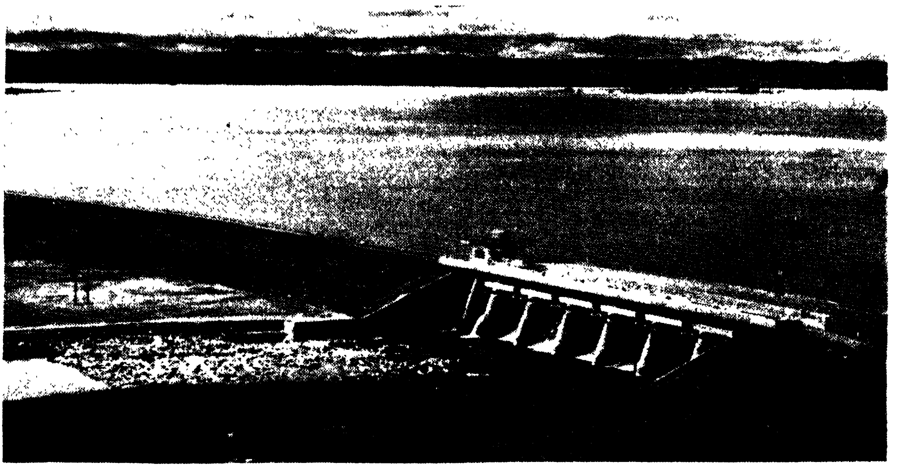


Fig. 15·2. Fern Ridge Dam and Reservoir.

4. *Power.* Firm power capacity 376,900 kw and firm output of 1,957,000,000 kwhr yearly from plants upon tributaries, and an additional 21,000,000 kwhr at the existing development at Willamette Falls. The total potential power value due to reservoirs is estimated at $4,083,000 yearly.
5. *Sanitary Conditions.* Minimum discharges on the upper river increased threefold, and double on the lower river. This dilution and benefit to sanitary conditions and fish life are estimated at $90,000 yearly.

Summarizing. The total yearly benefits accruing from the coordinated plan total about $7,052,000 yearly, distributed as follows:

	Per Cent
Flood control	22
Navigation	12
Irrigation	7
Power	58
Sanitation	1
Total	100

If power benefits are excluded from consideration, because of the uncertainty as to future power requirements, yearly benefits would be $2,969,000 and would carry about 70 per cent of the yearly cost. The remainder of the yearly cost, or $1,326,000, must be carried by the present worth of future power values, which were estimated at about three times this amount. This is consistent with a present worth, if used 28 years hence, at 4 per cent, which is approximately one-third.

Initial Development. An initial development is proposed as a first step, excluding power storage and with the initial power dams and reservoirs constructed to partial height, with certain provisions for future raising when additional storage is justified by power requirements. This reduces the cost of the power reservoirs as follows:

Lookout Point	$ 4,510,000
Quartz Creek	7,165,000
Detroit	6,593,000
Total	$18,268,000

The total first cost is reduced from $79,955,000 to about $59,387,000 and the yearly cost to $3,200,000, of which a small unit, $152,000 yearly, represents initial cost chargeable to future power values.

The net yearly cost of the initial development is a little over $3,000,000 against yearly benefits as follows:

Items	Yearly benefit	Per cent of total
Flood control	$1,526,000	49
Navigation	834,000	27
Irrigation	519,000	16
Power	157,000	5
Sanitation	90,000	3
Total	$3,126,000	100

The initial development is just about economically feasible, although it must be kept in mind that some 20 per cent of the estimated yearly benefits are credited to irrigation and incidental power use, which are not matters of certainty. The Flood Control Committee approved the comprehensive plan of development, as well as the initial plan, for the Willamette River Basin, and an appropriation was made in 1938 of $11,300,000 toward the initiation and partial accomplishment of the initial plan.

References

H.R. Rept. 2353, 75th Cong., 3d Sess., 1938, pp. 25–26.
Public Doc. 761, 75th Cong., 3d Sess., 1938, H.R. 10618, p. 9.

Report of Chief of Engineers, 1941.
Willamette River, H.R. Doc. 544, 75th Cong., 3d Sess.

Fern Ridge Reservoir. This is on the Long Tom River 38 miles from its mouth. This stream rises in Lane County, Oregon, on the eastern slope of the Coast Range, flows north for 50 miles, and enters the Willamette River 147 miles above its mouth (see Fig. 15·1). The drainage area is 252 sq miles.

The dam is of earth fill, 44 ft in maximum height and 6,360 ft long at the crest with two auxiliary dikes, totaling about 4,500 ft long at the north-

FIG. 15·3. Bank protection, Long Tom River.

easterly boundary of the reservoir. The spillway, near the left abutment, is of concrete, controlled by six automatic radial gates, with outlet works within the spillway structure. In Fig. 15·2 is a view of the dam, showing the concrete spillway in the foreground and the earth dam at the left.

The project includes the rectification and stabilization of 23.5 miles of the channel of the Long Tom River downstream from the dam. In Fig. 15·3 is a view of a portion of this work, showing bank-protection measures. The new canal section will have a capacity of 3,000 cfs at the dam and 4,800 cfs at the mouth of the Long Tom River and will reduce the area of lands submerged during flood stages of the river or peak discharges from the reservoir.

Detroit Reservoir. This site is on the upper part of the North Santiam River, in the easterly portion of the Willamette Basin. The North Santiam

River rises in Marion and Linn Counties, Oregon, on the western slope of the Cascade Range, flows north and west for 85 miles, and unites with the South Santiam River to form the Santiam River, which enters the Willamette River 10 miles downstream, 108 miles above its mouth. The drainage area tributary to this reservoir is 438 sq miles.

The dam is to be constructed to its ultimate height, together with power-generating facilities, in one step. It will be of the concrete gravity type 370 ft in height, from river to normal pool level at El. 1,569, and approximately 1,580 ft in length. The spillway will be an overfall section in the dam controlled by radial gates. The outlet works will consist of needle-valve and ring-seal gate-controlled conduits through the dam. The powerhouse will be located on the right bank immediately downstream from the dam. Three penstocks, each 12 ft in diameter, will supply water through a maximum head of 371 ft to three Francis-type turbines. These turbines will have a capacity of 45,000 hp each, when operating under a head of 275 ft, and will be directly connected to three 30,000-kw generators, giving a total capacity at this head of 90,000 kw. The reservoir will have a usable capacity of 322,000 acre-ft. It will be operated as a unit in the coordinated reservoir system to protect the Willamette Valley from floods, to develop power, to increase low-water flows in the interests of navigation, and for other purposes. Work is in a preliminary stage.

Bonneville Dam and Power Development—Columbia River[1]

This project is located on the Columbia River 43 miles east of Portland Oreg., and about 144 miles above the mouth of the river. It was completed in 1938 by the U.S. War Department and provides for a dam, power plant, and lock, for power and navigation. The spillway dam extends across the main-river channel from the Washington shore to Bradford Island, which divides the river at this point (see Fig. 15·4). The dam is about 170 ft in maximum height and 1,100 ft long.

The overflow crest at 24 ft above mean sea level is surmounted by 18 vertical-lift steel gates placed between piers that extend to El. 99 ft, where a service roadway permits the operation of two 350-ton gantry cranes for regulating the gates. The normal peak elevation is 72 ft.

The powerhouse extending across Bradford Slough to the Oregon shore has an installation of 10 units, consisting of two units of 43,200 kw each and eight units of 54,000 kw each, totaling 518,400 kw, under a head averaging about 54 ft. Generation of power during 1945 was about 3.4 billion kwhr.

The ordinary and extreme fluctuations of the river at the lower lock gate are about 21 and 47 ft, respectively. The project includes fish ladders and

[1] *Report of Chief of Engineers*, 1944, Vol. 2, Part I, p. 1628.

locks to serve both the main channel and the Bradford Slough channel. The lock and powerhouse are founded on andesite, and the main dam rests upon solidified sedimentary rock of volcanic origin. The pool created by the spillway dam provides a navigable channel, with depths of 30 ft and over, between Bonneville and Big Eddy, a distance of 47 miles.

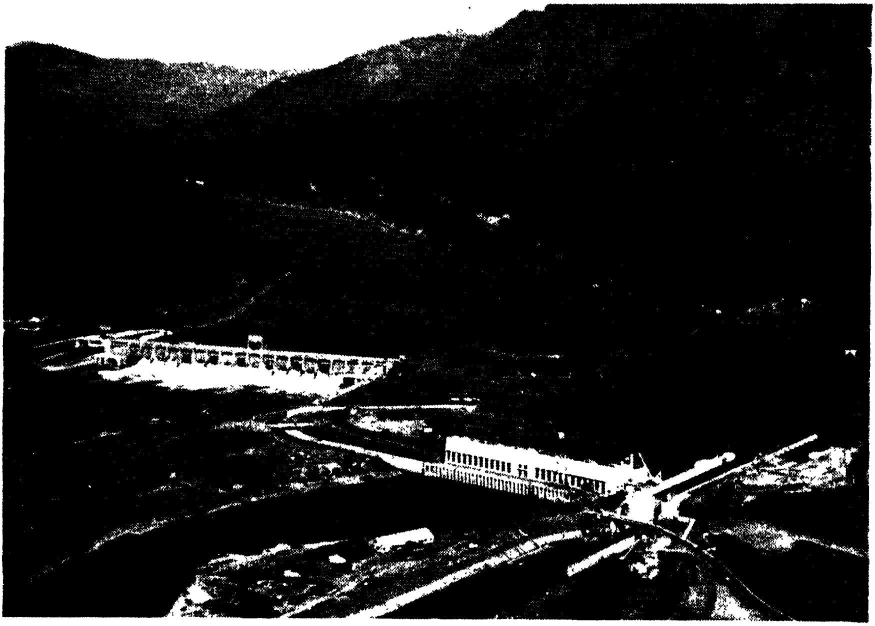


FIG. 15·4. Bonneville Dam and power development.

The estimated cost of the dam, lock, 10 power units, and fishways, revised in 1942, is $81,386,000, of which $39,186,000 is for power facilities and $42,200,000 for facilities other than power. The estimate of cost for annual operation and maintenance, made in 1937, is $312,000.

Los Angeles Flood-control Projects[1]

An area of 1,717 sq miles in the general vicinity of Los Angeles is drained by the Los Angeles and San Gabriel rivers and Ballona Creek. The March, 1938, flood upon these streams and resulting damages have been described on page 134. Floods are comparatively frequent, and about 30 per cent of their area is subject to overflow. Moreover, the natural channels in the lower reaches are not stable and tend to change their courses.

Not all of this area would be overflowed by any one flood; but, because of the instability of natural channels and the relative freedom of flood flows to

[1] H.R. Doc. 838, 76th Cong., 3d Sess.; also H.R. Doc. 426, 77th Cong., 1st Sess.

follow various courses over the debris cones and through the alluvial valleys, the whole area is threatened.

The population of the drainage area in 1939 was estimated at 2,400,000, with 44 incorporated cities. The largest of these is Los Angeles, with a population of 1,504,000 (in 1940) and the greatest area of any city in the United States. Its major productive activities include petroleum and refining, motion pictures, airplanes, automobile assembling, and agriculture.

Local agencies have made extensive flood-relief improvements. The Los Angeles County Flood Control District was enacted in 1915 and includes substantially the entire drainage area of 1,717 sq miles. The agency prepared a comprehensive plan of development and has constructed 12 flood-control dams, 2 flood-control and debris storage basins, 1 diversion dam, many miles of channel improvements, etc.; and it maintains and operates these works.

The total expenditure by the district to Dec. 1, 1939, was approximately \$58,000,000 for construction and \$11,000,000 for maintenance. In addition, expenditures aggregating about \$42,000,000 have been made by the city of Los Angeles and by other cities and towns.

Since 1935, Federal agencies, chiefly the Corps of Engineers, have participated in this flood-control work. Up to Jan. 1, 1940, the following amounts were expended:

Local	\$110,000,000
WPA	36,000,000
Other agencies	4,000,000
U.S. Engineer Department Corps	53,000,000
Total	\$203,000,000

In 1940, the Chief of Engineers recommended a further plan of flood-relief improvement on the three basins, which will require (to complete the existing project and make extensions) the following approximate expenditures (revised to 1942):

	Revised (1942)
Los Angeles River Basin	\$132,164,000
San Gabriel River Basin	88,618,000
Rio Hondo	22,558,000
Ballona Creek	19,855,000
Total	\$263,195,000

In Fig. 15·5 is a general plan of the projects for flood control in the general vicinity of Los Angeles, as proposed in 1940, showing also other previous projects.

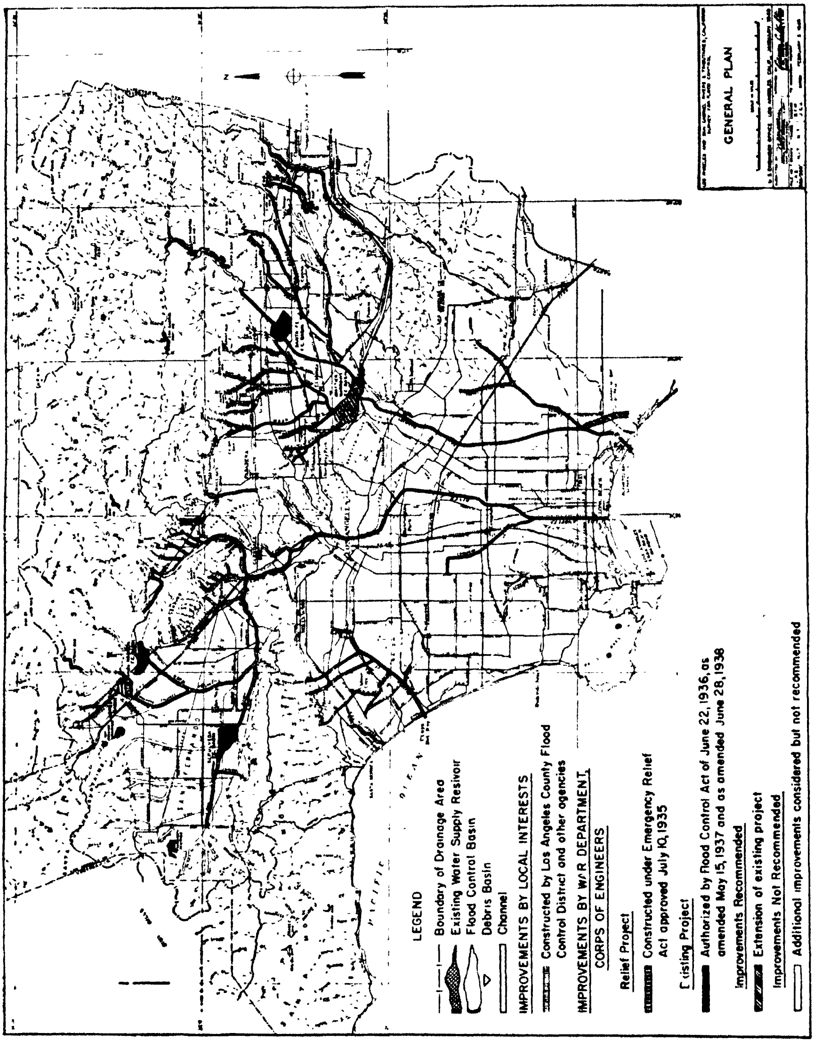


FIG. 15.5. Los Angeles flood control, general plan.

Of the above, about $215,000,000 is for construction purposes and the balance for relocations, bridges, lands, etc. The annual charges upon these works would be as follows:

Interest at 3½ per cent on total investment	$10,028,000
Amortization of obsolescence and depreciation in 50 years at 3½ per cent	2,178,000
Operation and maintenance of structures	1,331,000
Total annual carrying charges	$13,537,000

The estimated frequency of floods is assumed as follows:

Character of Flood	Number of Occurrences in 100 Years
Great floods, approaching the probable maximum, covering the entire flood plain in the valleys and extensive areas in the Coastal Plain; causing great damage to property and considerable loss of life	1
Large floods from general storms, inundating all low-lying valley areas and sections of the Coastal Plain adjacent to the stream channels, causing heavy damage and possible loss of life	6
Moderate floods, causing overflow and damage to marginal land along main streams and principal tributaries	10

The estimated damage from future floods for a 50-year period is as follows:

Direct	$586,200,000
Indirect	351,800,000
Total	$938,000,000

This gives an average yearly total damage of $18,758,000.

Additional benefit due to the removal of flood menace from alternate possible paths of overflow was estimated at $38,165,000, which at 5 per cent per annum would give an added annual benefit of $1,908,000, resulting in a total yearly benefit of $20,666,000, or a ratio of total benefits to total costs for the whole plan of 1.52.

In 1941, the Secretary of Agriculture, as authorized by the Flood Control Act of June 22, 1936 (49 Stat. 1570), recommended additional flood-control measures, including fire control, cover improvement, road improvement, mountain-channel improvements, farm-land improvement, treatment and debris basins, and improvement of river channels. This would require a Federal appropriation of $8,379,635, with state and local cooperation to the extent of $3,036,534, or a total of about $11,416,000.

Summarizing, the estimated cost of all flood-control work is as follows:

Expended up to January 1, 1940	$203,000,000
Included in existing project (Federal)	37,500,000
Total spent to 1940, not including existing project	165,500,000
Extimated cost of existing project (Federal)	82,541,000
Recommended extension (1940)	185,623,000
Total, expended and estimated	$433,664,000

Including the estimated cost of work recommended by the Secretary of Agriculture in 1941, of $11,416,000, the grand total estimated cost of flood-control work is about $445,000,000. Of this amount, about $11,000,000 is for maintenance, leaving about $434,000,000 for initial cost. Of this total, about 55 per cent is from Federal Funds. Of the total of $263,000,000 required in 1940 to complete the works, about 80 per cent is to be from Federal funds.

The grand total cost per square mile of drainage area (1,717 sq miles) is about $258,000, which is about $400 per acre. These areal unit costs are apparently among the highest amounts yet planned for expenditure in this country for flood control. Including expenditures made prior to 1940, on the basis of a total cost of $434,000,000, yearly costs would be about $19,-500,000, resulting in a ratio of total benefits to costs of 1.05, reflecting still further the difficulties and high cost of flood control in the Los Angeles area.

Character of Works. The character of flood-control work in the Los Angeles area is indicated in the following descriptions of units of the general project that appear to be typical.[1]

Hansen Dam. This unit is located on Tujunga Wash approximately 9.4 miles above its junction with the Los Angeles River. It includes a rolled-fill earth dam 9,050 ft long, maximum height 97 ft above stream bed, with a flood-control basin of 33,120 acre-ft capacity at spillway crest; an uncontrolled spillway 284 ft in length, located near the center of the dam where bedrock is available; and outlet works through the dam near the center of the structure.

The operation of the basin regulates the runoff from a tributary area of 147.4 sq miles, reducing the flow of a design flood from 64,800 to 12,000 cfs. This unit was completed in 1941. Its estimated cost is $11,223,000.

Sepulveda Dam. This unit is located on the upper Los Angeles River, approximately 43.1 miles above the mouth. It includes a rolled-fill earth dam 15,300 ft long, maximum height 57 ft above stream bed, with a flood-control basin of 16,700 acre-ft capacity at top of spillway crest gates; spillway 399 ft in length, located near the center of the dam; and outlet works through the dam near the center of the structure. The operation

[1] *Report of Chief of Engineers*, 1944, Vol. 2, Part I, pp. 1514–1518.

of the basin regulates the runoff from a tributary area of 155.3 sq miles, reducing the flow of a design flood from 69,000 to 7,100 cfs. It was completed in 1941. Its estimated cost is $6,757,000.

Los Angeles River Channel, Niagara St. to Stewart and Gray Road. The plan provides for channel improvements along the Los Angeles River as follows: A rectangular concrete channel from Niagara St. to the Burbank Western Wash (1.7 miles); a trapezoidal channel with concrete-slab or grouted-rock protection from the Burbank Western Wash to 1,400 ft above Dayton Ave. (7.5 miles); a channel with concrete invert and either sloping concrete side walls or vertical sheet-steel-pile or concrete retaining walls, depending on rights of way available and the proximity of main-line railroads on each bank, from 1,400 ft above Dayton Ave. to Soto St. (5.4 miles); a trapezoidal channel with concrete-slab or grouted-rock bank protection from Soto St. to Stewart and Gray Road (6.2 miles); and four highway bridges, four railroad bridges, and one footbridge. The unit will provide protection for the city of Los Angeles. The channel is designed for the probable maximum flood. Its estimated cost is $36,864,000.

Haines Canyon. This unit consists of the extension of the existing rectangular reinforced-concrete channel of Haines Canyon from Plainview Ave. to the mouth of Haines Canyon (a distance of 1.8 miles), thus completing a channel 3.6 miles long from Tujunga Wash to the existing debris basin. The unit also includes reconstruction of the debris basin by the construction of a reinforced-concrete spillway and outlet structure, the enlargement of the earth-fill embankment, and excavation in the basin. The completed portions of improvements were initiated for construction under Emergency Relief projects. The project intercepts the runoff from 7 sq miles of mountainous terrain and will protect the cities of Tujunga and Sunland. The unit is designed for the probable maximum flood. Its estimated cost is $626,000.

Sycamore Wash. This unit consists of 384 ft of covered reinforced-concrete conduit at the junction of Sycamore Wash with the Los Angeles River between San Fernando Road and the Southern Pacific Railroad, to provide an outlet for the existing improved channel through the city of Glendale. The unit, including 2,925 ft of improved channel completed under Emergency Relief projects, will receive the runoff from an area of 7.7 sq miles in the south portion of the city of Glendale. The channel is designed for 35 per cent of the probable maximum flood. Local interests are contemplating construction of a relief by-pass channel, when required by future development. Its estimated cost is $23,500.

Sante Fe Dam. This unit is located on the San Gabriel River, approximately 29 miles above its mouth. The plan provides for a rolled-fill earth dam 24,100 ft long, maximum height 92 ft above stream bed, with a flood-control basin of 33,000 acre-ft capacity at spillway crest; an uncontrolled spillway 1,200 ft in length, located near the right abutment; and outlet works through the dam near the center of the structure. The operation of the basin will regulate the runoff from a tributary area of 231 sq miles, reducing the flow of a design flood from 81,600 to 19,000 cfs. This unit is nearly completed. Its estimated cost is $11,848,000.

Ballona Creek Channel and Jetties. This unit includes channel improvements along Ballona Creek, consisting of parts of the following: 9,273 ft of rectangular reinforced-concrete channel between Redondo and Washington Boulevards, 8,800 ft of rectangular concrete conduit with trapezoidal high-water section between Washington St. and La Salle St., 29,175 ft of trapezoidal rock-paved section between La Salle St. and the ocean; two heavy rock jetties extending into the ocean for approximately 800 ft to ensure a free channel at the outlet; 14 highway bridges, 3 railroad bridges, the substructure for a future highway bridge; a temporary highway bridge and a temporary railroad bridge; and large regulating gates at the mouth of Playa del Rey Lagoon, to allow free tidal action within the lagoon. The unit provides drainage for 130 sq miles of area in West Los Angeles and, when side drainage channels are provided, will control floods in West Los Angeles, Venice, Culver City, and Beverly Hills. The channel is designed for the probable maximum flood. Its estimated cost is $1,578,000.

Three of the foregoing dams and reservoirs have the following features:

Dam and reservoir	Drainage area, sq miles	Storage capacity			Cost		
		Acre-ft total	Acre-ft per sq in	In.	Total	Per acre-ft	Per sq mile
Hansen	147	33,100	224	4.2	$11,223,000	$340	$76,000
Sepulveda	155	16,700	107	2.0	6,757,000	405	43,500
Santa Fe	231	33,000	142	2.7	11,848,000	360	51,500
Total or mean .	533	82,800	155	2.9	$29,828,000	$360	$55,500

Owing to the difficult construction conditions, these reservoirs are very costly, as may be seen by comparison with reservoir costs per acre-foot in other sections of the country, in Table 9·3. The costs per square mile of controlled drainage area, averaging $55,000 for these three reservoirs, is also very high (see Table 15·2, page 313) and far in excess of usual costs.

CHAPTER 16

UNITED STATES BUREAU OF RECLAMATION PROJECTS—DEPARTMENT OF INTERIOR[1]

The U.S. Bureau of Reclamation of the Department of Interior has been of importance in planning and constructing many large irrigation, power, and incidental flood-control projects in the West. Its activities are divided into seven regions, including all the western and coast states east to and including the Dakotas, Nebraska, Kansas, Oklahoma, and Texas. Some of its more recent and important projects include

Grand Coulee Dam, Montana.
Anderson Ranch Dam, Idaho.
Deschutes project, Oregon.
Central Valley project, including Shasta and Friant Dams, California.
Hoover Dam, Nevada and Arizona.
Gila project, Arizona.
Elephant Butte Dam, New Mexico.
Altus project, Oklahoma.
Tucumcare project, New Mexico.

The total investment of the Federal government in projects constructed by the Bureau of Reclamation was about $953,000,000 on June 30, 1945.

Division of Power—Department of Interior.[2] This division supervises and coordinates the power activities of the department. Its responsibilities have been enlarged by the Flood Control Act of 1944, which directed the Secretary of the Interior to transmit and market all excess power generated at reservoir projects constructed by the War Department.

The generating plants from which the department markets power delivered 18 billion kwhr of salable power in 1945 from a total installed capacity of about 3 million kw. Nearly all the power came from the three major power agencies of the department, *viz.*, the Bonneville Power Administration, the Bureau of Reclamation, and the Southwestern Power Administration. It totaled 17.82 billion kwhr at a given revenue of $45,500,000, or about 0.25 cent per kilowatt-hour.

The Southwestern Power Administration was created by the department in 1913, during the First World War, for operation, marketing, and control

[1] *Annual Report of the Secretary of the Interior*, 1945.
[2] *Ibid.*, pp. 41–44.

of the Grand River Dam, and the marketing of power generated at the Norfork and Denison Dams.

Hoover Dam.[1] The development of the Hoover Dam project upon the Colorado River, at the Black Canyon site between Nevada and Arizona near Las Vegas, Nev., for power, irrigation, and flood control by the U.S. Bureau of Reclamation was authorized in 1930, and power was generated, beginning in 1936. The cost of the project to June 30, 1945, was about $142,000,000.

Colorado River. The total drainage area is about 245,000 sq miles, including parts of the seven states of Arizona, California, Colorado, New Mexico, Nevada, Utah, and Wyoming. The drainage area above the dam is 167,000 sq miles. The average yearly runoff is estimated at 15 million acre-ft (or about 21,000 cfs, or 0.12 csm, or 1.7 in. yearly), varying from a maximum of 25 million acre-ft in 1909 to a minimum of about 4 million acre-ft in 1934.

Storage Capacity. Lake Mead, created by Hoover Dam, has a maximum storage capacity of 30.5 million acre-ft of water (or about 2 years average flow) with a water area of 146,000 acres (229 sq miles). The lake extends 115 miles up to the lower end of the Grand Canyon, the width varying from several hundred feet in the canyons to a maximum of 8 miles.

Initially it is contemplated that 9,500,000 acre-ft of storage capacity will be reserved primarily for flood-control purposes, with incidental use for the production of secondary electrical energy. The remaining capacity of 21,000,000 acre-ft will be used for regulation of water for irrigation, for firm-power production, and for dead storage for creating power head and a silt pocket.

Future irrigation and storage development upstream from the dam will decrease the inflow and permit a reduction in the storage capacity reserved for flood-control purposes, with a corresponding increase in the live storage capacity. Reductions in the total reservoir capacity due to silting will merely involve a reduction in dead storage. It is estimated that, in the first 50-year period of reservoir operation, about 3,000,000 acre-ft of reservoir capacity will be lost by silting.

The Dam. The dam (see Fig. 16·1) is a concrete arch-gravity structure, with water load carried by both gravity and arch action. It is 726 ft in maximum height (the highest dam in the world), 45 ft wide at the top and 660 ft at the base, and is 1,282 ft in depth along the crest. It contains 3,250,000 cu yd of concrete with 4,400,000 cu yd more in dam, power-plant, and appurtenant works.

The Spillway. This is of the side-channel type, with two identical spillways, each with a clear crest length of 400 ft, one upon each side of the

[1] "Dams and Control Works," pp. 1–27, U.S. Bureau of Reclamation. 1938.

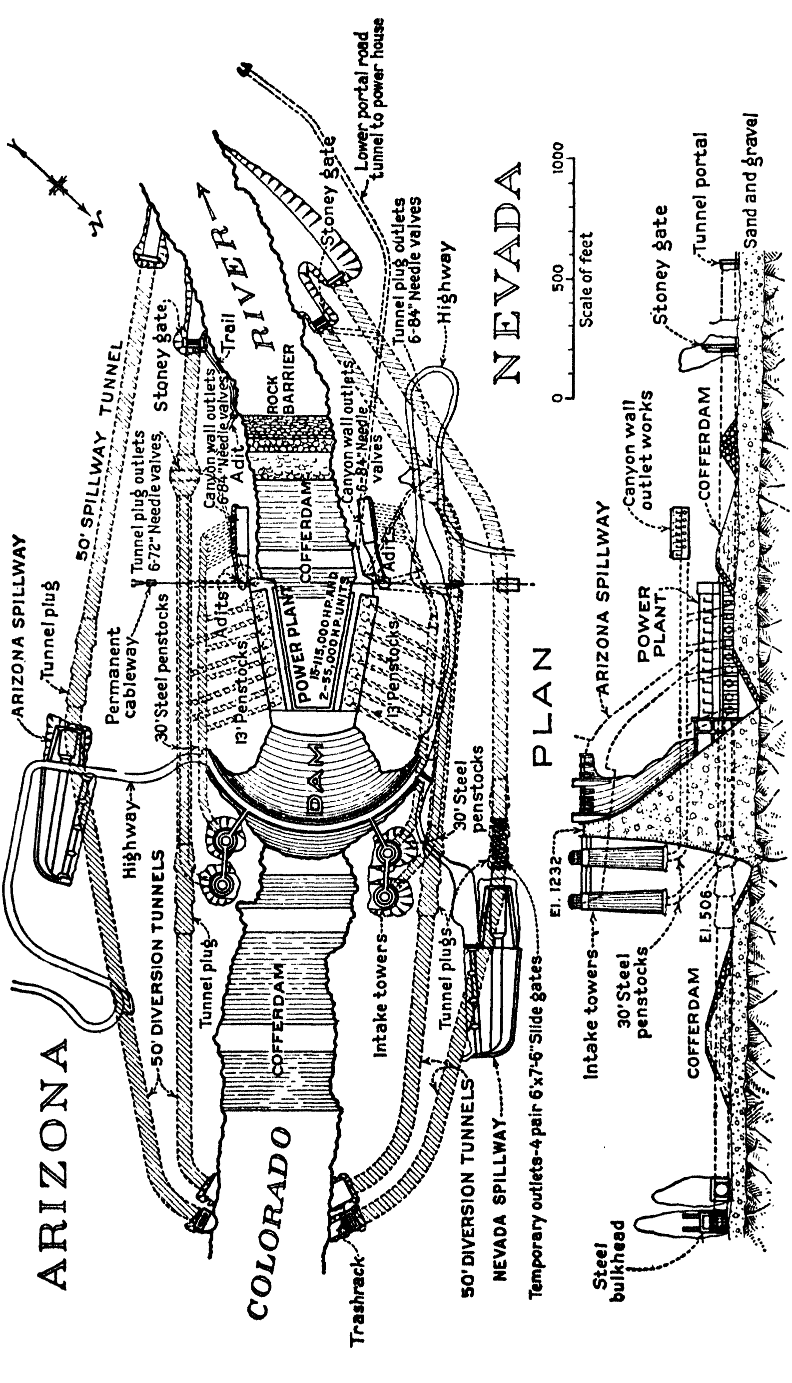


FIG. 16·1. Hoover Dam, plan and section.

canyon. These are each 650 ft long by 150 ft high and provided with four structural-steel Drum gates each 100 ft long and 16 ft high. Each gate weighs about 250 tons. In the lowered position, the top face of the gates will provide a curved crest surface; they are hinged at the top and upstream side of the concrete crest and will operate automatically with a maximum depth of about 24 ft over the fixed crest. The gates and controls are similar to those as described for Norris Dam (page 90). In addition to automatic operation, the gates can be operated manually so as to release water gradually when desired.

Four intake towers, two upon either side of the canyon, each about 30 ft in diameter and 342 ft high, control flow by means of large cylinder gates, including the flow to the canyon-wall outlet works, the tunnel-plug outlet works, and the power-plant turbines (see Fig. 16·1).

Power Plant. This is designed for an ultimate installation of 15 main units of 115,000-hp capacity each and 2 units of 55,000-hp capacity each, or a total of 1,835,000 hp. The initial installation as of Jan. 30, 1945, was 1,035,000 hp with 7 main units.

Operation of Lake Mead at Hoover Dam.[1] The act providing for the Hoover Canyon project requires that the reservoir be operated for five major purposes—river regulation and flood control, irrigation, silt control, power development and domestic water supply. It provides for amortizing the project through the sale of power and water, of which the former is expected to be much the larger item. This means that it is highly desirable to secure the greatest output of power without important loss to either flood control or irrigation. The lake should therefore be held as high as practicable, at the same time that adequate flood space is retained on top of the power and irrigation storage, for use in spring and early summer.

Lake Mead has a capacity of about 2½ times the average annual flow of 11,900,000 acre-ft, of which 9,500,000 has been reserved for flood control. This, together with the total safe release of some 2,500,000 acre-ft during the flood period, is considered ample to take care of flood runoff. For the most efficient use of the flood-storage space, forecasts of inflows are necessary. This is a difficult task in a basin of 170,000 sq miles, where elevations range from 1,200 to 12,000 ft and snowfall, rain, temperature, and wind conditions change frequently.

The operation of Lake Mead to date has been largely the initial filling of the reservoir. The flood-storage space is above El. 1,155, at which point the storage volume is 21,000,000 acre-ft. A sharp drawdown between January and March, 1939, was made to provide adequate storage space in the face of anticipated heavy flood runoff in the spring and early summer. The runoff, however, did not measure up to expectations, and the reservoir

[1] BOWDEN: Multipurpose Reservoir Operation, *Civil Eng.*, May, 1941, p. 293.

lacked some 23 ft of reaching spillway crest, El. 1,205.4, before drawdown again began in July.

It is understood that the reservoir water surface will be varied as necessary between about El. 1,155 and the spillway crest, El. 1,205.4, or even to the top of the spillway crest gates, El. 1,221.4, if necessary. Ultimate high-water surface is at El. 1,229, about 3 ft below the dam crest. Ordinarily refilling will take place from about Mar. 1 to July 1, when flood flows are being regulated, and drawdown to El. 1,155 will be made during the remaining eight months of the year.

During 1936, three power units were put in operation. By the end of 1939, there were nine units, two having been added each year, with the result that revenues increased from about $59,000 in 1936 to nearly $3,500,000 in 1939. Beginning with 1940, annual revenues exceeding $7,000,000 are expected.

Lake Mead regulates the Colorado River so as to provide a dependable all-year water supply for irrigation in the lower basin. The flow no longer varies between such wide limits as 3,000 to 200,000 cfs but is controlled between about 8,000 and 45,000, thus providing ample water for diversion at all times and decreasing the cost of operation and maintenance of flood-protection works from over $500,000 annually to about $50,000. Ultimately about 1,000,000 acre-ft of water will be diverted annually for domestic supplies for 13 cities, including Los Angeles, and this will produce additional revenues toward repaying the cost of Hoover Dam.

Grand Coulee Dam and Columbia Basin Reclamation Project.[1] The objectives of this project constructed by the U.S. Bureau of Reclamation are to irrigate in the future (the next 25 to 50 years) a million and a quarter acres of dry farms, abandoned land, and desert; to develop electrical energy for use on the project and elsewhere; and to regulate the flow of the upper Columbia for the benefit of present and future downstream power plants. Flood control is a minor consideration, but regulation of winter flow is important to navigation on the lower Columbia, where the project will add 2 ft to navigable depth.

Columbia River Basin. Second in flow to the Mississippi, the Columbia River alone drains an area of 259,000 sq miles and with its tributaries is greater than any other river in the United States in potential power. At the site of Grand Coulee Dam, the drainage area is 74,100 sq miles, of which 39,000 sq miles lies in Canada. The average flow at the dam site during the past 25 years was 110,000 cfs, or about 1.5 csm, or about 20 in. yearly upon the drainage area. The range is from 17,000 cfs (0.23 csm) to 492,000 cfs (6.6 csm).

[1] HUTTON, S. E.: Regional Director of Information, U.S. Bureau of Information, Northwest Section A.S.C.E., Nov. 6, 1939.

General Scheme of Project. The proposed scheme, approved in 1932 by the U.S. Engineers, after comprehensive study of the Columbia Basin, includes a dam, powerhouse, and pumping plant on the river near the mouth of the Grand Coulee and the use of the latter as the route to the irrigable lands.

In brief, the project involves building, in an isolated 1,600-ft canyon, a dam of unprecedented volume to raise the river above it about 335 ft; and,

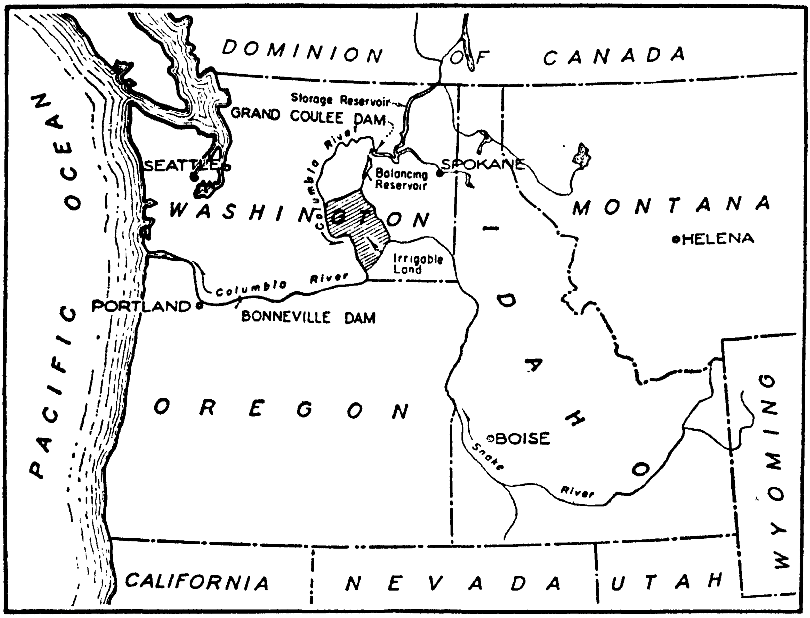


FIG. 16·2. Grand Coulee project, location plan.

as the project develops, the construction of the largest power plant in the world, a pumping plant of 12 units ultimately to raise irrigating water an additional 280 to 365 ft at the total rate of 16,000 cfs, a reservoir 27 miles long in the upper Grand Coulee, hundreds of miles of main and lateral canals, numerous auxiliary power and pumping plants on the project lands, and a variety of structures and control works.

The Columbia River falls 1,290 ft in its 750-mile course from the Canadian border to the sea. According to plans of the Army Engineers, 10 dams will someday utilize 92 per cent of this head. Two of the dams are already completed, one at Bonneville, at the head of tidewater, 146 miles from the sea, and one at Rock Island, 150 miles below the Coulee Dam, which is 600 miles from the sea. Uppermost and most important of the 10 will be the Grand Coulee Dam. It will utilize 27 per cent of the total

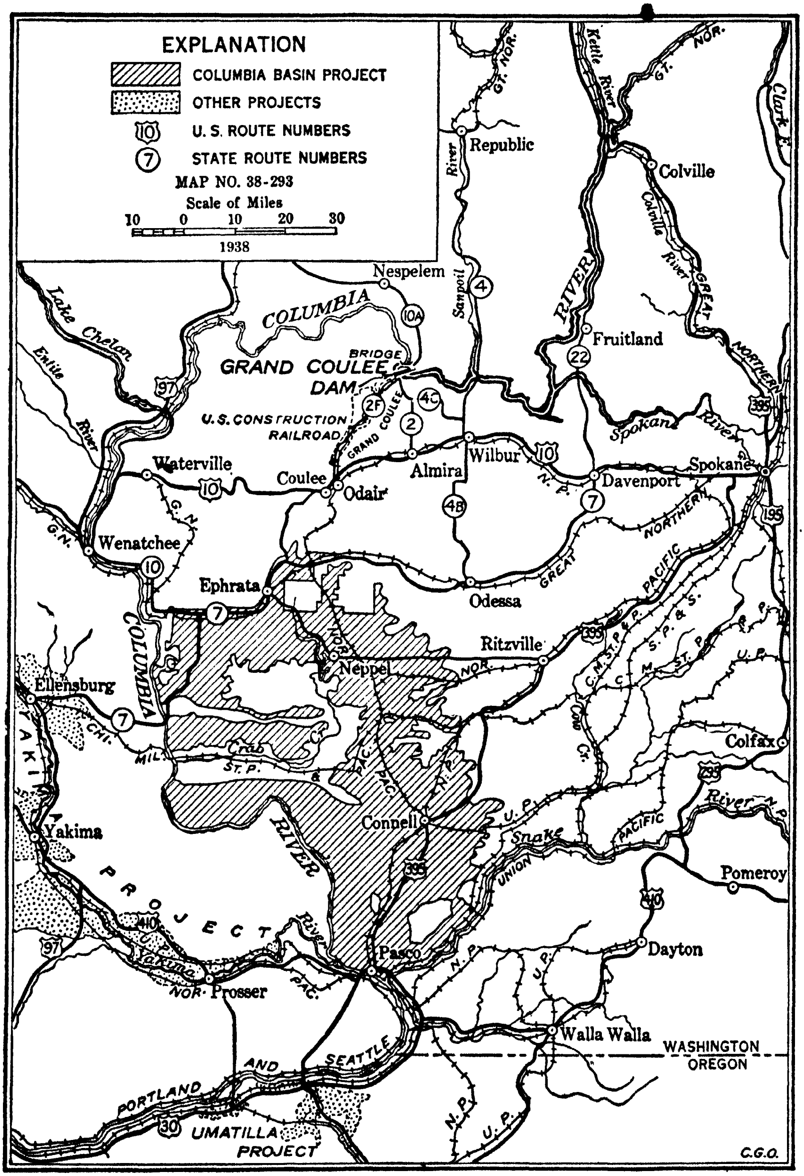


Fig. 16·3. Grand Coulee project, general plan.

head and will regulate the flow of the river for the benefit of power plants at other dams, doubling the firm-power capacities of such plants above the junction of the Columbia with the Snake River, and increasing by 50 per cent the firm-power capacities of others. The general location of the project is shown in Fig. 16·2; a detailed plan in Fig. 16·3; and a diagrammatic view of the dam, power plant, pumping plant, and balancing reservoir in Fig. 16·4.

Grand Coulee Dam. This dam is of concrete of gravity section, as shown in Fig. 16·4. Its maximum height from bedrock is 550 ft, and it is 4,170 ft long. The base is 500 ft wide at the bottom of the canyon and about 3,000 ft long. The spillway section is 1,650 ft long, at El. 1,260, terminating in a bucket 90 ft wide and 30 ft deep with crest at El. 900, about 33 ft below average low-water level and nearly 80 ft below average high-water level.

Control of river flow and of storage will be accomplished by means of 11 Drum gates at the crest of the spillway, each 135 ft long and 28 ft high, and 60 outlet conduits through the dam. The conduits are 8½ ft in diameter and are arranged in three sets of 10 pairs, one set at approximately low-water level, one set 100 ft higher, and one set 200 ft above low water. The estimated capacity of the spillway is 800,000 sec-ft, and that of the outlet conduits 195,000 to 315,000 sec-ft. The turbines, fully loaded, will pass 81,000 sec-ft.

Trash racks protect the entrances to the outlet works from floating debris, and two gate valves are installed near the upstream end of each conduit. Air ports will admit air to the conduits when the valves are closed, to prevent the creation of vacuum in the conduits, and consequent damage to linings and valves. Entrances to the conduits are lined with heavy, ribbed, semisteel castings, flared to reduce or eliminate cavitation.

At vertical intervals of 50 ft, galleries extend from end to end of the dam. These, with a number of adits extending from them toward the face of the dam, are provided for use in inspecting, cooling, and grouting the dam. One cross-channel gallery closely follows the bedrock. From it, holes will be drilled for final bedrock grouting, when the dam is nearly completed; and, later, 50-ft open uplift relief holes will be put down.

Storage Reservoir. The reservoir formed by the dam will extend 151 miles to the Canadian border and will average 4,000 ft in width, and have a water area of 82,000 acres. The storage capacity will be about 10 million acre-ft, of which a little more than 5 million will be useful for regulating purposes with an 80-ft drawdown. No stored water will be needed for irrigation or for pumping irrigating water, since the season of high water coincides with the growing season; hence the entire useful storage capacity of the reservoir will be available for the production of power at the dam and at downstream power plants.

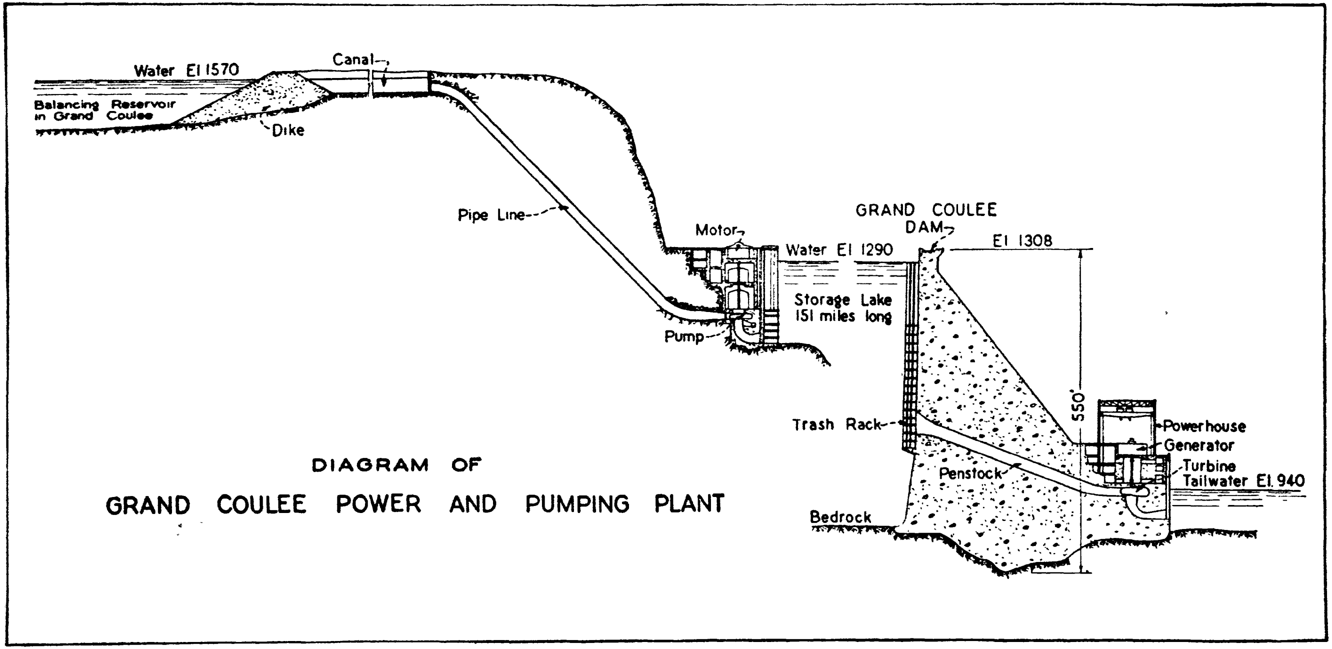


FIG. 16·4. Grand Coulee project, diagrammatic section.

The ordinary flood line will be at El. 1,288 to 1,290 above sea level, but all land below El. 1,310, an area of 128 sq miles, will be acquired by the government. It has been surveyed by the Bureau of Reclamation and evaluated for the government by local appraisers. With rare exceptions, owners have accepted the prices offered. Chiefly grazing and cutover timberland are involved. The small towns, and several small settlements, including a little orchard land, are included. Several miles of railroad and of primary and other highways will be relocated.

Power Plant. Flanking the spillway section on each side is a powerhouse and abutment section. The foundation for the right powerhouse has been completed, but no further work will be done on it until the left powerhouse is developed. The left powerhouse is a reinforced-concrete building 765 ft long and 84 ft wide, completed in 1941.

When completed, the power plant will contain 18 generating units, nine in each powerhouse, each consisting of a 108,000-kw generator, driven by a 150,000-hp turbine with a runner 16 ft 8 in. in diameter operating under a head of 330 ft. In the left powerhouse, there will also be three station-service units, each a 12,500-kva generator driven by a 14,000-hp turbine. The total present capacity (1945) is thus about 1.5 million kw, and the ultimate capacity will be nearly 2 million kw, the largest hydroelectric plant in the world.

The first three 108,000-kw generators cost $870,000 each, or $8 per kilowatt installed, and the turbines $492,000 each, or $3.30 per horsepower without installation cost. The generators will run at 120 rpm and 13,800 volts, with rotors 10 ft high and 31 ft in diameter and stator frame 22 ft high and 45 ft in diameter. The weight of each generator will be 2 million lb, that of each turbine 1.5 million lb.

Pumping Plant. This will be located in a wing dam (see Fig. 16·4) about 150 ft high and 600 ft long located at the left end of the dam. It will include ultimately 12 centrifugal pumps, of which two will be spare units each with a capacity of 1,600 cfs against a head of 295 ft, or a total capacity of 16,000 cfs. These units are comparable in cfs with those at the Turtle Creek project at East Pittsburgh (see page 374) of 2,500 cfs, but the latter operate under a very low head of from 4 to 10 ft.

Each pump will be driven by a 65,000-hp synchronous motor. Two motors will take their power supply directly from one generator, and the two pumps will be started by controlling generator and motor field excitation, as a turbine is started. It will be possible to adjust pump speeds to pumping heads by controlling turbine speeds. The output of each pump will be delivered through a 12-ft steel pipe and one of 12 tunnels driven diagonally upward through the canyon wall, to be carried thence 1¾ miles by an open canal to the upper of the two earth-fill dams that will form a 27-mile balancing reservoir in the upper Grand Coulee.

Balancing Reservoir. Formed by two earth-fill dams in the Upper Coulee, the balancing reservoir will take the place of 27 miles of main canal in adjusting the supply and demand of irrigation water.

Irrigation. The land to be irrigated extends from the vicinity of Soap Lake, 50 miles south of the dam, to Pasco, about 80 miles farther south. Precipitation varies from 3 to 17 in. yearly, with very little during the growing season. Land is held by individuals and corporations and includes desert, abandoned dry farms, and dry-farm wheat land.

Cost of Project. The estimated cost of the dam is $119,000,000, of the power plant $67,000,000, and of the irrigation system $208,000,000. It is expected that the total required financing will not exceed $260,000,000, since income from the sale of power and repayments by landowners will be relied on to finance the greater part of the irrigation system and the later development of the power plant. Costs, including that part of the irrigation system assessed against landowners, are to be repaid with interest.

At present, it is estimated that, after water is available, the landowner will pay $2.60 per acre per year for operation and maintenance of irrigation works, including $1 per acre per year for surplus power used in pumping. During the first 4 years after water is available, it is anticipated that no collections will be made on the construction account.

The construction charge is expected to be $85 to $100 per acre. On that basis, the settler would probably pay $2.60 per acre per year for operation and maintenance and nothing on account of construction during the first 4 years, then about $4.60 per acre per year for construction, and operation and maintenance for 4 years, and finally about $5.10 per acre per year for 32 years. Thereafter, only the charge for operation and maintenance would be paid.

Power Output. The estimated annual output of firm power, available at a uniform rate throughout the year, will be 8,100,000,000 kwhr. Of the seasonal output of 4,200,000,000 kwhr, generated largely during the summer, about 2 billion will ultimately be required for pumping irrigating water from the storage reservoir above the dam to the balancing reservoir in the Grand Coulee.

Financial estimates have been based on sales of firm power at 2¼ mills at the power plant, and on a charge of $1 per acre per year for power for pumping.

Market for Power.[1] The area of distribution of power from Coulee, as assumed, includes Washington, northern Oregon, northern Idaho, and

[1] *Ibid.*

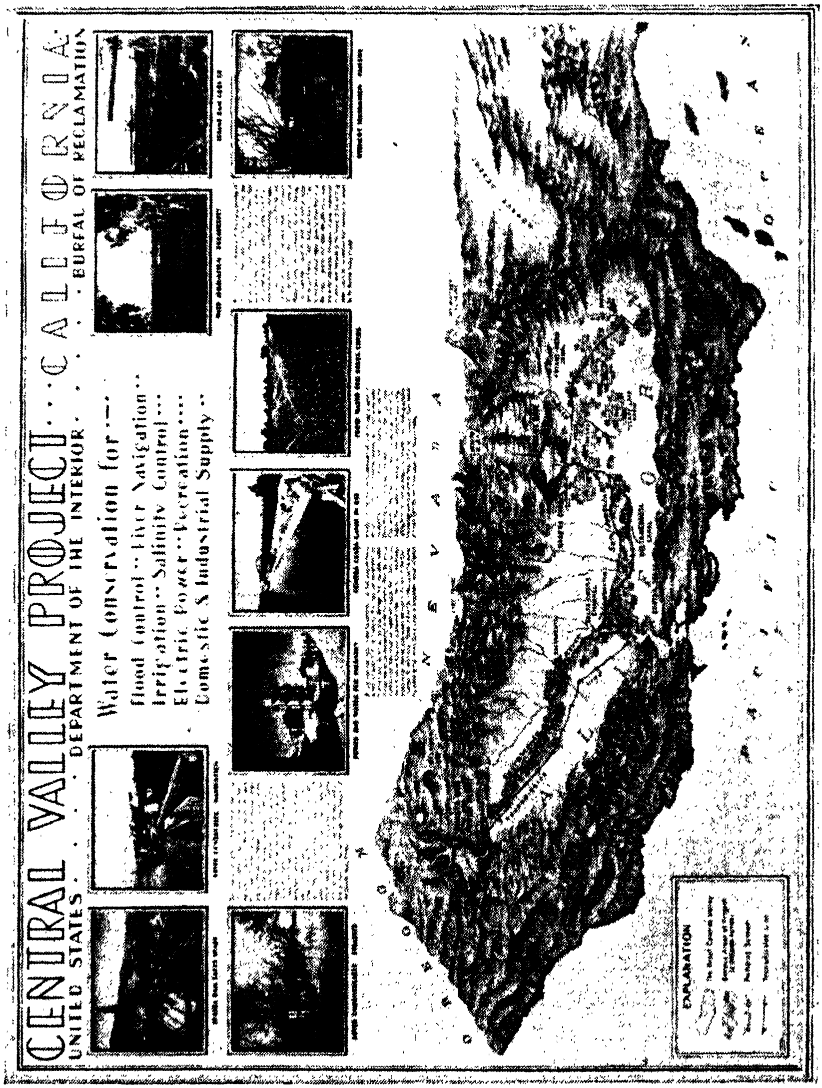


FIG. 16·5. Central Valley project, general map.

western Montana. Power requirements in this territory increased at an average rate of 9.5 per cent per year, compounded annually, between 1920 and 1930. This rate was reduced during 1920–1930 and since 1934 has exceeded the 1920–1930 rate.

Assuming that the demand will continue at a rate increase of 8 to 4 per cent in the next 30 years and allotting half the power to Grand Coulee the plant output would be absorbed in 15 years. At 2¼ mills per kilowatt-hour the cost of the dam and power plant would be liquidated in 50 years with a surplus of $144,000,000 to apply on the irrigation system. Therefore the annual surplus would be $15,000,000.

Central Valley Basin, California—Water Resources Development.[1]

The Great Central Valley.—The heart of California, the Great Central Valley is nearly 500 miles long and 50 miles wide and contains an area of over 58,000 sq miles of semiarid country, dependent upon irrigation for its development (see Fig. 16·5). The Sacramento River flowing from the north and the San Joaquin from the southwest meet in a common delta near Stockton and thence by many channels issue into San Francisco Bay.

Agricultural development in the Central Valley has outstripped the natural distribution of water. Roundly the Sacramento Basin produces two-thirds of the water, while the San Joaquin has two-thirds of the uplands. Seasonally, too, the water comes in winter and early spring, with a shortage in summer and fall. To meet these conditions, two large storage dams, one also developing power, and a 350-mile system of canals are planned by the U.S. Bureau of Reclamation.

Shasta Dam will create a larger reservoir on the Sacramento for flood control and low-water supplementation and will generate electric power to be transmitted to Antioch and the San Francisco region.

The Delta Cross Channel below Sacramento will facilitate salinity control and divert surplus Sacramento River water to the pumping plants of the Contra Costa and Delta-Mendota canals. The former will serve an industrial and agricultural region as far west as Martinez. The latter, starting at Tracy, will carry a new water supply up the San Joaquin Valley to Mendota to serve lands now irrigated from the San Joaquin River and to permit the storing of water in Friant Dam near Fresno for diversion into the Friant-Kern and Madera canals, which will, respectively, extend to the Kern River near Bakersfield and northerly toward the Chowchilla River.

The *ultimate development* is estimated to cost a total of about $1,800,000,000, as follows (including cost as noted for the 15-year program):

[1] Calland, R. S.: Acting Regional Director, U.S. Bureau of Reclamation, Memorandum, dated Sept. 5, 1945, announcing completion of report entitled "Comprehensive Plan for Water Resources Development of the Central Valley Basin, California," dated August, 1945.

1. Reservoirs, 40 or more, with over 30 billion acre-ft of storage capacity		$ 850,000,000
2. Canals and distribution systems, 6 groups		530,000,000
3*a*. Power facilities, 28 hydroelectric plants totaling about 700,000-kw capacity . .	$140,000,000	
b. Fuel electric plants 750,000-kw capacity	75,000,000	
c. Transmission facilities.	175,000,000	390,000,000
Total ultimate estimated development cost . . .		$1,770,000,000

Annual costs are estimated at about $100,000,000 (or about 5.7 per cent).
Annual benefits are estimated at $275,000,000.

The 15-year program will provide water for an increase in irrigated land of about 1,000,000 acres by 1960, to maintain normal growth, which has been 60,000 acres yearly for the past 40 years.

Initial Projects. The initial projects in the Central Valley Basin have been under way since 1938. They will be operated to regulate the water of the Sacramento and San Joaquin Rivers for better agricultural, industrial, and domestic use.

1. The *Shasta Dam*, Reservoir, and power plant are on the Sacramento River about 9 miles north of Redding, Calif.[1] The drainage area at this point is 6,665 sq miles. The dam is of concrete, 602 ft in maximum height and 3,500 ft long with central spillway; top thickness 37 ft, bottom thickness 540 ft, and containing 6 million cu yd of concrete (next in yardage to that of Grand Coulee Dam).

The reservoir area is 46 sq miles, and its capacity is 4.5 million acre-ft, or about 12.6 in. depth over the drainage area. It will check Sacramento River floods during the stormy months and release water during the dry seasons. This will maintain a minimum river flow of 5,000 cfs and irrigation as far upriver as Red Bluff, with 4 ft depth. The power plant has five 75,000-kw main units and a total capacity of 380,000 kw under a head of 330 ft with an estimated yearly output of 720 million kwhr of firm energy.

2. The *Keswick Dam*, a second development about 9 miles downstream from the Shasta plant, will provide an afterbay for the latter to maintain a uniform river flow and permit it to operate as a peak-load plant. The dam will also serve as a barrier and trap in the Sacramento River salmon-salvage program. It has a capacity of 75,000 kw, and it is estimated that Keswick will deliver 200 million kwhr of yearly firm power. The power at Shasta and Keswick will be used for irrigation pumping upon the San Joaquin–Mendota and Contra Costa irrigation canals, which require a total of about 220 million kwhr yearly so that surplus firm energy of 700

[1] The Central Valley Project, U.S. Bureau of Reclamation; also McClellan and Kilgore: Shasta and Keswick Power Developments, Central Valley Project, *Civil Eng.*, October, 1941, pp. 573–576.

million kwhr yearly will be available. An auxiliary steam plant of 150,000 kw is also planned.

The Shasta and Keswick projects are now (1945) nearly completed. Costs are as follows:

Shasta Dam and Reservoir	\$116,000,000 (about \$26 per acre-ft)
Shasta power plant	19,400,000
Keswick afterbay	16,000,000

3. The *Friant Dam* and Reservoir on the San Joaquin River near Fresno were started in 1939 and are now (1945) nearly completed. The drainage area at this point is 1,630 sq miles. The dam is of concrete 320 ft high and 3,430 ft long, top thickness 20 ft and base thickness 265 ft, with an overflow spillway in the center and four regulation conduits through the dam. The reservoir area is 7.3 sq miles, and its capacity is 520,000 acre-ft, or about 4.8 in. depth over the drainage area. The cost of Friant Reservoir is estimated at \$20,500,000, or about \$40 per acre-ft.

Ultimate Plan. The *comprehensive ultimate plan* of multiple-purpose dams, reservoirs, canals, power plants, and related facilities for irrigation, flood control, power, and other water uses is shown upon Fig. 16·5. The project is planned to make water available for

1. 2,000,000 acres of land that are now irrigated from inadequate groundwater supplies or that suffer damaging shortages in recurring cycles of dry years.
2. 360,000 acres of land in the Sacramento–San Joaquin Delta needing protection from detrimental intrusions of salt water from San Francisco Bay.
3. 3,040,000 acres of land not now irrigated.
4. Municipal and miscellaneous purposes within the basin, including use by cities, towns, industries, military areas, farmsteads, duck clubs, and game refuges to the extent of 300,000 acre-ft of water annually.
5. The generation of about 8.0 billion kwhr of electric energy annually.

In addition, the multiple-purpose reservoirs of the comprehensive plan, supplemented by certain levees, channel improvements, and related facilities proposed by the Corps of Engineers, U.S. War Department, would eliminate most of the present periodic agricultural and urban flood damage in the Central Valley.

The cost of this ultimate development would be approximately \$1,800,000,000. The monetary benefits to the nation from the plan are estimated at \$275,003,000 annually, and annual costs at \$99,730,000.

The first stage proposed is a construction program for immediate undertaking to be completed within the next 15 years. The plan is as follows:

1. To furnish the initial features of the project now under construction, and upon which about $160,000,000 has already been expended. This will require an amount of $208,000,000.
2. The remaining project items in the 15-year program are estimated to cost $527,000,000, made up as follows:

For postwar construction	$366,000,000
Other available projects for postwar development	161,000,000
Total	$527,000,000

3. The total now required will thus be $735,000,000.

CHAPTER 17

WATERSHED PROJECTS OF UNITED STATES DEPARTMENT OF AGRICULTURE (SOIL CONSERVATION SERVICE)

The various flood-control acts since and including the Act of 1936 have provided that "Federal investigations of watersheds and measures for runoff and water flow retardation and soil erosion prevention of watersheds shall be under the jurisdiction of and shall be prosecuted by the Department of Agriculture, under the direction of the Secretary of Agriculture."

Under this authority, numerous investigations of river basins have been made. Some of the important features of seven of these watershed projects, most of them in the West and South, are summarized in Table 17·1.

Columns 3 *and* 4. River basins totaling 54,280 sq miles of drainage area, with seven watershed projects, have been considered, of which about 35,000 sq miles, or about two-thirds, have been selected for remedial works and maintenance.

Column (5) In the plans, a 20- or 15-year period of installation, followed by routine maintenance and operation, is usually contemplated.

Columns (6) *to* (10). The total cost of seven projects is estimated at about \$121,000,000, or an average of about \$17,000,000 per project, allocated as follows:

	Per Cent
Installation:	
Federal	46
State or local	1
Farmers or others	30
Maintenance and operation, farmers or others	23
	100

Column (11). The cost for areas in column (4) averages about \$5.40 per acre improved, varying from \$0.48 to \$12 per acre.

Column (12). The yearly cost takes into account the time over which costs are incurred and averages \$1,800,000 for the seven projects, or about 10.7 per cent of the average project cost of \$17,000,000.

Columns (13) *to* (15). Total yearly benefits include

Flood control	\$2,797,000, or \$400,000 average per project
On site	\$32,977,000, or \$4,700,000 average per project

Thus about 92 per cent constitute on-site benefits of increased land values.

Column (16). The ratio of yearly benefits [column (15)] to yearly cost [column (12)] averages 2.82, varying between 1.71 and 3.4.

Column (17). The yearly cost per acre of land improved [column (12) ÷ column (4)] averages \$0.56, varying between \$0.21 and \$0.85.

Column (18). The yearly cost per acre after an initial period of 15 to 24 years, for which data are available for five of the projects. This cost averages \$0.28 per acre, varying from \$0.07 to \$0.50. It thus averages one-half of the initial yearly cost.

Project Description. Following is a description of the project for Little Tallahatchie watershed in Mississippi, which is fairly typical of the group in Table 17·1.

Little Tallahatchie Watershed, Mississippi.[1] The Little Tallahatchie River is one of four main tributaries of the Yazoo River system in northwest Mississippi upon which is being constructed the Sardis Dam and Reservoir. Its drainage area is 876,476 acres, or about 1,350 sq miles.

Since settlement, dating back over a century, the uplands have undergone serious erosion. Some 294,000 acres have been forced out of cultivation and about 75,000 acres virtually ruined. Failure to employ soil- and water-conserving practices on cultivated, open, and forest lands has enormously increased surface runoff, which, together with the sediment it transports, causes excessive flood damages on 65 per cent of all bottom lands. Such bad condition has been created that the runoff from ½ in. of rainfall or less, converging in sediment-choked stream channels, may cause damaging floods. Floods are exceedingly frequent; in fact, a normal year witnesses about 15 of them on some small tributaries, of which four normally occur during the growing season and cause damage amounting to an average of \$342,973 annually, principally to crops. On the average, cultivated bottom lands are annually damaged by floodwaters to an extent of about \$4.20 per acre. Sedimentation damages amount to about \$600,000 annually, and indirect flood damages are about \$238,000. Thus the total annual flood damages at present amount to approximately \$1,181,000.

The plan proposed provides for retardation of water flow and prevention of erosion through (1) a complete fire-control system for the watershed; (2) road-bank stabilization on 1,400 miles of state, county, and Forest Service roads; (3) treatment of all land suited for farming on a permanent basis by rotating crops, terracing slopes, treating critical areas, developing pasture, planting trees, improving and managing farm woodlands, and controlling gullies—this work to be done by the owner with financial assistance, material, and other services contributed by the Department of

[1] H.R. Doc. 892, 77th Cong., 2d Sess.

Agriculture; and (4) public purchase of land not suited to remain in farms and treatment of this land by planting trees, controlling gullies, and improving and managing forests on a sustained-yield basis.

The program of water-flow retardation and soil-erosion prevention is planned for a 20-year period with an expenditure as shown in the following table:

Item	Federal	State and local government	Private	Total
Fire protection on 577,125 acres	$ 80,800	. . . .	. . . .	$ 80,800
Road-bank stabilization on 1,400 miles of road	314,720	$472,080	. . . .	786,800
Treatment of cultivated, pasture, and woods in agricultural area	810,470	. . . .	$1,223,920	2,034,390
Land acquisition in nonagricultural area (162,880 acres)	1,091,000	. . . .	. . . .	1,091,300
Treatment of federally acquired land (tree planting, etc.)	959,805	. . . .	. . . .	959,805
Technical design and administration of program	580,000	. . . .	. . . .	580,000
Contingency	383,710	. . . . .	. . . .	383,710
Total	$4,220,805	$472,080	$1,223,920	$5,916,805

The total cost of operation and maintenance of the 20-year installation period will be $4,525,000, and the entire cost of installation and maintenance for the 20-year period will therefore be $10,442,000, of which about one-half will be from private funds. The average yearly cost, taking into account the fact that expenditures are made over the 20-year period, will be $464,-526. It is estimated that yearly benefits will be

Flood control	$338,000	0.28
On-site improvements	825,000	0.72
Total	$1,163,000	1.00

After the twentieth year, the yearly cost of maintenance and operation has been estimated at $439,000, of which about 90 per cent will be from private funds.

Table 17·1 U.S. Department of Agriculture Watershed Projects

River and state and reference	Date of report	Drainage area: Total, sq miles	Drainage area: Area included in program, acres	Time	Program and costs: Federal	State or local	Farmers (or others): Labor and materials	Farmers (or others): Maintenance and operation	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Trinity, Tex. H.R. Doc. 708	1942	17,600	8,260,000	1st 15 yr	$32,000,000	$ 125,000	$13,784,000	$13,148,000	$ 58,950,000
				yearly maintenance after 15 yr	23,000	8,000	1,395,000		1,426,000
Little Tallahatchie, Miss. H.R. Doc. 892	1942	1,350	867,000	1st 20 yr	$ 4,221,000	$ 472,000	$ 1,224,000	$ 4,525,000	$ 10,442,000
				yearly maintenance after 20 yr	42,000	. . . . .	. . .	397,000	439,000
Washita, Okla. and Tex. H.R. Doc. 275	1943	8,000	3,106,000	1st 15 yr	$10,929,000	. . . .	$ 3,170,000	$ 8,317,000	$ 22,416,000
				yearly maintenance after 15 yr	33,000	. . . . . .	. . .	942,000	975,000
Middle Colorado, Tex. H.R. Doc. 270	1943	7,200	2,265,000	1st 20 yr	$ 2,693,000	. .	$ 67,000	$ 206,000	$ 2,966,000
Coosa, Ga. and Tenn. H.R. Doc. 236	1943	4,020	2,054,000	1st 20 yr	$ 715,000	$ 188,000	$15,828,000	. .	$ 16,730,000
				yearly maintenance after 20 yr	8,400	1,600	132,000	. . . . . .	142,000
Little Sioux, Iowa and Minn. H.R. Doc. 638	1943	4,450	1,750,000	. . . .	$ 4,280,000	$ 368,000	$ 1,250,000	. . . .	$ 5,907,000
Potomac, Va. H.R. Doc. 269	1943	11,560	4,210,000	1st 24 yr	$ 859,000	$ 97,000	$ 1,074,000		$ 2,030,000
				yearly maintenance after 24 yr	47,000	13,000	. . . .	$1,530,000	1,590,000
Total or mean	. . . .	54,280	22,532,000	. . . .	$55,697,000 (0.46)	$1,250,000 (0.01)	$36,417,000 (0.30)	$27,720,000 (0.23)	$121,084,000 (1.00)

Table 17·1 U.S. Department of Agriculture Watershed Projects (continued)

River and state and reference	Program and costs: Per acre Col. (4) (11)	Program and costs: Yearly cost (12)	Benefits (yearly): Flood control (13)	Benefits (yearly): On site (14)	Benefits (yearly): Total yearly (15)	Ratio Col. (15) / Col. (12) (16)	Yearly cost per acre Col. (12) / Col. (4) (17)	Yearly cost per acre after initial period (18)
Trinity, Tex. H.R.Doc. 708	$ 7.50	$ 5.478,000	$1.438,000 (0.09)	$14.362,000 (0.91)	$15.800,000	2.88	$0.66	$0.17
Little Tallahatchie, Miss. H.R. Doc. 892	$12.00	$ 464.526 439,000	$ 338.000 (0.28)	$ 825,000 (0.72)	$ 1.163,000	2.5	$0.54	$0.50
Washita, Okla. and Tex. H.R. Doc. 275	$ 7.20	$ 1,469,000 975,000	$ 407,000 (0.7)	$ 5,246,000 (0.93)	$ 5,653,000	2.7	$0.47	$0.31
Middle Colorado, Tex. H.R. Doc. 270	$ 1.30	$ 482,000	$ 137,000 (0.08)	$ 1.519,000 (0.92)	$ 1,656,000	3.4	$0.21	
Coosa, Ga. and Tenn. H.R. Doc. 236	$ 8 20	$ 1.749,000 142,000	$ 73,000 (0.01)	$ 5.702,000 (0.99)	$ 5,775,000	3.3	$0.85	$0.07
Little Sioux, Iowa and Minn. H.R. Doc 638	$ 3.38	$ 1,325,000	$ 254.000 (0.09)	$ 2.628,000 (0.91)	$ 2,881,000	2.16	$0.76	
Potomac, Va. H.R. Doc. 269	$ 0.48	$1,661,000 1,590,000	$ 150,000	$ 2.695,000	$ 2,845,000	1.71	$0.39	$0.38
Total or mean	$ 5.40	$12,629,000	$2,797,000	$32.977,000	$35,773,000	2.82	$0.56	$0.28

CHAPTER 18

LOCAL FLOOD PROTECTION BY LEVEES AND RIVER WALLS

As a means of local protection from high river stages, it frequently becomes necessary, for partial or complete flood protection, to confine river waters by means of earth levees at some distance from and parallel to the stream, where space and conditions of habitation permit, or by means of masonry river walls in towns and cities where buildings and factories or other structures occupy lands near the river.

Levees. These are virtually continuous earth dams that must be high enough to withstand a maximum flood and are subject to water pressure on the river side. As with any earth dam, a proper freeboard between maximum water level and the top of the dam must be provided, so that overtopping and failure by breaching or overtopping will not occur. The term "dike," as often used, is synonymous with levee.

River Walls. Usually of concrete masonry, river walls are also virtually continuous dams and must be of adequate dimensions to carry safely the unbalanced water load during floods.

Effect of Levees and River Walls on River Regimen

With levees or walls, the river is confined to a narrower channel than in nature, and valley storage outside them is no longer available. The river cross section is lessened in area, and its velocity of flow and hydraulic slope must be increased, in order to carry the flood, which formerly spread out over adjacent lowlands and occupied a much greater width of channel The flood wave will therefore pass this section of the river more quickly and will tend to increase flood peaks downstream, below the leveed portion.

In the usual sense, therefore, levees or walls built for local protection do not contribute to flood relief at points downstream unless, as in a large river like the Lower Mississippi, levees are continuous over long distances and are used as a primary means of flood control.

Local Flood-protection Works

In local flood-protection works in flat country, it may be necessary virtually to surround a town or city with levees or walls or at least to carry them to the outlying land elevations. This is often complicated by the entrance to the river within the town limits of smaller streams or creeks requiring auxiliary levees or walls along them to keep out possible high levels backing up from the main river.

Where a town or city is thus wholly or partly walled off, provision must be made of openings in levees or walls for access by any intercepted railroad tracks and highways. These openings must be provided with temporary gate structures to be erected and closed at flood stages. This isolates the municipality or part of it during high floods and makes it necessary to provide adequate pumping facilities to handle storm water and drainage from the protected area and pump it over the levees or walls into the main river.

The temporary isolation of part or all of the municipality is unpleasant and interferes with business and other functions but is better than subjecting it to the much greater damage and interruption of service that would result without this local flood protection.

Local flood-protection works of the character described are of value only to the municipality they protect. They do not in any degree lessen flood flows, and their economic desirability is purely a local problem as to whether the relative cost of the works is justified by the resulting local saving from flood damages.

On the other hand, the use of headwater reservoirs may lessen maximum flood levels and thus be of advantage in respect to requirements for local flood-protection works or at least give them a better margin of safety. Thus, upon the Ohio River, as described elsewhere, a system of headwater reservoirs is planned for aiding flood relief on the Ohio, as well as the lower Mississippi, but in general local flood-protection works are also required and planned for many of the towns and cities along that river.

Local Flood-protection Works, Ohio River. Local flood-protection works along the Ohio River have been planned with care and furnish good examples of this kind of flood control. Project descriptions at Huntington, W. Va., Ironton and Cincinnati, Ohio, and Lawrenceburg, Ind., will now be given, followed by some details of special interest in these various projects. Condensed descriptions of projects at Paducah and Louisville, Ky., and Tell City and Jeffersonville-Clarksville, Ind., are also given.

Huntington, W. Va., Local Flood-protection Works

This project is located within the limits of the city of Huntington, W. Va. (population 78,836), on the left bank of the Ohio River, extending from about mile 304 to mile 313 below Pittsburgh, Pa.

Project Description. *Section* I. *West End of City.* Approximately 19,400 ft of earth levees and 2,200 ft of concrete wall extend downstream along the Ohio River from the western extremity of Section II of the project, a point approximately 200 ft west of Third St. West to the western limit of the city, thence turning landward to high ground at the Kanawha and

James River turnpike. Included are also six motor-operated pumping stations for the disposal of interior drainage, appurtenant drainage structures, and six gate openings.

Section II. *Business Section of City.* This includes 17,900 ft of concrete flood wall, extending downstream along the left bank of the Guyandot River from high ground immediately south of the Chesapeake & Ohio Railroad tracks to the high ground near the mouth of the river and downstream along the Ohio River from the high ground near Twenty-fifth St. to a point near Third St. West where the wall joins the levee at the eastern extremity of Section I of the project. There are also 24 gate openings and storage houses, seven motor-operated pumping stations for the disposal of interior drainage, and a continuous toe drain toward the landward side of the base of the wall.

Section III. *East End of City.* This includes 16,800 ft of concrete walls and 4,600 ft of earth levees, extending riverward from high ground near the eastern corporate limit of the city; thence downstream along the Ohio River to the mouth of the Guyandot River, and from this point upstream along that river to Sixth St. where a levee section, extending back from the Guyandot River completes the tie to high ground. There are also four electrically motor operated pumping stations for the disposal of interior drainage, appurtenant drainage structures, and 15 gate openings. In all three sections, protection is provided against floods of 1937 magnitude (maximum of record) with a 3-ft freeboard.

The project has been completed and turned over to the city for operation. Estimated cost was as follows:

	Construction	Lands and damage	Total
Section I	$2,438,000	$ 655,000	$3,093,000
Section II	2,540,000	412,000	2,952,000
Section III . . .	2,290,000	440,000	2,730,000
Total	$7,268,000	$1,507,000	$8,775,000

Floods and Flood Damages. The drainage area of the Ohio River at Huntington is 55,620 sq miles. The maximum flood of record occurred in January, 1937, with a gauge height of 69.4 at Lock 28 gauge located near the outlet of Four Pole Creek in the westerly portion of Huntington. The gauge zero is at El. 490.2, or a maximum elevation of water surface in 1937 of 559.6. At this stage, the discharge was about 654,000 cfs (or about 12 csm).

The March, 1938, flood reached about gauge height 57.1 with a discharge of 536,000 cfs (or about 10 csm), whereas that of March, 1913, reached gauge height 64.5 (discharge approximately 580,000 cfs).

In these three large floods, about one-half of the residential section and most of the business section were inundated; and, in January, 1937, all normal activities were suspended, about 40 per cent of the people in the flooded area were removed for 2 weeks, and all highway and rail traffic interrupted. Figure 18·1 is an air photograph showing flood conditions in

Fig. 18·1. Flood of January, 1937, at Huntington, W. Va., on the Ohio River.

Huntington on Jan. 26, 1937, in the business and residential sections. The Ohio River is at the left of the view; and the entrance of the Guyandot River, with a large flooded area in its valley, appears in the upper right of the picture.

Flood damages in 1937 as determined by surveys were estimated at about \$7,000,000 direct and \$11,000,000 indirect, or a total of about \$18,-000,000. The value of property affected was about \$125,000,000. The ratio of maximum flood damage to cost of protection (about \$9,000,000) was thus 2 to 1. No estimates are available as to average yearly damages.

Levees. A total of 24,000 ft of levees was constructed in the three sections of the Huntington project. A typical cross section of the levee built at Huntington at Sta. 202 + 15 near Krouts Creek is shown in Fig. 18·2. The top of levee is at El. 562.4, or 3 ft above maximum flood level at that location in 1937. In construction, the top is made higher in level by 5 per cent of the height H (Fig. 18·2) to allow for shrinkage and settle-

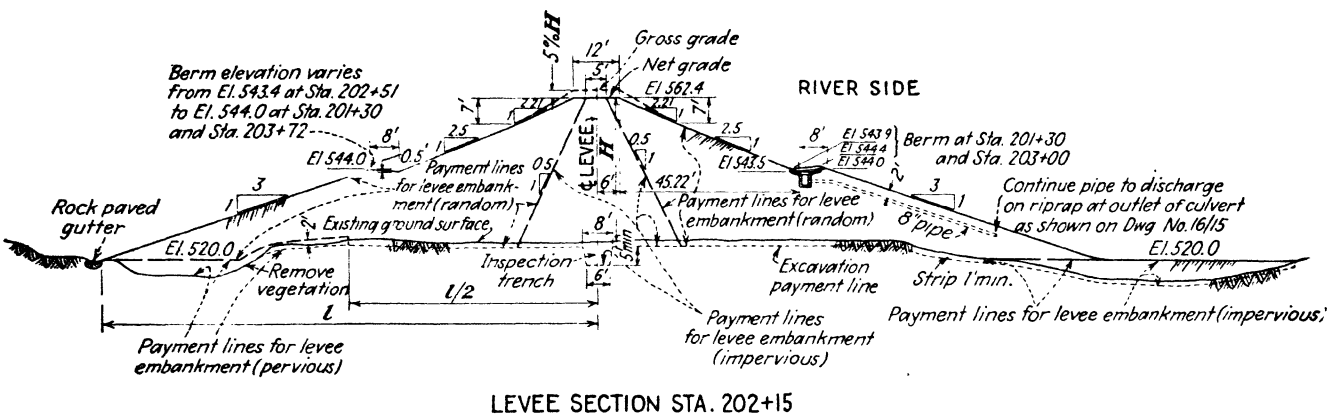


FIG. 18·2. Typical levee cross section at Huntington, W. Va., on the Ohio River.

ment. The top width is 12 ft with similar slopes on each side, varying from 1 on 2.21 near the top to 1 on 3 below El. 544.0, at which elevation a berm and drainage system is constructed on the river side. A portion of the levee is paved with a 2-ft thickness of stone riprap below El. 544.5 on the river side. The normal pool level upon the Ohio River near this point (Lock 28) is at about El. 499. The payment lines for impervious-fill and random-fill embankment are shown in Fig. 18·2.

River Walls. A total of about 37,000 ft of protective river walls was constructed in the three sections of the Huntington project, varying in height from about 12 to 24 ft. A wide-base cantilever section of reinforced-concrete wall was used, as shown in Fig. 18·3, which includes a cross section and reinforcement schedule for a 24-ft wall. The walls were all 18 in. thick at a level 12 in. below the top level with the top widened out to a 24-in. thickness with beveled corners at the top and bottom. The side batter is 1 in 15. The width of the wall base approximates the height, and a toe, or key wall, on the river end of the base approximates a depth of about 0.3 the wall height. The thickness of the base approximates the thickness of the wall at the base.

Cost of Levees and Walls. The 61,000 ft of levees and walls cost about $4,783,000, or an average of $71 per linear foot. The embankment in the levees included 1,480,000 cu yd and the concrete in the walls 95,000 cu yd.

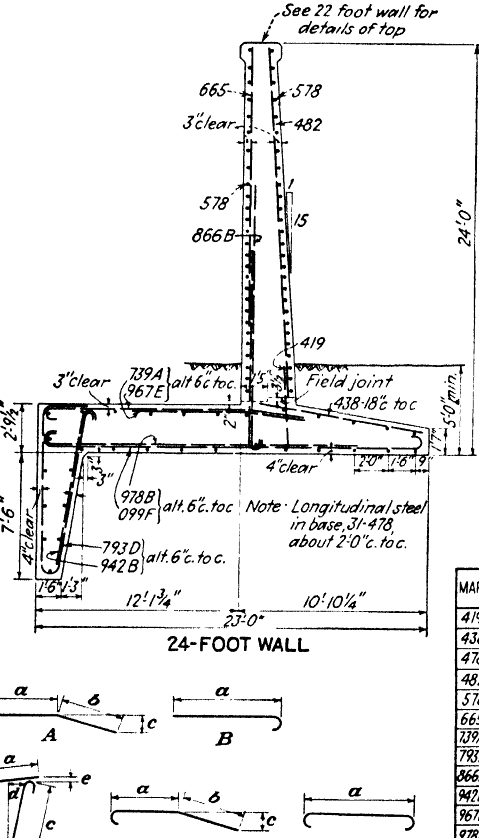
Preparation for Floods. The work generally to be done in case of flood consists of

1. Erecting gate closures.
2. Energizing substations and transmission and distribution lines.
3. Placing pumping stations in operation.
4. Patrolling walls and levees to discover any leaks or boils or other unusual disturbances.
5. Patrolling transmission and distribution lines.
6. Taking the flood works out of operation at the end of the flood period.

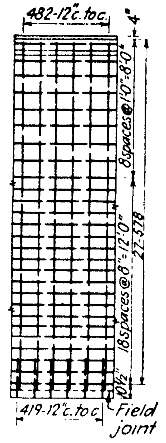
Under ordinary conditions, one man on each shift is adequate for operation at each pumping station during flood periods, with occasionally an extra man during rainstorms when frequent operation of the pumps is required.

Gate Closures. In the Huntington project, there are 45 gate openings. Essential features and locations and the character of these are given in Table 18·1, with the priority of action in each of the three districts. Included in Table 18·1 are also details of the 17 pumping stations. This table is a copy of a part of the manual of instructions used for this work.

The gate closures vary in character from sandbags and simple stop logs for 12 ft and lesser widths to timbers reinforced with I beams and for still



24-FOOT WALL

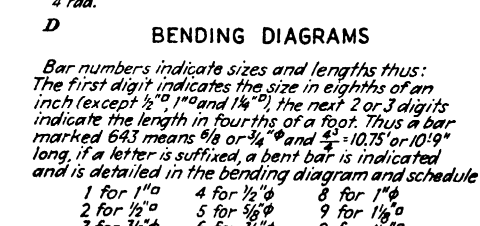


LAND FACE

24-FOOT WALL

MARK	SIZE	L'GTH	BENDING SCHEDULE a	b	c	d	e	NO.	UNIT WT.	TOTAL WT
419	½"ϕ	4'9"						20	3.17	63
438	½"ϕ	9'6"						14	6.35	89
478	½"ϕ	19'6"						31	13.03	404
482	½"ϕ	20'6"						20	13.69	274
578	⅝"ϕ	19'6"						54	20.34	1,098
665	¾"ϕ	16'3"						20	24.41	488
739A	⅞"ϕ	9'9"	6'10"	2'11"	5"			20	19.93	399
793D	⅞"ϕ	23'3"	2'10"	9'3"	9'1"	1'3"	2½"	20	47.52	951
866B	1"ϕ	16'6"	15'6"					20	44.06	881
942B	1⅛"□	10'6"	9'5"					20	45.18	904
967E	1⅛"□	16'9"	11'10"	3'10"	5"			20	72.08	1,442
978B	1⅛"□	19'6"	18'5"					20	83.91	1,678
050B	1¼"□	12'6"	11'3"					20	66.41	1,328
099F	1½"□	24'9"	22'3"					20	131.50	2,630
								Total		12,629

The steel shown in the reinforcing schedule is for one 20 foot monolith for the respective heights of wall



BENDING DIAGRAMS

Bar numbers indicate sizes and lengths thus: The first digit indicates the size in eighths of an inch (except ½"□, 1"□ and 1¼"□), the next 2 or 3 digits indicate the length in fourths of a foot. Thus a bar marked 643 means 6/8 or ¾"ϕ and $\frac{43}{4}$ = 10.75' or 10'9" long, if a letter is suffixed, a bent bar is indicated and is detailed in the bending diagram and schedule

1 for 1"□	4 for ½"ϕ	8 for 1"ϕ
2 for ½"□	5 for ⅝"ϕ	9 for 1⅛"□
3 for ⅜"ϕ	6 for ¾"ϕ	0 for 1¼"□
	7 for ⅞"ϕ	

FIG. 18·3. Twenty-four-foot wall, part of Huntington project.

larger openings by steel trestles. An assembly view of one of these trestles as used at opening No. 21 at Waverly Ave. is shown in Fig. 18·4.

Stages by gauge at which to begin closures are shown in Table18·1, varying from 49.5 at Tenth St. (opening 16) to 67.5 at the Chesapeake & Ohio Railroad (opening 1), where only sandbags are used. These closure stages also apply for going out of operation on a falling river.

The estimated time required to erect each gate closure is given and is of importance with reference to the river stages at which closing of openings

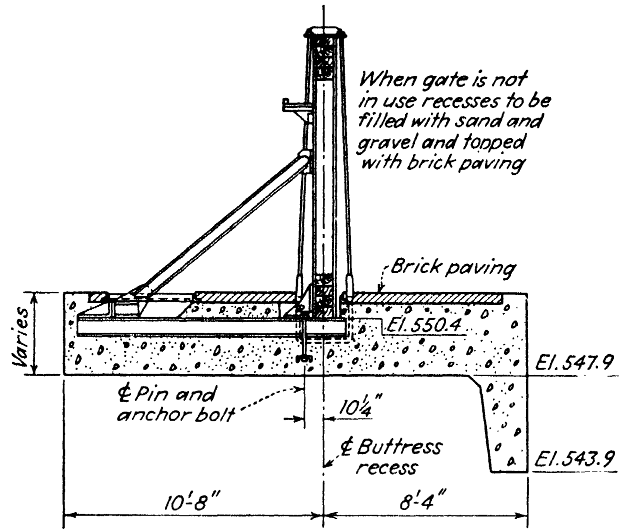


FIG. 18·4. Huntington project. Trestle used at opening No. 21, Waverly Avenue.

shall begin. In general, a 14-man crew requires 1 hr to erect a one-bay structure or one bay of a multiple-bay structure, where uncovering of sill is required. Preparing a bay for closure requires about 4 man-hours per bay (except for sandbag closures).

In Table 18·2 is a summary of requirements based upon Table 18·1 and estimated time of closure by sections. The analysis indicates 13 man-hours per bay for Section I; 21 man-hours per bay for Section II, which includes 15 openings with no trestles; and 14 man-hours per bay for Section III. The average for the three sections is 15.7 man-hours per bay, which is fairly consistent with the previous statement of 14 man-hours required.

Pumping Stations. In Table 18·3 are given contributing areas and capacities of pumps as well as costs of stations and equipment. In general, contributing areas vary from about 70 to 1,200 acres, with the exception of the Four Pole Creek station, which has 14,000 acres. The average contributing area for the 17 stations is 1,150 acres, but, omitting the large Four Pole Creek station, is 244 acres.

Capacities vary from about 60 to 1,200 cfs, averaging 184 cfs for all 17 stations and 120 cfs, omitting the Four Pole Creek. Capacity in cfs per acre and csm varies with the size of contributing areas and is shown in Fig. 18·5, which is a semilogarithmic plot of pump capacity in csm against contributing areas in acres. Three curves of maximum flows in csm are also given for comparison based upon the Meyer formula $Q = C\sqrt{M}$, where M is the drainage area in square miles (auxiliary vertical scale in Fig. 18·5) and values of C are 100, 500, and 1,000, respectively.

Costs of pumping stations as given in Table 18·3 vary from about $48,000 for the smallest at Ninth St. to $771,000 for the large Four Pole

Table 18·1 Huntington, W. Va., Local Protection Project—Pumping Stations and Gate Openings

Section I—West End

Pumping Stations

Priority	Location	Pumping equipment				Stage for starting*	Stage of max. allowable sump level
		Sewage pumps		Storm pumps			
		No.	Gpm	No.	Gpm		
2	13th St., W.	1	1,500	2	25,000	37.5	39.3
3	Four Pole Creek . .	. .	. . . . .	3	127,000	39.8	49.8
4	Krouts Creek . . .	. .	. . . . .	2	17,500	40.0	51.6
5	22d St., W.	1	1,500	2	17,000	43.5	44.6
7	East Road.	1	1,500	2	20,000	44.0	48.9
12	7th St., W.	1	1,500	2	25,000	40.0	43.1

Gate Openings

Priority	Opening Number	Location	No. of trestles	Sill el.	Stage to begin closure*
5	19	Wood Mosaic Co.	None	544.20	51.5
25	22	B. & O. R. R. (West) . . .	1	549.40	58.0
30	21	Waverly Road	6	550.40	59.0
31	23	U.S. Route 60.	3	550.40	59.0
34	20	Sunset Drive	1	551.15	59.5
40	24	C. & O. R. R. (west). . . .	3 posts	554.40	63.0

Section II—Business District

Pumping Stations

Priority	Location	Pumping equipment				Stage for starting*	Stage of max. allowable sump level
		Sewage pumps		Storm pumps			
		No.	Gpm	No.	Gpm		
6	5th Ave.	1	2,500	3	42,000	44.0	44.5
8	20th St.	1	2,500	2	29,000	44.2	44.9
9	16th St.	1	2,500	3	33,000	44.4	44.6
10	11th St.	1	2,500	2	17,500	44.6	45.8
11	4th St.	1	2,500	3	24,000	44.8	44.9
16	1st St.	1	1,500	2	17,500	47.0	48.0
17	9th St.	1	2,500	2	12,000	48.0	49.9

Gate openings

Priority	Opening Number	Location	No. of trestles	Sill el.	Stage to begin closure*
1	16	10th St.	4	543 13	49.5
2	15C	Huntington Boiler Works	4	544.23	51.0
7	15	Coughenour Boiler Works	None	545.70	52.0
8	12	West Virginia Rail Co.	1	545.98	52.0
11	17	3d St. at R. R.	1	546.58	53.0
13	18	1st St. at R. R.	1	546.58	53.5
14	14	Island Creek Coal Co.	None	547.20	53.5
15	12A	Island Creek Coal Co.	None	547.30	53.5
19	11	Wilson Sand & Gravel Co.	None	549.00	55.0
20	16A	C. & O. yards	None	548.60	55.0
22	15A	Greene Terminal Co.	None	550.00	56.5
23	15B	Greene Terminal Co.	None	550.00	56.5
26	9	Union Sand & Gravel Co.	None	552.10	58.0
27	10	20th St.	None	552.10	58.0
28	5	5th Ave.	2	553.33	58.5
29	2	31st St.	None	553.50	58.5
32	3	B. & O. R. R.	None	553.75	59.0
33	4	Frasher Boat Yard	None	554.00	59.0
36	6	3d Ave.	2	555.08	60.0
37	13	Island Creek conveyor	None	554.06	60.5
38	7	A St.	None	556.10	62.0
39	8	Water works	None	556.10	62.0
42	8A	Water works	2-stanchion	557.90	64.0
45	1	C. & O. R. R. (east)	Sandbags	563.2±	67.5

Section III—Guyandot

Pumping Stations

Priority	Location	Pumping equipment				Stage for starting*	Stage of max. allowable sump level
		Sewage pumps		Storm pumps			
		No.	Gpm	No.	Gpm		
1	Pats Branch	.	.....	2	30,000	37.4	47.4
13	35th St.	1	1,500	2	18,000	45.4	47.4
14	Oak St.	1	1,500	2	18,500	45.5	47.5
15	Richmond St.	1	1,500	2	12,500	45.5	47.5

Gate Openings

Priority	Opening Number	Location	No. of trestles	Sill el.	Stage to begin closure*
3	12	3d Ave.	4	545.83	51.0
4	6	Roby Road	4	546.10	51.5
6	13	Buffington St.	2	546.75	52.0
9	8	7th St.	3	547.63	53.0
10	11	5th Ave.	4	547.83	53.0
12	5	International Nickel Co.	4	548.18	53.5
16	14	W. Va. State Route 2	6	548.75	53.5
17	4	Riverside Drive	4	549.83	55.0
18	15	B. & O. R. R.	1	550.25	55.0
21	2	Altizer Ave.	3	551.03	56.0
24	10	Main St.	4	551.50	57.0
35	9	B. & O. R. R. near Main St.	1	554.75	60.0
41	1	C. & O. R. R. at 6th St.	Sandbags	558.20	63.6
43	3	5th St.	Sandbags	560.35	65.5
44	7	C. & O. R. R. near Hite St.	Sandbags	560.90	66.0

NOTES: River stages are readings on upper gauge at lock 28 Ohio River, with zero at El. 490.2.
GPM for pumps is design delivery of each at maximum head.
* Also stage for taking out of operation on a falling river.

Creek station. They average about $700 per cfs of capacity and $430 per acre of contributing area. Plans and sections of the Four Pole Creek pumping station are shown in Fig. 18·6.

Ironton, Ohio, Local Flood-protection Project

This is located on the right bank of the Ohio River 327 miles below Pittsburgh and 143 miles above Cincinnati. It is in the Cincinnati district.

Project Description. The area it protects includes the main part of the city of Ironton, population 16,621 in 1930, a residential and industrial

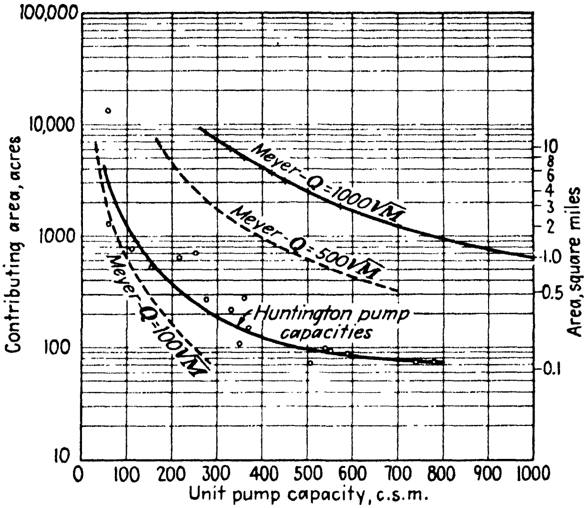


FIG. 18·5. Huntington project, pump capacities.

city (see Fig. 18·7). The city lies on a comparatively high flood plain about 5,000 ft wide at the upstream limits of the city, narrowing to about one-half that at the downstream limit. The average elevation is about 545 ft, but

Table 18·2 Summary of Time for Gate Closures—Huntington Project

Section	No. of openings	Bays	Closure time			Man-hr per opening	Man-hr per bay
			Crew	Hr	Man-hr		
I	1 5 6	20	2 14	4 19	8 266 274	46	13
II	3 2 18 23	25	4 10 14	6 4 33	24 40 462 526	23	21
III	12	52	14	52	728	61	14
Totals or means	41	97	....	....	1,528	37	15.7

Table 18·3 Pumping Stations—Huntington Project

Stations (by sections)	Contributing area, acres	Capacity			Cost		
		Cfs	Cfs per acre	Csm	Total	Per cfs	Per acre
Section I:							
13th St. W.	225	115	0.51	330	$ 68,000	$ 590	$ 300
Four Pole Creek	14,000	1,210	0.09	58	771,000	640	55
Krouts Creek	1,238	111	0.09	58	68,940	620	56
22d St., W.	68	79	1.16	740	71,490	900	1,050
East Road	78	92	1.18	752	75,880	820	970
7th St., W.	271	115	0.43	276	98,160	850	400
Total	15,880	1,722	....	......	$1,153,470	$ 4,420	$2,831
Mean	2,647	287	0.11	.....	192,200	731	472
Section II:							
5th Ave.	713	285	0.40	256	143,160	500	200
20th St.	552	135	0.24	154	71,220	530	129
16th St.	660	226	0.34	218	86,070	380	130
11th St.	90	83	0.92	590	55,360	665	613
4th St.	295	165	0.56	360	76,880	465	260
1st St.	159	91	0.57	366	51,380	562	323
9th St.	75	59	0.79	505	48,400	820	640
Total	2,544	1,044	....	......	$ 532,470	$ 3,922	$2,295
Mean	363	150	0.41	.....	76,070	560	328
Section III:							
Pats Branch	776	134	0.17	110	162,810	1,210	210
35th St.	96	83	0.86	550	66,530	800	690
Oak St.	101	85	0.84	540	66,360	780	655
Richmond St.	108	59	0.55	352	68,460	1,150	630
Total	1,081	361	....	6,215	$ 364,160	$ 3,940	$2,185
Mean	270	90	0.33	366	91,040	1,011	546
Grand total	19,505	3,127	....	......	$2,050,100	$12,282	$7,311
Mean	1,150	184	0.16	......	130,000	720	430
Same, omitting Four Pole Creek	5,505	1,917	....	......	$1,279,100		
Mean	344	120	0.35	......	75,000		

flooding of the major part of the city begins at El. 538. It reaches El. 534 usually in January of each year with little damage. There have been six serious floods in the last 50 years—in 1884, 1907, 1913, 1933, 1936, and 1937. The flood of 1937 reached El. 554, maximum stage of record. This caused an average depth of flooding of about 11 ft, with 12 to 15 ft in the main business section. Damages in 1937 were estimated at about $2,700,000. All business activity was suspended and utility service interrupted.

There are about 31,000 ft of levee from 3 to 65 ft high with an average height of about 14 ft, also concrete walls averaging about 11 ft high and

18 ft as a maximum for a distance of 5,500 ft, the walls and levees forming continuous barriers around the protected areas. The crest grade will be 3 ft above the 1937 flood level for El. 556 to 558.

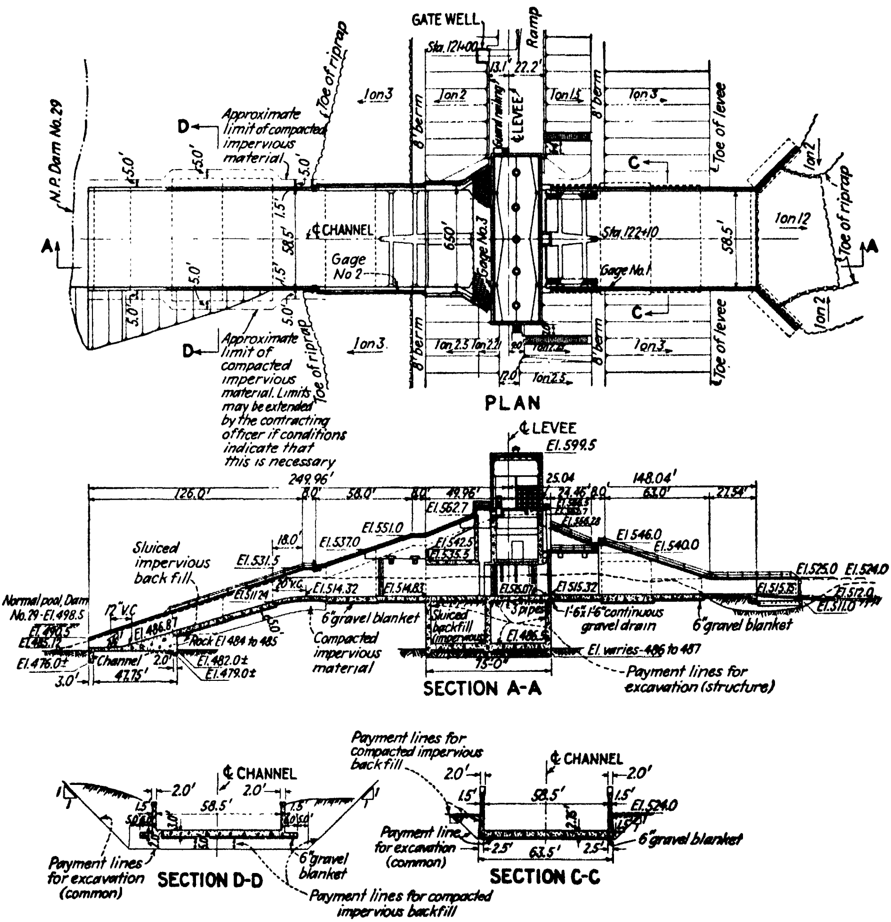


FIG. 18·6. Four Pole Creek pumping station at Huntington.

Storms Creek, with a drainage area of about 40 miles, flows through Ironton, dividing the area into two units, respectively, north and south of the creek. These will be enclosed by lines of walls and levees tied into high ground, lying generally east of the town. A diversion will be made in the lower channel of Storms Creek to give better railroad protection. Seventeen traffic openings have been left in the wall and levee system for use during normal periods. Six pumping stations are also provided.

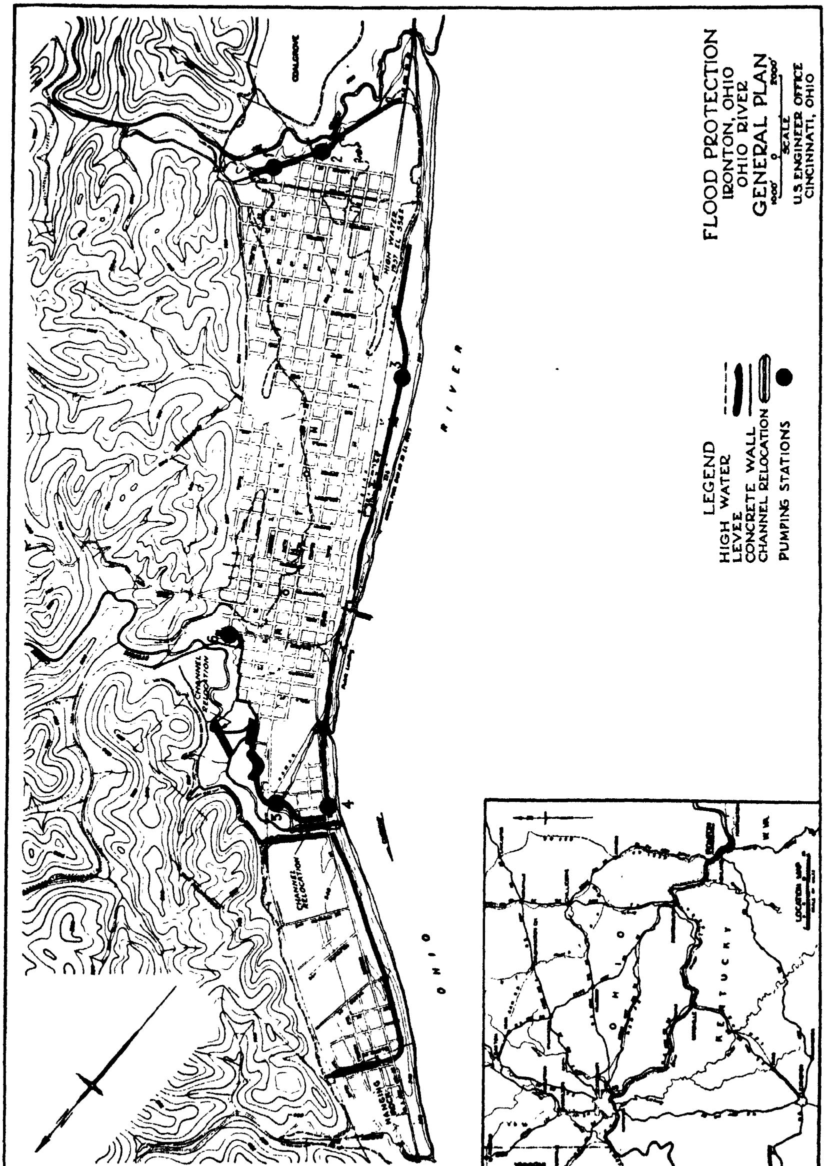


FIG. 18.7. Ironton project, general plan.

The total estimated cost of the project is $3,360,000, made up as follows:

Walls and levees	$2,590,000
Traffic openings	115,000
Internal drainage works	655,000
Total	$3,360,000

Local costs, assumed by the city of Ironton, totaled about $669,000.

Levees. The basic levee section has a top width of 12 ft, a riverside slope of 1 on 3 and a landside slope of 1 on 2½. On high levee sections, 10-ft berms are provided for each 20 ft of height, and wide berms are occasionally provided where required by soil conditions. A 7-ft-deep inspection trench is provided except for low heights. Levee slopes are usually grass-covered except where riprap is required. A typical levee section is shown in Fig. 18·8).

Walls. These are of reinforced concrete of standard cantilever type, with a key at the river end of the horizontal base for a cutoff and to provide resistance to sliding. Walls are not less than 18 in. thick with 21-in. top copings with special foundation treatment such as grouting, sheet piles, bearing piles, or removal of unsuitable material where necessary (see Fig. 18·8).

Gates. Seventeen traffic openings are provided in the wall and levee. Gates for closure during flood periods are similar in character to those used at Huntington.

Cincinnati, Ohio, Local Flood-protection Project

This is located in the city of Cincinnati on the right bank of the Ohio River about 472 miles below Pittsburgh.

Project Description (see Fig. 18·9). The project provides for the construction of a barrier dam across Mill Creek near its mouth, and a concrete wall, with appurtenant works, extending from the east bank of the creek along the Ohio River to high ground near the intersection of Baymiller and Kenyon St. The project will protect Mill Creek Valley and a small portion of the river-front section of Cincinnati against Ohio River floods of 1937 magnitude (maximum of record) with a 3-ft freeboard. The construction of the barrier dam will include a pumping installation to dispose of discharges of Mill Creek (drainage area 165 sq miles) into the Ohio River during flood periods. The entire project will involve the construction of a barrier dam, pump house including pumps, motors, and all necessary electrical and other mechanical equipment; appropriate outlet works to pass Mill Creek discharges during normal periods; levee and wall between the western abutment of the dam and pump house; and about 5,660 lin ft of concrete wall to form the eastern closure of the dam. The project also provides for the construction of gates, drainage-disposal facilities, and other appurtenant works.

The estimated cost of the project (1942) was $8,338,000, which included $1,078,000 for lands and damages. Flood damages in 1937 at Cincinnati were estimated at over $17,000,000, which, however, was for the entire city.

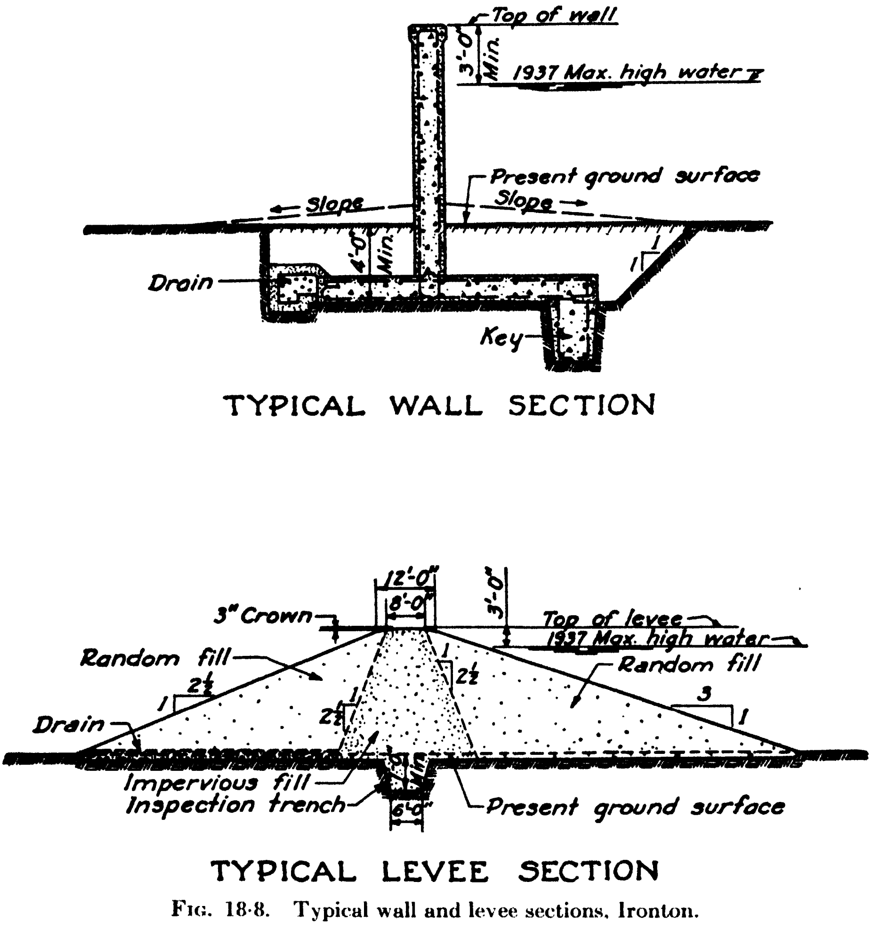


FIG. 18·8. Typical wall and levee sections, Ironton.

The pumping equipment on the Mill Creek project includes unusually large pumping units and total capacity. The units have a capacity of 1,500 cfs, against a head of about 29 ft; the vertical turbine pumps have impellers 120 in. in diameter, made by S. Morgan Smith Co.; the vertical, direct-connected General Electric motors are 15 ft in diameter, 6,500 hp, 180 rpm, and 6,600 volts.

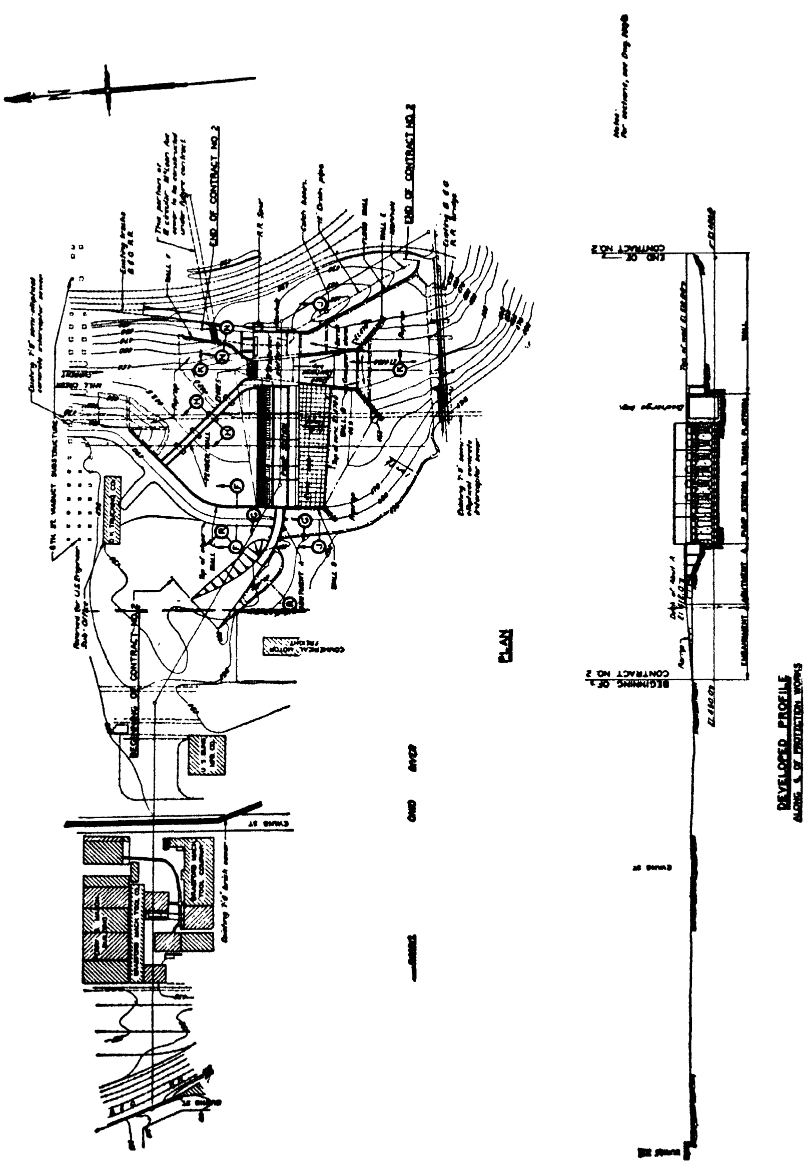


FIG. 18·9. Mill Creek project, Cincinnati.

Six units with 39,000 hp of motors give a total present pumping capacity of 9,000 cfs. Provision is made and the station structure completed for the installation of three more units so that the ultimate capacity of the station will be 13,500 cfs. Its outside dimensions are 270 ft 10 in. by 35 ft 10 in. and about 40 ft high above flow level, inside.

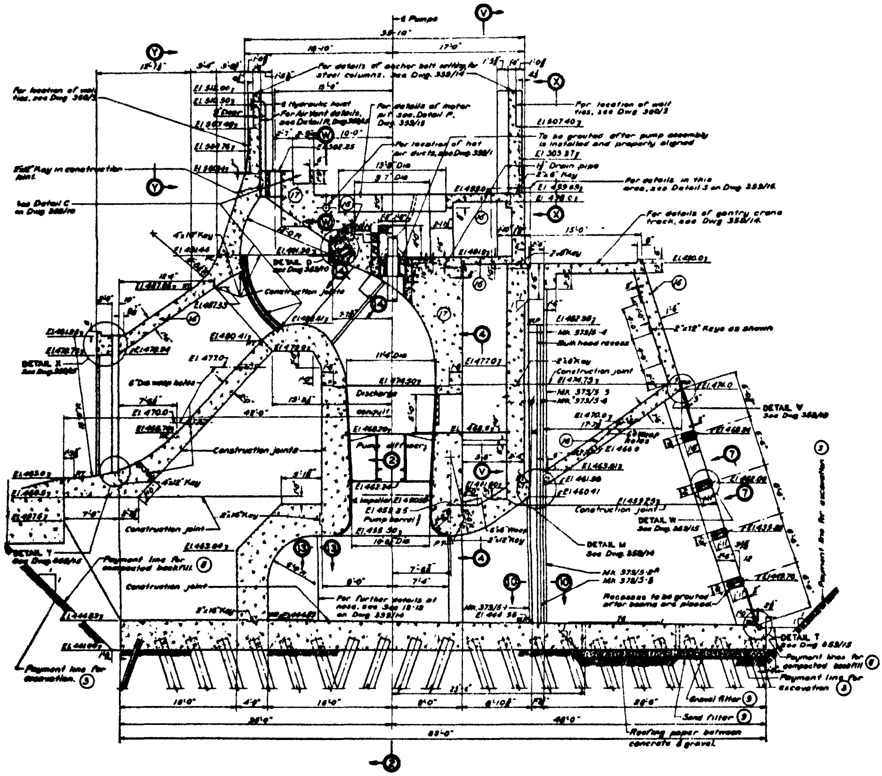


FIG. 18·10. Mill Creek project, pumping-station substructure.

The arrangement of the substructure with the intake, pump setting, and discharge conduit are shown in Fig. 18·10, from which it will be noted that the bottom of the intake channel and pump well is at El. 444.38; the center of the pump impeller at El. 450.58; and the station floor level and motor base at El. 499.0. Water is lifted vertically by the pumps to an invert level of El. 482.61, carrying and discharging therefrom to El. 463.0 at the discharge outlet.

The maximum probable summer flood with 80 per cent runoff and maximum rainfall of about 11 in. in 8 hr is estimated at 300 csm, or 47,600 cfs, on 165 sq miles. It is expected that the Mill Creek pumps will handle

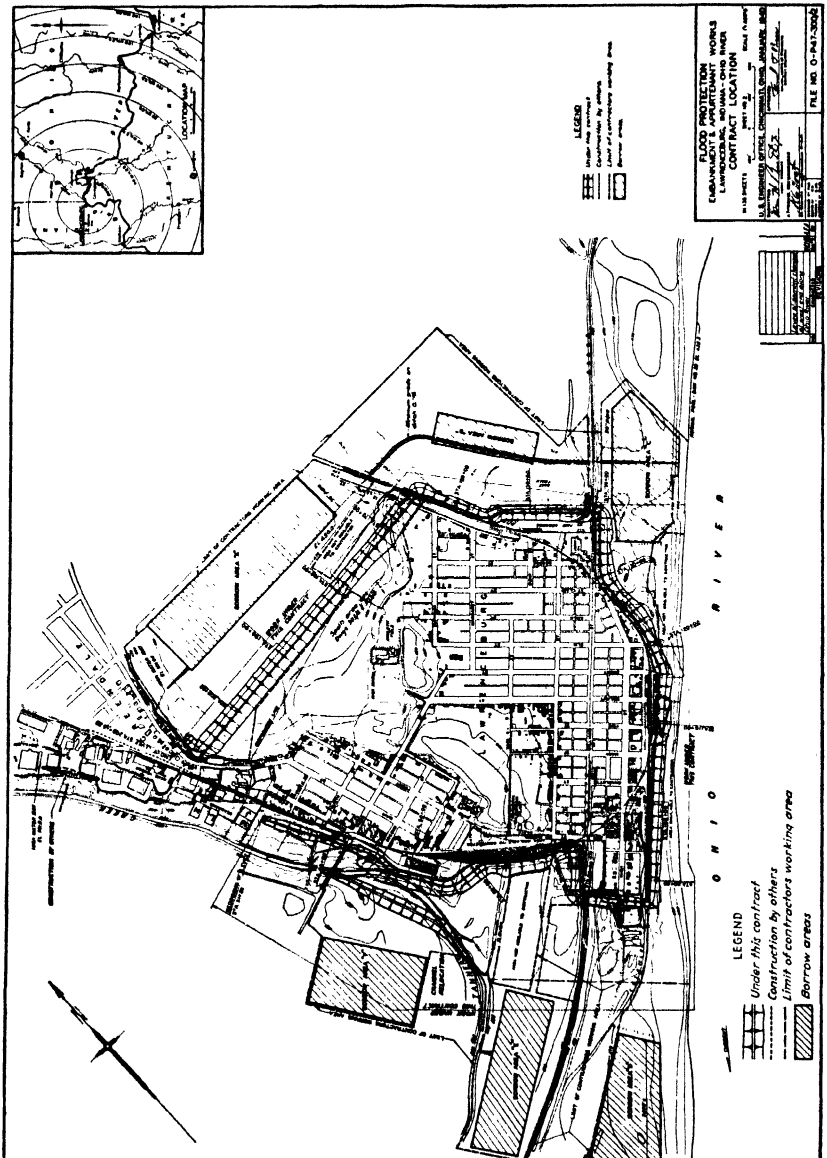


FIG. 18·11. Lawrenceburg, Ind., project.

flows from the creek without reservoir capacity except for the small amount available at the barrier dam.

Lawrenceburg, Ind., Local Flood-protection Works

This project is located in the city of Lawrenceburg on the right bank of the Ohio River, 493 miles below Pittsburgh and about 22 miles below Cincinnati in the southeastern corner of Dearborn County, Indiana.

Project Description. The existing project provides for reinforcing, supplementing, and extending the present protective works at the city of Lawrenceburg (see Fig. 18·11). Beginning at high ground on the eastern side of Greendale, a levee extends across the low ground in northern Lawrenceburg to the present levee system and follows the present system to and along the Ohio River and along Tanners Creek to tie into high ground with a concrete well on the western side of Greendale in the vicinity of Seagram's warehouses. The entire project comprises the construction of about 18,000 lin ft of levee; 320 lin ft of reinforced-concrete wall; the relocation of a part of Tanners Creek involving the excavation of about 130,000 cu yd of earth; the provision of eight traffic openings to permit free passage of traffic during normal periods; the construction of appropriate gates to close the aforementioned openings during flood periods; and the provision of appropriate internal drainage facilities and other appurtenant works. The project will protect practically the entire city of Lawrenceburg and the entire suburb of Greendale against floods equivalent to that of 1937 (maximum of record) with a 3-ft freeboard.

The estimated cost of the project (1943) is $2,460,000, which includes $120,000 for rights of way and damages. It is now (1945) substantially completed. Damages in the 1937 flood were estimated at about $2,900,000.

Levee sections have a 12 ft crest width with a freeboard of 3 ft above maximum high water (1937 flood) with side slopes of 1 on 2½ ft for a height of 20 ft, then 1 on 3 ft both up- and downstream slopes.

An interesting feature of the project is the use of relief wells back of the levees. The levee foundation material is impervious for a depth of 5 to 15 ft, underneath which is a deep deposit of sand and gravel. During floods, the water from the river side of the levee exerts through the pervious layer under the levee an upward pressure on the impervious layer above, tending to cause at such times outbreaks of water back of the levees.

The wells are constructed of a 16-in. steel pipe casing, with 3-in. tile inner pipe, gravel-packed. They serve to relieve the upward water pressure, with some outflow from the top. A total of 160 such relief wells were placed, with a total length of 8,394 ft, averaging about 52 ft deep at a cost of about $100,000. The use of relief wells in this manner is novel.

The contract price, with details of some of the more important items, was as follows:

Contract No. 1 Collector drains, etc.			$ 50,500
Contract No. 2			
Borrow excavation	2,700,000 cu yd at $0.14	= $378,000	
Embankment	2,500,000 cu yd at $0.04	= 100,000	
Dumped riprap	47,300 cu yd at $4.00	= 189,200	
Sand and gravel filters and drains	30,500 cu yd at $1.50	= 45,750	
Concrete	2,400 cu yd at $20.00	= 48,000	
		$760,950	
Total for contract			$ 931,300
Contract No. 3 Pumping stations, relief wells, etc.			
Excavation (borrow)	125,200 cu yd at $0.35	= $43,800	
Embankment	106,500 cu yd at $0.15	= 16,000	
Seepage drains, 8 to 18 in.	10,300 lin ft	= 19,150	
Collector drains, 8 to 30 in.	11,000	= 38,750	
Concrete	5,240 cu yd at $26.00	= 136,250	
Steel sheet piling	52,650 sq ft at $1.76	= 92,650	
Pumps, motors, and accessories		= 87,000	
Outfall sewers, 30, 42, 48, 78 in.	638 lin ft	= 38,400	
		$472,000	
Total payments to contractors			$1,782,900
Total cost			$2,076,800

Pumping stations as follows:

Station No. 1	3 units, 8,400 gpm	= 25,200 gpm	= 56 cfs
Station No. 2 (Seagram)	3 units, 11,500 gpm	= 34,500 gpm	= 77 cfs
Station No. 3 (Greendale)	3 units, 22,500 gpm	= 67,500 gpm	= 150 cfs
Total		127,200 gpm	= 283 cfs

During the flood stage in the spring of 1945, electric power was used at these stations as follows:

Station No. 1	78,000 kwhr	$1,572.80
Station No. 2	23,500	470.00
Station No. 3	8,800	176.00
Total	110,300 kwhr	$2,218.80

or a cost of 2 cents per kilowatt-hour (with no demand charge).

Operating and other elevations at No. 2 station are as follows: Top of levee 3 ft above 1937 flood elevation of 504, or El. 507. Gauge zero = El.

424.3, and gauge height for 1937 flood = 79.7. Close gates at gauge height 45.7 or El. 470.

Motor No.	Starts	Stops
1	El. 469	El. 465
2	El. 469.5	El. 465.5
3	El. 470	El. 466

Elevations are as follows:

Bottom of discharge well	458.75
Bottom of inlet well	460.45
Pumping-station floor	461.5
Pump intake	463.25
Bottom float-control pipe	462.95
Moment of float raise (zero on indicator)	463.3

In Fig. 18·12 are shown details of gates for the crossing of the levee by the double-track Baltimore & Ohio R. R. at about Sta. 112 + 68 of the levee line (see Fig. 18·11). These include a plan, sectional elevation, and assembly of this large gate structure, which includes an elaborate arrangement of steel beams and supporting steel trestles.

Paducah, Ky., Local Flood-protection Project, Ohio River Basin

Paducah is located in McCracken County, Kentucky, on the left bank of the Ohio River, 934 miles below Pittsburgh, Pa. The project there provides for the construction of a system of levees and concrete flood walls as follows:

Section A. An earth levee 13,920 ft in length, extending from high ground south of United States Highway No. 60, near Levin Ave. along the bank of the Ohio River to the foot of Terrell St.

Section B. Unit 1: 530 ft of new earth levee and 7,100 ft of concrete wall. Unit 2: 2,730 ft of new earth levee, 8,770 ft of concrete wall, and a barrier dam 367 ft long across Island Creek, the entire section extending from the upstream end of Section A along the bank of the Ohio River to the foot of Mill St.

Section C. An earth levee 31,520 ft in length, extending from the upstream end of Section B in a general southerly direction to high ground north of the Illinois Central Railroad Bridge over Bee Slough.

Twelve pumping plants for the disposal of interior drainage and appurtenant structures are included. The usual freeboard of 3 ft above the 1937 flood is provided. The estimated cost, revised in 1944, is $3,954,000 for construction and $100,000 for lands and damages, or a total of about $4,350,000.

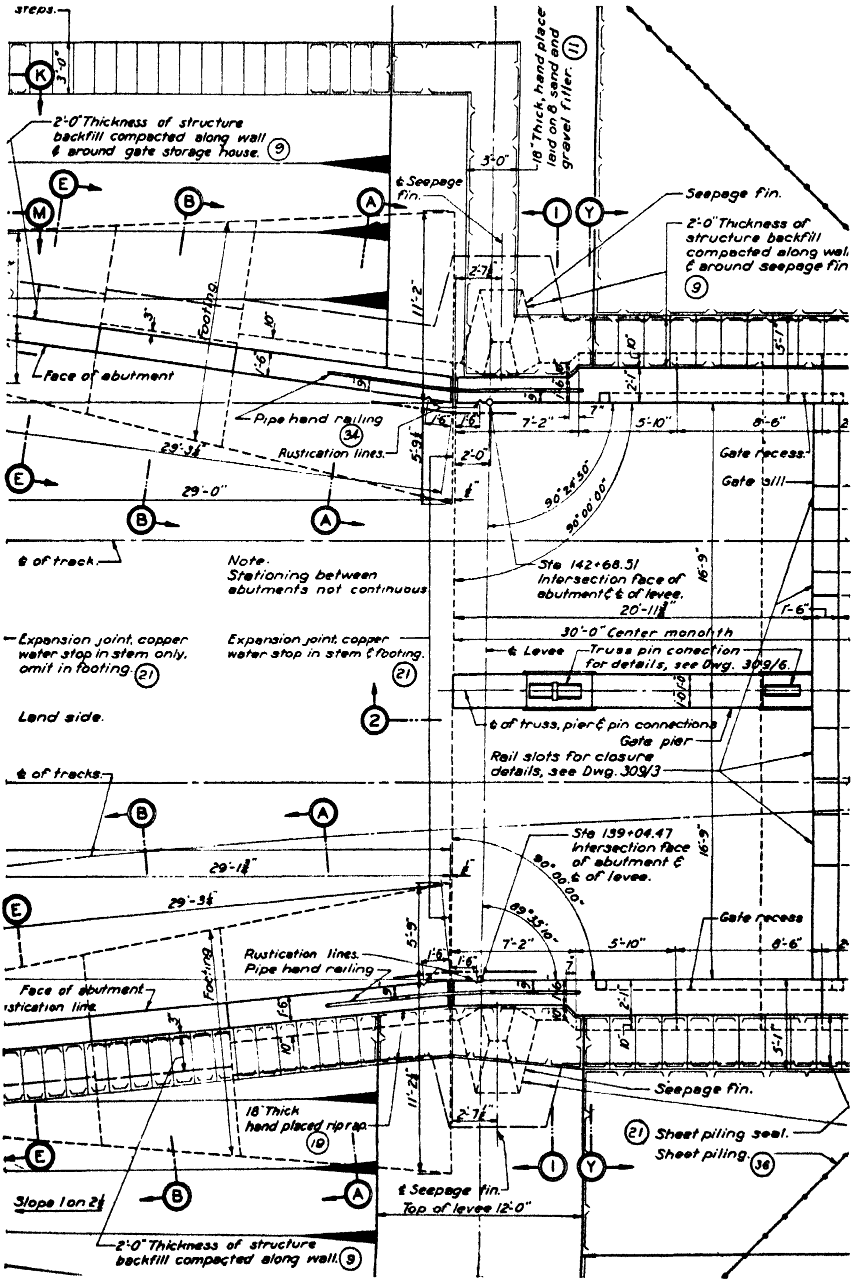


Fig. 18·12. Gates at lower railroad crossing, Lawrenceburg project. (*See also pages 370 and 371.*)

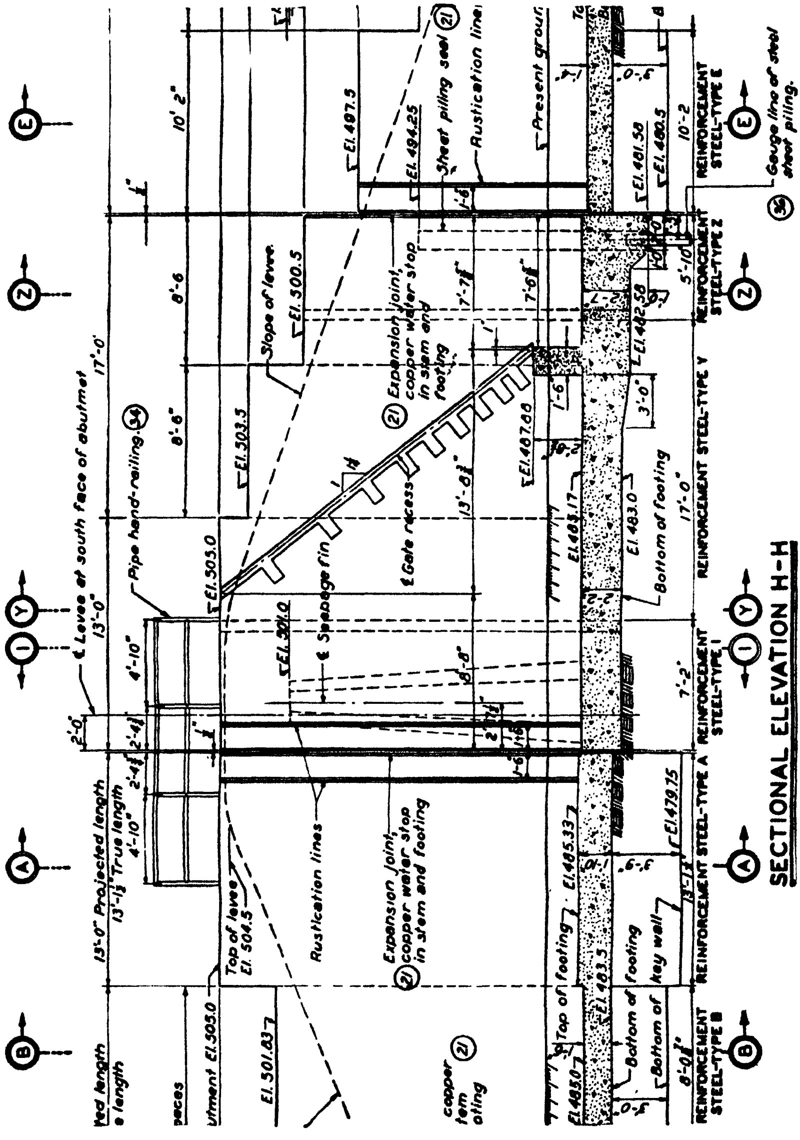
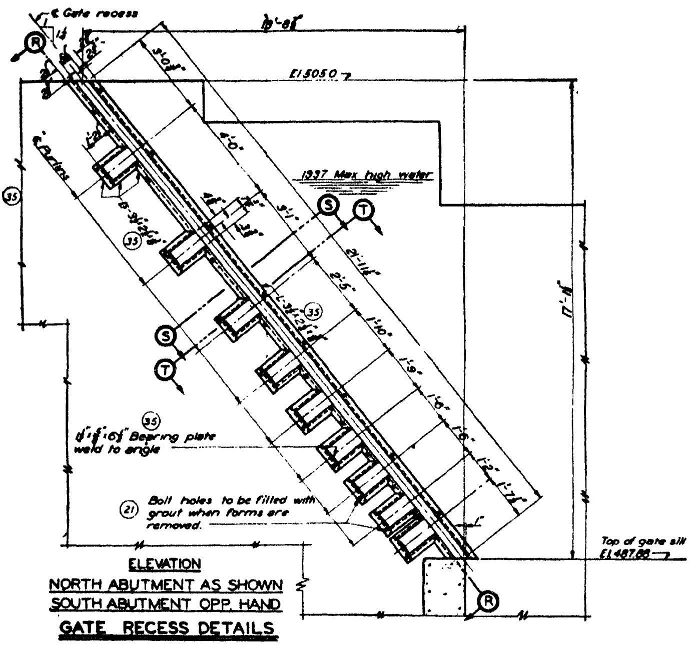


Fig. 18·12. (Continued.)



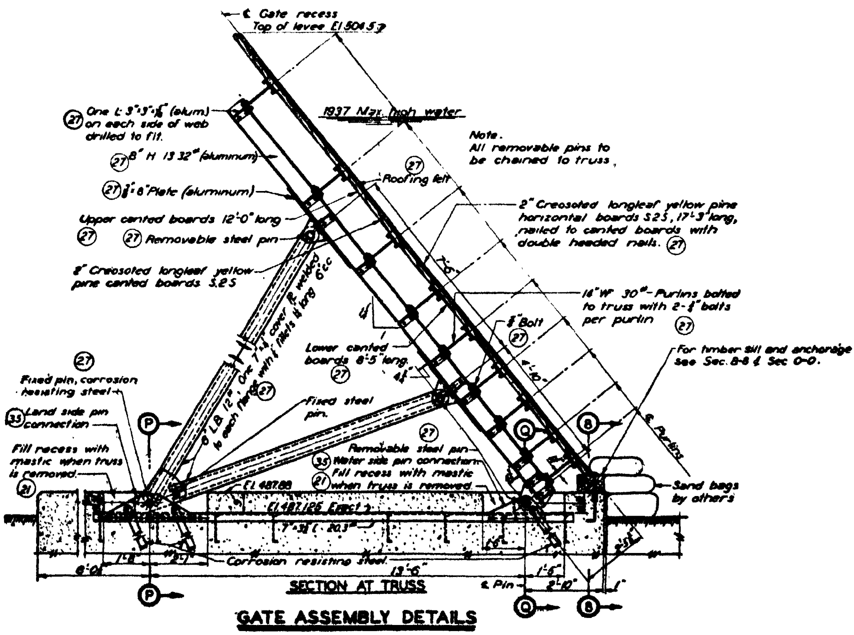


FIG. 18·12. (Continued.)

At the end of the fiscal year in 1944, Sections A, B, and C and seven pumping plants (exclusive of pumps and motors) were substantially completed. Island Creek barrier dam was about two-thirds completed. The remaining work was deferred for the duration of the war.

In Fig. 18·13 is shown the flood wall and provision for closure at Sta. 119 + 62 at one of the streets.

Tell City, Ind., Local Flood-protection Project, Ohio River Basin

Tell City is located in Perry County, Indiana, on the right bank of the Ohio River, 726 miles below Pittsburgh, Pa. The project there provides

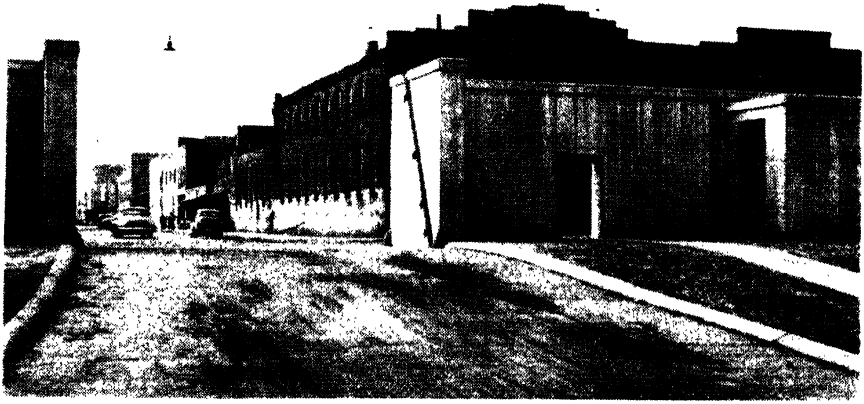


FIG. 18·13. Provision for street closure, Paducah project.

for the construction of a system of levees and concrete flood walls. Section I: an earth levee 2,900 ft in length, extending from high ground near the corner of Eighth and Lafayette St. along the bank of the Ohio River to Fourth St. between Tell and Fulton St. and a closure levee 750 ft in length along the north side of Payne St. Section II: a concrete flood wall 6,200 ft in length extending from the upstream end of Section I along the bank of the Ohio River to high ground at City Boundary St.

Two pumping plants for the disposal of interior drainage and appurtenant structures are also included. The project provides for a freeboard 3 ft above the level of the flood of 1937. The estimated cost is about $920,000 for construction and $37,000 for lands and damages, or a total of $957,000.

In Fig. 18·14 is shown a portion of concrete flood wall, including the closure structure at Sta. 46 + 96 (Section II); the gauge reading at the time was 45.85.

Louisville, Ky., Local Flood-protection Project, Ohio River Basin

Louisville is located in Jefferson County, Kentucky, 607 miles below Pittsburgh, Pa., on the left bank of the Ohio River. Plans for the project include construction of about 40,000 ft of new earth levee and about 16,000 ft of concrete wall, extending from high ground on the right bank of Beargrass Creek about 1 mile above its mouth along the left bank of the Ohio River to high ground in the vicinity of Iroquois Park.

About 14 pumping plants will be provided for the disposal of interior drainage. The project will have the usual freeboard of 3 ft above flood

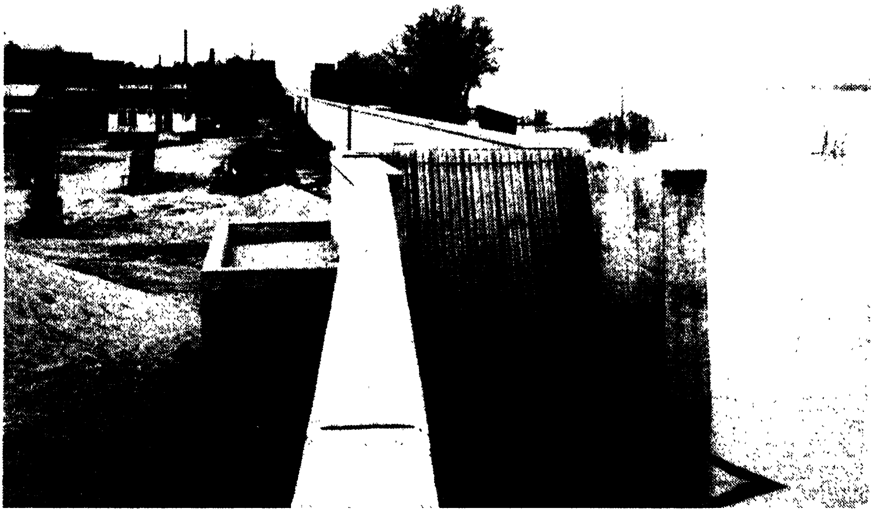


FIG. 18·14. Concrete flood wall and closure structure, Tell City project.

of that equivalent to 1937. The estimated cost is about $7,000,000 for construction and $1,000,000 for lands and damages.

Figure 18·15 shows the view at the time of the 1937 flood on River Road from the Munroe bridge. The water surface was 8½ ft above the crest height.

Jeffersonville-Clarksville Local Flood-protection Project, Ohio River Basin

The cities of Jeffersonville–Clarksville are located in Clark County, Indiana, on the right side of the Ohio River, 603 miles below Pittsburgh, Pa.

The project includes Section I: an earth levee 19,630 ft in length, including 7,030 ft of roadway levee and 12,600 ft of new earth levee, extending from high ground from Silver creek and Emery Crossing Road along the right bank of the Ohio River to the foot of Smyser Ave. in Clarksville.

Section II. Unit 1: 10,070 ft of concrete wall extending along the right bank of the Ohio River from near the foot of Duncan St. downstream to the vicinity of Smyser Ave. to join with Section I. Unit 2: 1,560 ft of concrete wall from high ground near the foot of Jackson St. in Jeffersonville along the right bank of the Ohio River to join Unit 1 near Division St.; 6,320 ft

Fig. 18·15. 1937 flood at Louisville, Ky.

of new levee and 1,150 ft of roadway levee across three swales northeast of Jeffersonville.

Fourteen pumping plants and appurtenant works for disposal of interior drainage are included. The estimated cost (revised in 1944) is $3,323,300 for construction and $462,000 for lands and damages, or a total of $3,785,300.

In Fig. 18·16 is shown a portion of concrete flood wall looking toward West Market St. in Unit 1 of Section II, including a closure section.

This project will have the usual 3-ft freeboard above the flood of 1937.

Turtle Creek Local Flood-protection Project of Westinghouse Electric Corporation at East Pittsburgh, Pa.

Turtle Creek, with a drainage area of 146 sq miles, flows into the Monongahela River about 1½ miles below East Pittsburgh, Pa. Along the banks of Turtle Creek are the East Pittsburgh Works of the Westinghouse Electric Corporation with floor levels of plants El. 738.5 to 742. Normally the water level in the creek is at about El. 726; but, during the 1907 flood, some plant floors were covered with water 50 in. deep; in 1924, this was 2 ft; and, in 1936, one aisle was buried under 7½ ft of water.

This submergence is caused by backwater from the Monongahela, and the flood-control project is designed to hold back the Monongahela and to handle the flow of Turtle Creek when the Monongahela exceeds El. 738, at which time the river stage at the gauge above Lock No. 2 is at 31.25 ft.

FIG. 18·16. Flood wall and closure section, Jeffersonville-Clarksville project.

Maximum flow in Turtle Creek usually occurs at least 24 hr earlier than high water in the Monongahela. When the Monongahela reaches a stage of 31.25 ft, flow in the creek has decreased to 4,000 or 5,000 cfs.

Water from the Monongahela is kept from backing up Turtle Creek by a steel gate 80 ft long and 30 ft high installed between masonry side walls. A smaller gate 40 ft long and 20 ft high is required to close off Braddock Ave., which parallels Turtle Creek at the low point in the valley. These gates are lifted by a hoisting mechanism consisting of two drums; one for the creek gate, and one for the street gate. Steel cables run over sheaves to each gate, as shown in Fig. 18·17, which is a diagrammatic sketch of their arrangement. The drums are engaged through a clutch to a 25-hp a-c motor with another auxiliary clutch for emergency use of a 25-hp gas engine.

The top of the sill for the larger gate is at El. 721.25; the top of both

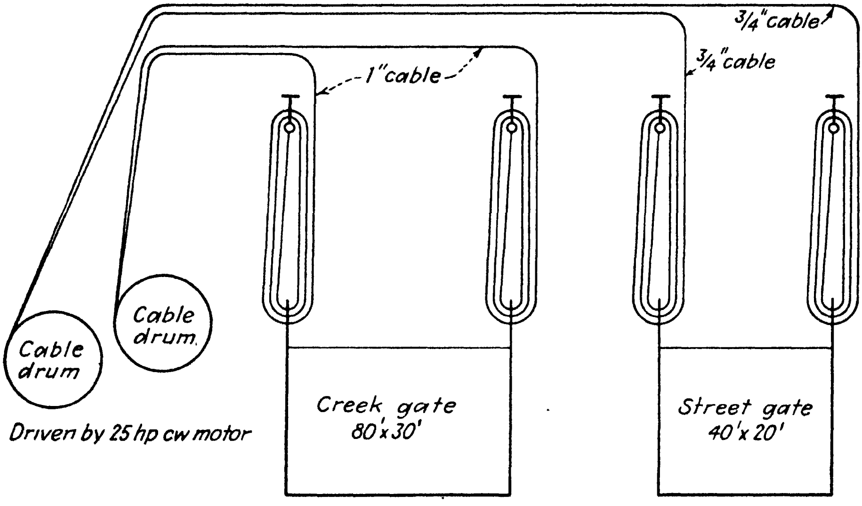


FIG. 18·17. Gate control, Turtle Creek project.

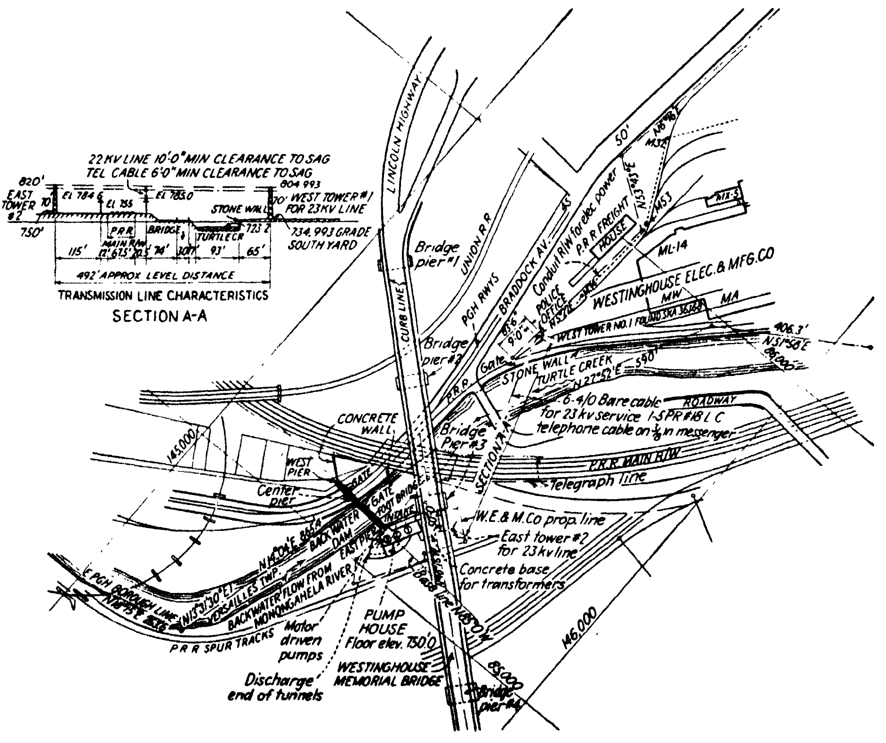


FIG. 18·18. Plan of Turtle Creek project.

gates is at El. 750 when closed; the sill of the smaller gate is at El. 730.5 (approximate street level). The gate shells are of steel plates with channels between, framed with I beams and heavily trussed. The larger gate weighs about 75 tons and the smaller gate 35 tons. When not in use, the gates are held at 20 ft above stream and highway level.

The general arrangement of the project is shown in plan in Fig. 18·18. Figure 18·19 is a view of the downstream side of the barrier dam with water

FIG. 18·19. Pumping water at downstream side of barrier dam, Turtle Creek project.

being pumped into the Monongahela. Two of the three pumping units are in operation.

At the east end of the dam above high-water level is the pumping station. Three 5,000-hp vertical motor units are installed here. The pump impellers for the three units are 13 ft. 11 in. in diameter and are set at a level 10 ft below creek flood stage. Each is direct-connected by a 30-ft vertical steel shaft to a 5,000 hp 112 rpm vertical synchronous motor 23 ft in diameter that is installed above high-water level. The total pump capacity under a head of 3 or 4 ft is 7,500 cfs for the three units, or 51 csm. At 10 ft pumping head, the total capacity is about 5,000 cfs, or 45 csm. These 2,500 cfs pumping units are the largest in cfs capacity that have come to the author's attention.

During the 1936 flood, it has been estimated that about 24 per cent of the additional water flowing into Turtle Creek from the Monongahela was unable to flow downstream until the Monongahela had absorbed its slack water. The pumping project will be effective in keeping the creek level down at times when the gates are closed, unless a subsequent local flood of magnitude should occur on Turtle Creek while the Monongahela is in high flood. The chance of this occurring appears to be remote, however.

Moreover, the U.S. Engineers have planned a storage reservoir for flood protection in the lower portions of Turtle Creek.[1] This will be about 8 miles above the mouth of the creek where the drainage area is 54 sq miles. This reservoir, with a 128-ft dam with a spillway level at El. 900, will have a capacity of about 22,000 acre-ft for flood control, or 7.5 in. depth upon its drainage area, and is estimated to cost about $4,300,000, of which $1,700,000 will be supplied by local interests. This will eliminate floods from Turtle Creek itself, and the Westinghouse project will prevent backwater floods from the Monongahela River.

The total cost of the Turtle Creek barrier-gate dam, pumping facilities, and auxiliaries was about $550,000. The area protected in this manner—essentially the works of the Westinghouse Company— is about 150 acres. Without this project, flood damages in the Turtle Creek district could be expected to occur aggregating $1,000,000 or more.

The Turtle Creek project is of especial interest as an example of a flood-control problem undertaken and solved by a private corporation. It is similar in the method of solution to the Mill Creek project at Cincinnati.

Local Flood-protection Works upon the Tennessee River at Chattanooga[2]

The city of Chattanooga is located in the southern part of Tennessee between the Watts Bar and Chickamauga projects on the Tennessee River, where the drainage area is 21,400 sq miles. The population was about 120,000 in 1930 and 128,163 in 1940. It leads the valley in industrial activity. In 1935, the yearly value of manufactures was $46,000,000. The maximum monthly rainfall averages about 5.6 in. and occurs in March, and the minimum of about 3 in. occurs in October. The average yearly rainfall is about 52 in.

The city is well located with respect to raw materials for its major industries, including textiles, steel, lime, and cement. It is favorably situated with natural, local, and regional markets and enjoys a dominant position in a tributary trade area 100 to 150 miles in diameter. It has

[1] H.R. Doc. 507, 78th Cong., 2d Sess., 1944.

[2] The Chattanooga Flood Control Problem, H.R. Doc. 91, 76th Cong., 1st Sess., 1929; also H.R. Doc. 479, 76th Cong., 2d Sess.

excellent transportation facilities both by railroad and by water in the 648-mile 9-ft navigable channel of the Tennessee River of the TVA with its accompanying hydroelectric power.

Floods. Chattanooga is particularly vulnerable to floods, although the largest occurred long ago, in 1867, when it was a small town. Such a flood, if repeated with the city's present population and development, would flood about 8,000 acres, including the larger part of the business district, the manufacturing district, the railway station and yards, and the greater part of the railroad tracks within the city limits.

Maximum flood stages with total flood runoff have been as follows:

Floods	Maximum stage, ft	Total runoff, in.
1867	57.9	
1875	53.8	7.4
1886	52.2	6.9
1917	47.7	3.8
1918	42.7	
1920	43.6	3.7
1936 (with Norris Reservoir in operation)	37.1	8.0
1936 (without Norris Reservoir estimated)	41.0	8.0

The design flood for Chattanooga, as determined by the Board of Consultants, is based upon a maximum runoff of 5,000 $\sqrt{\text{drainage area}}$, or 730,000 cfs, which is about 60 per cent greater than the record flood of March, 1867.

The total flood runoff for the design flood is assumed as follows:

Days	Total Runoff, In.
1	1.3
3	3.7
4	5.0
7	8.0
10	10.3
12	11.3
15	12.3

Reservoir Development. The reservoirs of the TVA system above Chattanooga include a total ultimate capacity of about 9 million acre-ft of storage capacity, of which about 7 million acre-ft had been constructed in 1945 (see Table 13·2, page 285). This effective flood control of the system is expected to reduce the peak flow in the design flood, of 730,000 cfs, to about 500,000 cfs, which will enable local protective works at Chattanooga to be based upon a maximum stage of 60 ft.

As far as Chattanooga is concerned, most floods could be handled at less

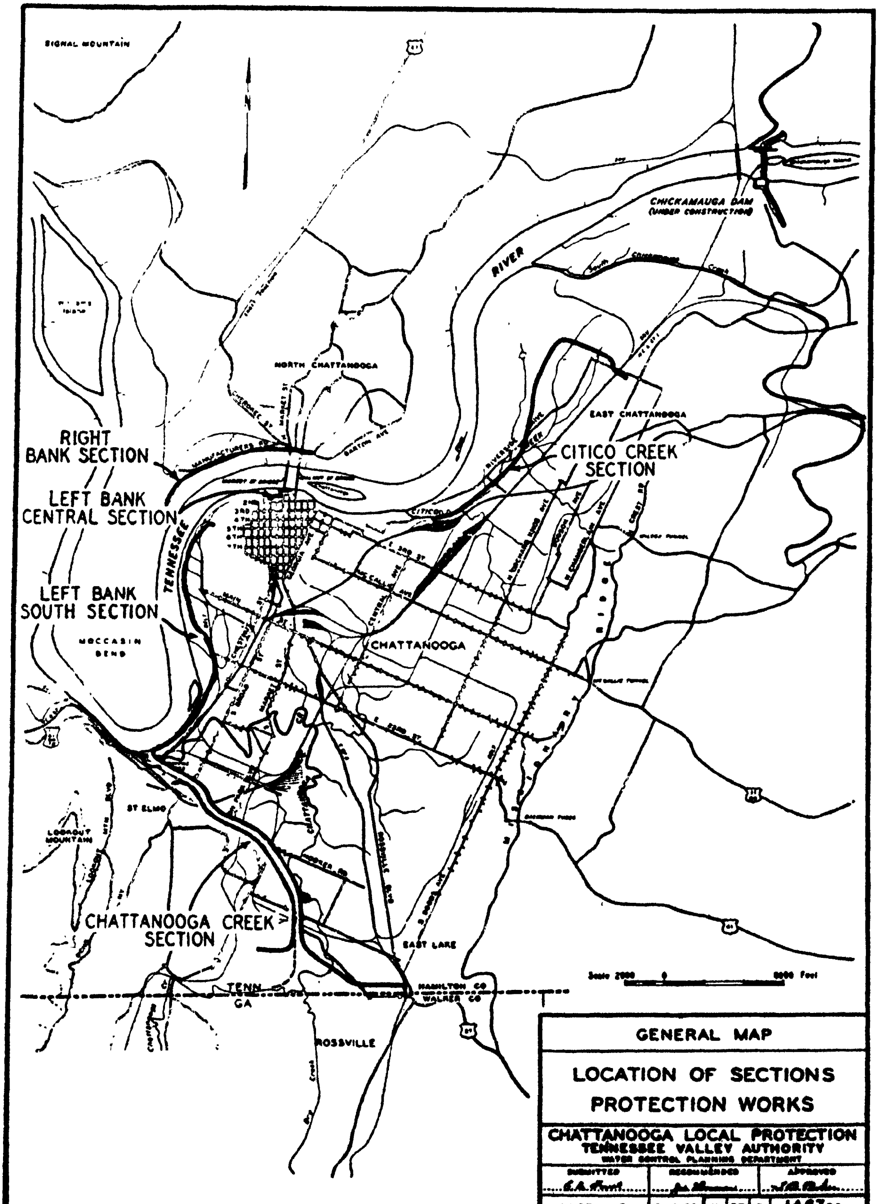


FIG. 18·20. Proposed flood-protection works at Chattanooga.

cost by local protection works than by reservoirs. The latter are required, however, and are a part of the comprehensive development of the main river. Moreover, a levee system above stage 60 introduces some elements of uncertainty and danger, and the plan was adopted of basing the local protection works upon a safe channel capacity of about 500,000 cfs, equivalent to stage 60, requiring upstream reservoir flood-control storage of approximately 4 million acre-ft.

Proposed Local Protection Works. A series of low levees, totaling about 7,400 ft in length, and a 2,300-gpm pumping station, for the protection of the airport, were constructed under WPA auspices. The levee crest protects against a flood stage of 49 ft and reaches a height of 12 ft, with a top width of 10 ft and side slopes 1 on 2½. The protected area is about 200 acres.

The proposed protection works include (see Fig. 18·20)

1. The excavation of a river channel for Chattanooga Creek, shortening its length from 41,000 ft to about 16,000 ft, with the construction of levees on each bank for a total length of 30,000 ft. Included are two pumping stations, 10 new bridges, and levee openings with gate structures and a major and minor railroad modification.
2. On the Tennessee River left bank, 9,600 ft of levee and 4,900 ft of flood wall in the south section and 2,400 ft of flood wall in the central section.
3. On Citico Creek, a channel relocation, with 2,500 ft of new channel and 17,300 ft of levee, with two pumping stations and three levee openings with flood gates.
4. On the Tennessee River right bank, 9,000 ft of levee, two pumping stations, and one levee opening.
5. Rossville section—7,000 ft of levee and 6,000 ft of drainage ditch.
6. Brainerd and airport section—14,500 ft of levee, 4,500 ft of diversion channel on South Chickamauga Creek, one pumping station, and one levee opening. The local improvements previously described will not be utilized.
7. St. Elmo section—1,000 ft of levee, 100 ft of flood wall, one pumping station, and one levee opening.

Levee sections will have an 8-ft crown width, 1 on 3 slopes on the river side, and variable slopes from 1 on 2 to 1 on 3 on the land side. Both walls and levees will have a minimum freeboard of 5 ft above the 57.9-ft stage. Standard sections of walls, levees, and the Chattanooga Creek diversion channel are shown in Fig. 18·21.

Preliminary cost estimates and benefits are as given in Table 18·4.

Since the Brainerd, airport, and St. Elmo areas show an excess of yearly cost over benefits, whereas the latter are based upon anticipated develop-

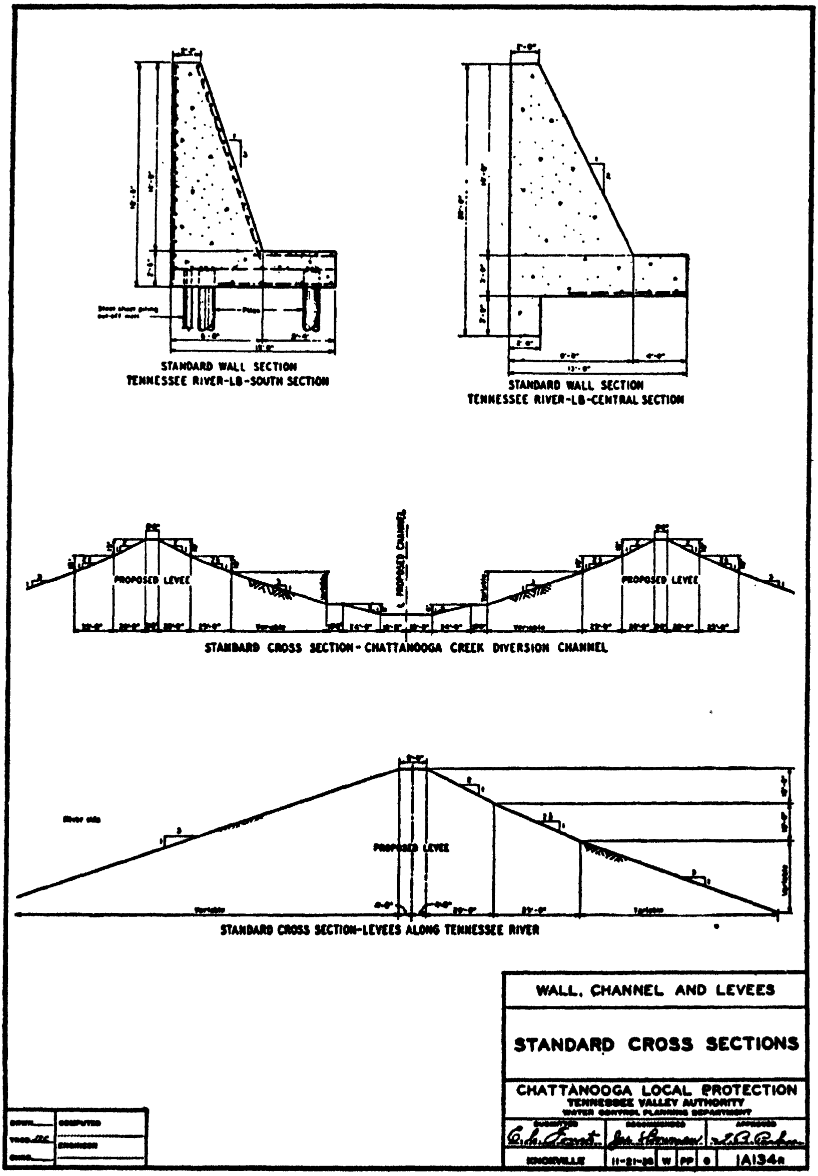


FIG. 18·21. Standard wall and levee sections at the Chattanooga project.

Table 18·4 Chattanooga Local Flood Works—Costs

Section	Total investment	Total yearly cost	Total yearly benefits
Chattanooga Creek	$ 6,942,000		
Tennessee River, left bank	4,540,000		
Citico Creek	3,149,000		
Tennessee River, right bank	1,448,000		
Rossville	563,000		
Subtotal	$16,642,000	$810,000	$1,838,000
Brainerd and airport	$ 3,132,000		
St. Elmo	282,000		
Subtotal	$ 3,414,000	168,000*	159,000
Total	$20,056,000	$978,000	$1,997,000

* From anticipated developments.

ments, these portions of the project are to be deferred and the project limited to the cost of $16,642,000, for which the area protected is about 20,000 acres, or a cost of about $800 per acre protected.

In table 18·4, the yearly cost basis was as follows:

Interest. 3½ per cent Federal and 4½ per cent non-Federal (which was about 20 per cent of the total cost).

Amortization. Life of 50 years for fixed parts and 25 years for movable parts. Changes based upon above interest rates compounded annually.

Benefits. Average annual flood damage $1,838,000 based upon appraisals and estimates of damage that would occur at present time (except for the omitted areas to be defined).

The project of local flood protection at Chattanooga has not yet (1947) reached the construction stage.

Connecticut River Basin, Local Flood-protection Works

In addition to the proposed 20-reservoir scheme for flood control upon the Connecticut River, 10 local protection projects are proposed, including dikes and walls, supplemented by pumping stations and in some cases by drainage conduits. These are now (1945) nearly completed and include works in some of the cities in Massachusetts and Connecticut upon the main river.

Existing Dikes. State and local interests have previously provided local protection by means of dikes at various points along the Connecticut River in Massachusetts and Connecticut, in most cases to give protection against a flood of the magnitude of that of November, 1927. The flood of March, 1936, however, topped all dikes in the valley. The cost of these works has been about $1,400,000 in Connecticut and $900,000 in Massachusetts, or a total of about $2,300,000.

New Works. Under the various flood-control acts of 1938–1942, appropriations totaling about $19,500,000 were made for local flood-protection works at 10 cities in Massachusetts and Connecticut between Northampton, Mass., and Hartford, Conn., as shown in Table 18·5. The total cost of the 10 projects is about $18,800,000, or a yearly cost of about $900,000. The cost per acre protected, averaging $2,560, is high, reflecting rather more then usual difficulties in construction. This is particularly true in the case of Holyoke, with a cost per acre protected of $24,500, due to the complication of structures and waterways and a relatively small area protected. Two of these projects, *viz.*, at East Hartford and Holyoke, will be described in detail.

East Hartford Local-protection Works. The local-protection works at East Hartford, Conn., were constructed during 1939–1941 to protect an area of approximately 762 acres consisting of industrial, commercial, agricultural, residential, and some undeveloped areas. The protection works consist of approximately 21,000 lin ft of earth dike with a maximum height of 37 ft and 600 ft of concrete flood wall with a maximum height of 25 ft aboveground along the Connecticut and Hockanum Rivers with three pumping stations, and two stop-log structures. The earth dike is of rolled- and hydraulic-fill construction with slopes of 1 on 3 and a cutoff and blanket on the river side of selected impervious materials protected by riprap. Where necessary because of pervious foundation conditions, sheet-steel piling has been used as a cutoff.

The pumping stations are constructed of concrete, structural steel, and brick and provide for pumping storm flow and sanitary sewage during flood periods. Cherry St. pumping station provides for a drainage area of 35 acres, has a capacity of 38 cfs, and is equipped with two 16-in. engine-driven volute pumps. Pitkin St. pumping station provides for a drainage area of 57 acres, has a capacity of 62 cfs, and is equipped with two 20-in. engine-driven volute pumps. Meadow Hill pumping station provides for a drainage area of 980 acres, has a capacity of 224 cfs, is equipped with four 36-in. engine-driven propeller pumps and one 20-in. motor-driven volute pump, and is provided with a storage pond with a capacity of 40 acre-ft. One stop-log structure is for the railroad tracks, and one is for Main St. highway.

The total cost to the government of the project is approximately $2,229,000, as follows:

Earth dike and concrete wall	$1,902,000
Cherry St. pumping station	53,000
Pitkin St. pumping station	57,000
Meadow Hill pumping station	217,000
Total including engineering, overhead, etc.	$2,229,000

The average annual cost of maintenance and operation, including interest and amortization, for the project is $128,000, and the annual benefits are $190,000 or more.

The levees and walls are designed to protect against a design flood greater than any of record, reduced by the proposed 20-reservoir scheme for flood control. Levee and wall grades are at all points above the maximum stage of the record flood of March, 1936.

Holyoke Local Protection Works. The local protection works at Holyoke, Mass., were constructed in 1939–1940, to protect an area of 105 acres, principally industrial and commercial. They consist of 5,020 ft of concrete flood wall with an average height of 12 ft aboveground and 380 ft of earth dike with an average height of 6 ft, from the Holyoke Water Power Company dam to high ground in the vicinity of Mosher St. with 5,500 lin ft of steel-sheet pile cutoff, four concrete pumping stations, five stop-log structures, nine tailrace gate structures, and one concrete plug in an abandoned tailrace conduit.

The pumping stations were constructed at certain of the tailrace structures to provide for pumping storm water, local sanitary sewage, and some industrial sewage during flood periods. Pumping stations Nos. 1 and 2 each have two 24-in. centrifugal electrically driven pumps with a total pumping capacity of 62 cfs in each station. Pumping stations Nos. 3 and 4 each have two 30-in. centrifugal electrically driven pumps with a total pumping capacity of 78 cfs in each station.

Three stop-log structures permit railroad-track crossings of the New Haven Railroad, and the remaining two stop-log structures provide passage through the walls along the No. 2 wasteway. The gate structures were constructed at the tailrace outlets of the mills situated along the site of the flood wall. Figure 18·22 shows pumping station No. 2 and also the tailrace gate structure.

The total cost to the government of the project is approximately $1,349,-000.

In addition to the above described project, which has already been completed, designs are in progress for completing the local protection works for Holyoke, including the raising of the existing dikes in the Springdale section of the city. This proposed additional protection consists of concrete flood walls, an earth dike, and three pumping stations.

The total estimated cost of the Holyoke project, including the Springdale dike, is $2,814,000. The average annual cost of maintenance and operation, including interest and amortization, is $156,000, and the total benefits are approximately $230,000.

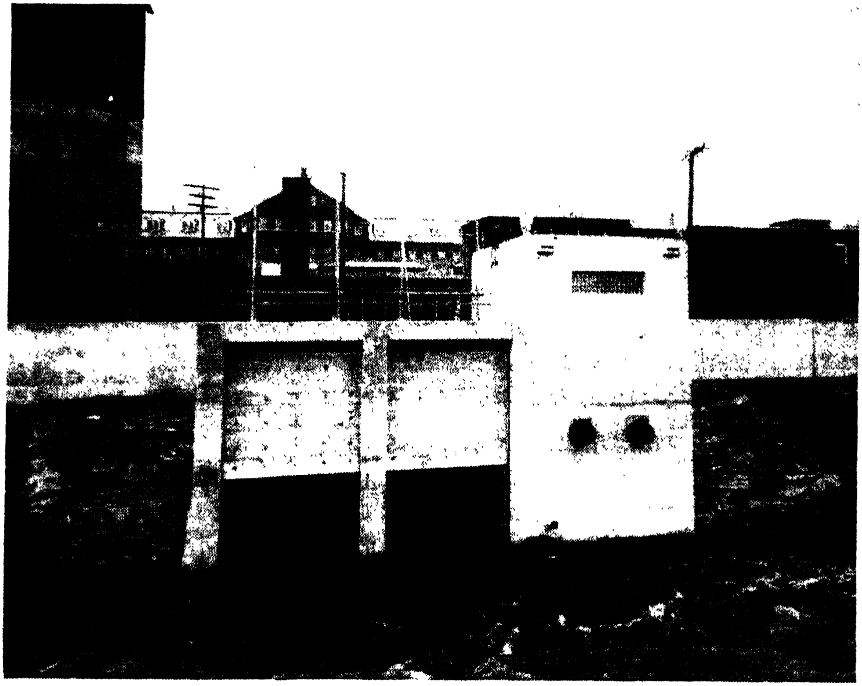


FIG. 18·22. Pumping station and tailrace structure, Holyoke project.

Levee and wall grades are designed upon a basis similar to that for those described for the East Hartford project.

Merrimack River, Local Flood-protection Works

Certain cities and towns in the Merrimack River Basin required local flood-protection works, including levees or walls, pumping stations, etc. These include Fitchburg, Mass., and Nashua, N.H., upon the Nashua River, Lowell, Lawrence, Haverhill, and North Andover in Massachusetts. The project at Fitchburg was completed in 1938; the other projects are in only preliminary stages. The project at Lowell, which supplements the system of reservoirs planned for the Merrimack River, is described in some detail, as a typical project.

In Table 18·6 are given data of area and cost for these six projects, which

Table 18·5 Local Flood-protection Projects—Connecticut River Basin

Project	Date completed	Area protected, acres	Cost	
			Total	Per acre protected
Hartford, Conn.	1944	3,100	$ 6,805,000	$ 2,200
East Hartford, Conn.	1941	762	2,620,000	4,500
Springfield, Mass.	1942	819	1,197,000	1,460
West Springdale, Mass.	1943	1,044	1,282,000	1,220
Chicopee, Mass.	1944	1,020	2,092,000	2,050
Holyoke, Mass.	1940	105	2,574,000	24,500
Northampton, Mass.	1943	223	1,100,000	5,000
Springdale, Mass.	. . .	128	447,000	3,500
Riverdale, Mass.	. . .	450	540,000	1,200
Winsted, Conn.	. . .	73	143,000	2,000
Total or mean	. . .	7,724	$18,800,000	$2,560

will protect a total area of 1,155 acres at an estimated cost of $5,218,000, or an average cost per acre protected of $4,500.

Table 18·6 Local Flood-protection Works—Merrimack River Basin

Project	Area protected, acres	Cost	
		Total	Per acre protected
Fitchburg, Mass.	870	$1,375,000	$ 1,580
Nashua, N. H.	70	217,000	3,100
Lowell, Mass.	120	1,295,000	10,800
Lawrence, Mass.	65	277,000	9,200
North Andover, Mass.		323,400	
Haverhill, Mass.	30	1,730,000	57,700
Total or mean	1,155	$5,217,400	$ 4,500

Lowell, Mass., Local Flood-control Works. The city of Lowell is located about 40 miles above the mouth of the Merrimack River in northeastern Massachusetts. The flood-control project includes about 5,100 lin ft of earth dike, sheet piling, and concrete wall, protecting about 120 acres of urban land in the Lakeview Ave. and Rosemont sections of the city.

Two pumping stations on West St. and Beaver St. will carry sanitary and storm-water flow through the dikes during flood periods.

The estimated cost of construction is $465,000 for construction and $90,000 for lands and damages, or a total of $555,000. The project is now (1945) practically completed. Prior cost to be included is about $800,000 expended for channel improvements under the Emergency Relief Act of 1935.

Lowell has an interesting flood history, which is valuable because of its length, including records back to 1852 and with some information back to 1785. Heads of water upon the Pawtucket Dam at Lowell during the greater floods have been as follows:

Date	Head on Pawtucket Dam, Ft
1785	(A flood of about same magnitude as in 1852)
Apr. 23, 1852	13.6
Apr. 21, 1870	12.7
Apr. 16, 1895	11.5
Mar. 3, 1896	12.8
Nov. 6, 1927	10.6
Mar. 20, 1936	20.0

The flood of Nov. 6, 1927, had a discharge of about 83,000 cfs at Lowell; the flood of Mar. 20, 1936, about 173,000 cfs. The flood of 1852 was rated by James B. Francis, engineer in charge of the water power at Lowell, as the highest in 120 years, so that the 1936 flood was the highest at Lowell in a period of about 200 years of record.

Mr. Francis figures[1] in an interesting bit of flood history at Lowell.

A Tribute to James B. Francis

When in 1848 Mr. Francis—then engineer for the Locks and Canal Co. of Lowell, was constructing the great water power development at that city, he placed near the head of the Pawtucket Canal—one of the principal feeders of the power system, an emergency gate about 27 ft wide and 25 ft high housed and held well above ordinary stages of the canal.

Mr. Francis from his studies of the Merrimack appreciated the probability of disastrous floods—due to combinations of rain and melting snow, and put in this gate for such possible emergencies. It was held up by an iron hook so that it could be quickly dropped when needed.

The people of Lowell scoffed at the probability of ever needing such a device and dubbed it "Francis' Folly."

However, his predictions were soon verified. In 1852 came a great flood and the gate was used—shutting out floodwaters and saving the industrial section of the

[1] Barrows, H. K.: A Tribute to James B. Francis, *Eng. News-Record* Vol. 1, p. 539 (1936).

city through which the canal runs. Then Mr. Francis was given a dinner and presented with a silver service by his grateful fellow citizens.

Since 1852, so far as information is available, this emergency gate has not again been used until the present flood. On Mar. 19, 1936, after 84 years of rest, it again functioned, and once more the city was saved from unparalleled damage.

Thus, as stated in the press, the hero of the flood in Lowell was a hydraulic engineer, whose foresight and good judgment were again vindicated, 40 years after his death.

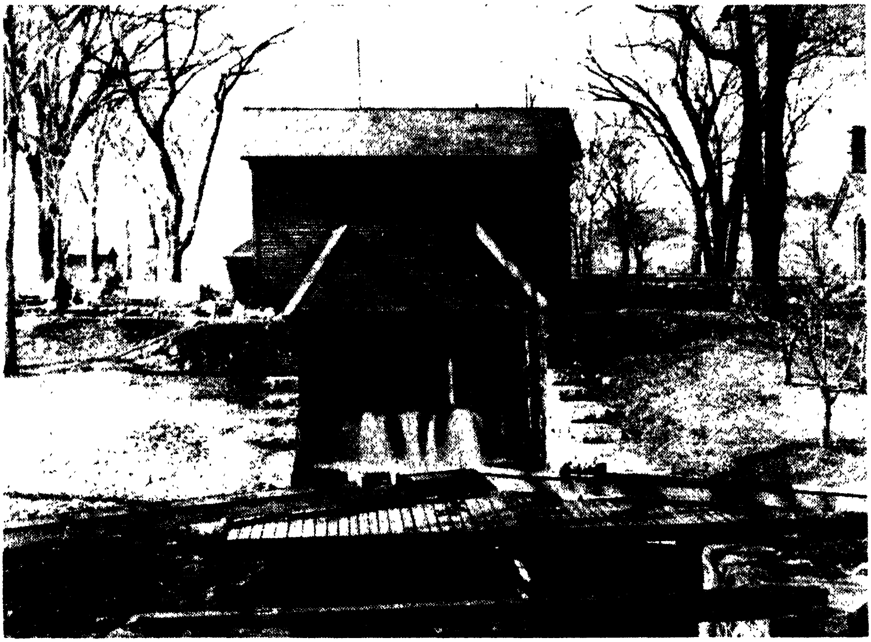


FIG. 18·23. The Francis floodgate at Lowell, Mass.

In Fig. 18·23 is a view of the Francis floodgate at the head of the Pawtucket Canal in Lowell, showing in the background the temporary addition of sandbags to its height, which was required in the 1936 flood. This floodgate is one of the earliest planned local flood-protection works that has come to the notice of the writer.

Characteristics of Local Flood-protection Projects

In Table 18·7 are compiled data of cost and other features of 29 local flood-protection projects scattered widely over the United States. These include drainage area of streams, general type of project, area protected, total and yearly costs at 5½ per cent, cost per acre protected, yearly benefits, and ratios to yearly costs, with summaries as follows:

1. Total cost of 29 projects = $38,609,000
   Average cost of 29 projects = $1,330,000 ($16,640,000 to $18,000)
   Total acres protected = 1,024,120
   Average acres protected = 35,000 (437,000 to 30)
   Cost per acre protected, based upon total = $38 ($2,800 to $8)
   Total yearly costs = $2,115,700
   Per cent of yearly costs = 5.5
   Total yearly benefits = $3,835,800
   Ratio of benefits to cost, based upon totals = 1.8 (5.5 to 1.0)
2. As will be noted, three projects numbered 1, 6, and 7, are very large in cost and acres protected, with the following characteristics:
   Total cost of three projects = $26,800,000
   Average cost of 3 projects = $8,930,000
   Acres protected = 878,000
   Average acres protected = 293,000
   Cost per acre protected, based upon totals = $31
   Total yearly costs = $1,517,000
   Total yearly benefits = $3,031,000
   Ratio of benefits to cost, based upon totals = 2.0
3. The remaining 26 projects have the following:
   Total cost of 26 projects = $11,800,000
   Average cost of 26 projects = $450,000 ($1,953,000 to $18,000)
   Acres protected = 146,000
   Average acres protected = 5,600 (35,000 to 30)
   Cost per acre protected, based upon totals = $81 ($2,800 to $8)
   Total yearly cost = $579,000
   Total yearly benefits = $805,000
   Ratio of benefits to cost, based upon totals = 1.3 (5.5 to 1.0)

It will be noted that, in general, the ratios of yearly benefits to yearly cost vary approximately inversely as the cost per acre protected. This cost varies widely with the amount and character of the development and the area as well as the total cost of the project. Thus the first project in Table 18·7, *viz.*, Augusta-Clarendon upon the White River in Missouri, is high in cost but protects a large area of 437,000 acres. The cost of $8 per acre protected is therefore the lowest of any project.

The highest cost of $2,800 per acre protected, in Table 18·7, Project No.

Table 18·7 Local Flood-protection Projects—Characteristics

River and state	Date of report	Drainage area, sq miles	Project	Area protected, acres	Cost			Yearly benefits	Ratio Col. (9) / Col. (8)
					Total	Per acre protected	Yearly		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1. White River, Mo., Augusta-Clarendon	1937	28,000	Levees	437,000	$ 3,500,000	$ 8	$ 157,000	$ 197,000	1.3
2. Ouachita, Calion, Ark.	1939	6,675	Levees	150	50,000	355	2,800	4,100	1.4
3. Cottonwood Canyon, Cherokee, Okla.	1939	55	Levees and bridges	4,650	803,000	172	29,600	33,000	1.1
4. Sebewaing River, Mich.	1939	105	Channel improvement	800	261,000	326	13,600	17,000	1.25
5. Susquehanna, Pa., Sunbury	1939	18,296	Levees	900	1,953,000	2,180	97,500	93,000	1.0
6. Tennessee, Tenn., Chattanooga, Rossville	1939	21,400	Levees	20,000	16,640,000	800	978,000	1,884,000	1.9
7. Lower Colorado, Tex., LaGrange, etc.	1939	3,450	Levees	421,000	6,700,000	158	382,000	950,000	2.5
8. Autauga Creek, Prattville, Ala.	1940	126	Dam and channel	300	538,000	1,800	20,000	20,000	1.0
9. Touchet, Wash., Dayton	1940	776	Levees	3,000	156,000	52	8,100	9,600	1.2
10. Umpqua, Oreg.	1940	4,550	Levees	3,320	176,000	53	11,700	21,300	1.8
11. Walla Walla, Wash., Milton, etc.	1940	1,763	Levees	5,000	587,000	117	27,000	43,000	1.6
12. Arkansas, Ark., Crawford County Levee District	1940	150,400	Levees strengthened	14,930	301,000	20	115,000	18,100	1.2
13. Rancocas Creek, N. J., Mt. Holly	1940	340	Dikes	230	320,000	1,400	18,000	18,000	1.0
14. Licking, Ky., Salyersville	1941	3,672	Levees	70	186,000	2,600	9,500	12,300	1.3
15. Ventura, Calif.	1941	228	Levees	7,160	1,740,000	244	87,000	7,000	1.0
16. Galena, Ill. and Wis., Galena	1941	207	Levees	700	48,000	2,240	26,400	56,000	2.2
17. Six Mile Creek, Ark.	1942	304	Levees and diversion	10,500	337,000	32	19,000	54,000	2.8
18. Palouse, Wash. and Idaho, Pullman, etc.	1942	2,980	Levees	3,500	518,000	148	22,600	66,400	2.9
19. Coquille, Oreg.	1942	1,070	Levees	1,620	168,000	100	11,700	16,500	1.4
20. Chittenango Creek, N. Y.	1942	326	Channel improvement	7,970	143,000	18	9,600	10,400	1.1
21. San Diego, Calif., San Diego	1942	435	Levees	1,270	370,000	290	16,500	50,000	3.0
22. Conewango Creek, etc., N. Y., Chautauqua	1942	902	Channel improvement	20,000	193,000	10	13,000	71,000	5.5
23. Willapa, Wash.	1942	240	Levees	2,850	140,000	49	7,600	9,500	1.25
24. Short Creek, Ohio, Dillonvale and Adena	1942	148	Levees	230	186,000	800	8,400	9,800	1.2
25. Turkey, Iowa, Elkport	1942	1,696	Levees	30	18,000	570	800	1,900	2.4
26. Nehalem, Oreg., Tillamook County	1942	862	Levees	770	33,000	43	3,700	4,800	1.3
27. Arkansas, Ark., Conway County	1943	155,000	Levees	20,800	520,000	250	33,400	35,800	1.3
28. Red Lake River, Minn., and Clearwater River	1943	5,711	Channel and lake control	35,000	989,000	29	44,000	80,700	1.8
29. Picket Wire, Colo., Trinidad	1944	3,390	Levees	370	1,035,000	2,800	42,200	41,660	1.0
29 projects:									
Total				1,024,120	$38,609,000		$2,115,700	$3,835,800	
Mean				35,000	1,330,000	$ 38	70,000		1.82
26 projects (Omitting 1, 6, and 7):									
Total				146,000	$11,800,000		$ 579,000	805,000	
Mean				5,600	450,000	$ 81	25,000		1.33

29 at Trinidad, Colo., is relatively high in total cost and with an area of only 370 acres.

Comparing the data in Table 18·7 with those of Table 18·5 for the Connecticut River and those of Table 18·6 for the Merrimack River, it will be noted that the costs per acre for the New England projects are much higher, averaging $2,560 per acre for the Connecticut and $4,500 per acre for the Merrimack. This is due to the highly developed, urban character of these areas and the high construction cost as well as the relatively small areas protected. The project at Holyoke, Mass., on the Connecticut River, costing $2,574,000 and protecting 105 acres, or $24,500 per acre, appears to be the most costly, because of unusually difficult construction conditions in a crowded manufacturing district, for a small area.

CHAPTER 19

FEDERAL LEGISLATION AND ECONOMIC ASPECTS OF FLOOD CONTROL

The more important aspects of Federal flood-control legislation since 1935 are here outlined.

1. Act Approved June 22, 1936 (Public Law No. 738, 74th Cong., H.R. Doc. 8455). This was the first legislation recognizing flood control on navigable waters or their tributaries "as a proper activity of the Federal government, in cooperation with states, their political subdivisions and localities thereof" and that destructive floods "constitute a menace to national welfare." Further, "that investigations and improvements of rivers and other waterways, including watersheds thereof, for flood-control purposes are in the interest of the general welfare and that the Federal government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, if the benefits to whomsoever they may accrue are in excess of the estimated costs and if the lives and social security of people are otherwise adversely affected."

Local or state agencies were (1) to provide without cost to the United States all necessary lands, easements, and rights of way, (2) hold and save the United States free from all damages due to construction works, and (3) maintain and operate works after completion under regulations prescribed by the Secretary of War.

The consent of Congress was given to any two or more states to enter into compacts or agreements to be further ratified by Congress in connection with flood-control projects, where streams and their tributaries lie in two or more states, to provide funds for construction, maintenance, purchase of rights of way, lands, etc. It was further provided that penstocks or similar facilities, adapted to possible future use in the development of adequate electric power, may be installed in any authorized dam when approved by the Secretary of War, upon recommendation of the Chief of Engineers.

Under this act, the sum of $310,000,000 was authorized for carrying out the flood-control projects as listed, and provision was also made for examinations and surveys for a large number of additional projects.

2. Act Approved June 28, 1938 (Public Law No. 761, 75th Cong., 3d Sess. H.R. Doc. 10618). This act provided that Federal investigations and improvements for flood control should be under the jurisdiction of and prosecuted by the War Department, under direction of the Secretary of

War and supervision of the Chief of Engineers. A change was also made with reference to the cost of acquiring lands, easements, and rights of way for flood-control projects, providing that states, local agencies, etc., should be reimbursed by the United States for actual expenditures by them for those purposes.

It was further provided that the installation of penstocks or other similar facilities adapted to possible future use in the development of hydroelectric power should require approval by the Secretary of War upon the recommendation of the Chief of Engineers and also the Federal Power Commission.

The sum of $375,000,000 was authorized as an initial amount for flood-control projects for work during the 5-year period ending June 30, 1944, with additional amounts for examinations and surveys by the Departments of War and Agriculture and the Federal Power Commission.

The so-called "Comprehensive Flood Control Plan," including reservoirs, levees, and flood walls, upon which hearings were held by the House Committee on Flood Control as a preliminary to the Act of 1938, included consideration of flood-control projects upon some 13 river systems, including the Ohio, Missouri, and Mississippi, covering much of the United States and totaling approximately $1,000,000,000 in estimated cost.

3. Act Approved Aug. 11, 1939 (Public Law No. 396, 76th Cong., 1st Sess., H.R. Doc 6334). No additional funds were authorized.

4. Act Approved Aug. 18, 1941 (Public Law No. 228, 77th Cong., 1st Sess., H.R. Doc. 4911). Plans for flood control of the Lower Mississippi River were modified by the abandonment of the Boeuf floodway and the Eudora floodway, its northward extension and bank-protection levees.

The sum of $275,000,000 was authorized for flood-control works, with $10,000,000 additional for examinations and surveys and $5,000,000 for water-flow retardation and soil-erosion prevention. The Secretary of War was also authorized to expend an amount not to exceed $1,000,000 in any one fiscal year in flood-rescue work or repairs or maintenance of flood-control works.

5. Act Approved Dec. 22, 1944 (Public Law No. 534, 78th Cong., 2d Sess., H.R. Doc. 4485). The policy of Congress with reference to states rights was defined as follows:

In connection with the exercise of jurisdiction over the rivers of the nation through the construction of works of improvement, for navigation or flood control, as herein authorized, it is hereby declared to be the policy of the Congress to recognize the interests and rights of the states in determining the development of the watersheds within their borders and likewise their interests and rights in water utilization and control, as herein authorized to preserve and protect to the fullest possible extent established and potential uses, for all purposes, of the waters of the nation's rivers; to facilitate the consideration of projects on a basis of comprehensive and coordinated development; and to limit the authorization and construction of navigation works to those in which a substantial benefit to navigation will be realized therefrom and which can be operated consistently with appropriate and economic use of the waters of such rivers by other users.

In conformity with the foregoing policy, plans and investigations for navigation or flood improvements must, before submission to Congress, be referred to the affected state or states for investigation and consultation and to the extent practicable for cooperation. Federal investigations and improvements of rivers for flood control and allied purposes shall be under the jurisdiction of the War Department, and Federal investigations of watersheds and reservoirs for runoff and water-flow retardation and soil-erosion prevention on watersheds shall be under the jurisdiction of the Department of Agriculture. Electric power and energy generated at reservoir projects under the control of the War Department and not required for operating purposes shall be delivered to the Secretary of the Interior for transmission and disposal.

Hereafter, it shall be the duty of the Secretary of War to prescribe regulations for the use of storage allocated for flood control or navigation at all reservoirs constructed wholly or in part with Federal funds provided on the basis of such purposes, and the operation of any such project shall be in accordance with such regulations: Provided, That this section shall not apply to the Tennessee Valley Authority, except that in case of danger from floods on the Lower Ohio and Mississippi Rivers the Tennessee Valley Authority is directed to regulate the release of water from the Tennessee River into the Ohio River in accordance with such instructions as may be issued by the War Department.

Plans for Missouri River flood control, power, and navigation (H.R. Doc. 475, Sen. Doc. 191, and Sen. Doc. 247) were approved, and $200,000,000 was authorized for the partial accomplishment of works under the expanded plans under the Corps of Engineers and $200,000,000 for those under the Secretary of the Interior.

To provide an adequate reservoir of public works for postwar construction, an extensive list of projects was adopted and authorized, and authorization was made for appropriation of $950,000,000 for the War Department, $10,000,000 by the Department of Agriculture and $1,500,000 by the Federal Power Commission.

6. Act Approved July 24, 1946 (Public Law No. 526, 79th Cong., 2d Sess., H.R. Doc. 6597). *Authorized Appropriations.*

$772,000,000	War Department
30,000,000	Department of Agriculture
150,000,000	Missouri River, Department of Interior
$952,000,000	Total

Summary of Authorized Appropriations, Federal Flood Control, 1936-1946, Inclusive

1936			$ 310,000,000	
1938			385,000,000	
1941			290,000,000	
1944	War Department, general	$750,000,000		
	Lower Mississippi River	200,000,000		
	Missouri River, Department of Interior	200,000,000		
	Department of Agriculture	10,000,000		
	Federal Power Commission	1,500,000	$1,161,500,000	
1946	War Department, general	$672,000,000		
	Lower Mississippi River	100,000,000		
	Missouri River, Department of Interior	150,000,000		
	Department of Agriculture	30,000,000	952,000,000	
	Total 1936–1946, inclusive			$3,098,500,000

The entire approved flood-control program of the War Department, exclusive of maintenance and examinations and surveys, as of Jan. 1, 1945, may be summarized as follows:[1]

Flood control, general:		
Flood-control works authorized prior to Flood Control Act of 1944		$ 930,400,000
Flood-control works authorized in Flood Control Act of 1944		750,000,000
Remaining portions of approved basin plans for which further increases in authorizations will be required for completion		1,386,557,000
		$3,066,957,000
Mississippi River:		
Previous project	$125,486,000	
Current project pursuant to Act of May 15, 1928, as amended	862,982,000	988,468,000
Sacramento River project authorized by Act of Mar. 1, 1917, as amended		30,750,000
Total		$4,086,175,000

The total number of flood-control projects including reservoirs and local control works in the foregoing setup is about 600. Numerous additional projects in the various preliminary stages of examination, investigation, and recommendation are not, of course, included as to either number or cost.

The potential postwar flood-control program under date of Feb. 1, 1945, consisting of those works authorized by Congress but not yet completed under Flood Control, General, Mississippi River, and Sacramento River programs, by Army Engineer Divisions, is as follows:[2]

[1] Letter from Chief of Engineers, Feb. 27, 1945.

[2] *Ibid.*

Division	Flood-control Projects Adopted by Congress
New England	$ 89,756,000
North Atlantic	43,082,000
Middle Atlantic	125,372,000
South Atlantic	156,735,000
Lower Mississippi Valley	431,778,000
Southwestern	331,222,000
Upper Mississippi Valley	103,640,000
Missouri River	578,150,000
Ohio River	613,651,000
Great Lakes	24,336,000
Pacific	395,488,000
Territories	73,000
Total	$2,893,283,000

As of June 30, 1946, funds actually appropriated by Congress for flood-control works have totaled $1,479,083,000, or to date about one-third of the entire flood-control program of about $4,100,000,000 has been funded.

Flood Control in the United States—Economic Aspects

Tables 8·6 and 8·7 afford a basis for an approximation of total yearly flood losses in the United States as follows:

The period covered by Table 8·6 is about 32 years—from 1913 to 1945—although three of the smaller floods listed occurred prior to 1913. In this period are included major floods upon the Mississippi and Ohio River basins, with losses totaling about $1,500,000,000 on about 1,300,000 sq miles, or about $1,150 per square mile. The floods in the remaining portion of the United States, taking account of river systems and avoiding duplication, show losses of about $800,000,000 upon about 500,000 sq miles of drainage area, or about $1,600 per square mile. The total loss of $2,300,000,000 averages about $1,270 per square mile and represents approximate flood history upon 1,800,000 sq miles or about 60 per cent of the 3 million sq miles in the entire United States. The areas included in Table 8·6 are obviously those most subject to floods, and a considerable portion of the country in the aggregate does not have the flood problem. On the other hand, the list of floods in Table 8·6 is not complete, particularly for smaller floods.

Hence it will be assumed that, in the entire United States in the 32-year period 1913–1945, total flood losses have aggregated about $3,000,000,000, or a yearly flood loss of nearly $100,000,000 as an average amount. It is perhaps more rational to conclude that yearly flood losses may have averaged from $80,000,000 to $100,000,000. Roughly half of the losses occur in the Mississippi River Basin, including the Ohio, Missouri, and other tributaries, which aggregate about 42 per cent of the area of the United States. On the basis of $100,000,000 average yearly flood losses, this would represent at 5 per cent a limit in capital expenditures of $2,000,000,000 as a rough measure of the limit in economy.

It has already been shown that, in the flood-control program, as constructed and proposed as of Jan. 1, 1945, a total expenditure of over $4,000,000,000 is contemplated, or double the amount of expenditure that appears to be warranted, based upon average yearly losses. Furthermore, the ultimate program of flood-control projects will contain additional projects, now only in tentative form, and total costs will exceed $4,000,000,000, perhaps materially. Only about one-third of this amount has been funded, as noted previously.

The foregoing analysis does not include allowance for benefits obtained, aside from flood control, which incidentally accrue to navigation and power. This applies upon the Mississippi River and its tributaries, which have important navigation facilities, and for some of the large multiple-purpose reservoir projects where power benefits have resulted—particularly in the Southwest where this type of flood control has been more fully utilized.

It is obvious, however, that flood control per se will involve a heavy cost to the nation. Wherever practicable, it should be accomplished by multiple-purpose projects, where better economy may often be obtained and more useful development made of our river systems.

Flood-control Program

In approving the Rivers and Harbors and Flood Control Bills on July 24, 1946, the President included the following statement:

Statement by the President

In Approving H.R. 6407 (Rivers and Harbors Bill)
and H.R. 6597 (Flood Control Bill)
July 24, 1946

I have today approved two bills, H.R. 6407 and H.R. 6597, authorizing the construction of river and harbor improvements. One of these bills is what is generally known as a River and Harbor Bill and authorizes projects primarily for the improvement of navigation, while the other is a Flood Control Bill. Both bills, however, authorize projects that would include other developments of rivers, including irrigation, water supply, and hydroelectric power development. The River and Harbor Bill authorizes projects which are estimated to have an ultimate cost of $945,000,000. The Flood Control Bill authorizes works estimated to cost $952,000,000, of which $772,000,000 is for projects under the jurisdiction of the War Department. These two bills bring the authorized backlog of river-improvement work under jurisdiction of the War Department to approximately five billion dollars. Assuming that this estimate of five billion dollars is accurate, and experience would indicate that it is probably low, and assuming the new work can be prosecuted at the 1947 appropriation rate, it will take 35 years to bring to completion the river and harbor projects and 20 years to complete the flood control projects now authorized.

In consonance with the intent of Congress as indicated in its consideration of these two bills, I take them to be primarily authorizations to enable the War Department to plan its future programs soundly, and I understand that there is no expectation of early appropriations. I do not intend to request funds for any of these projects during the current fiscal year. Financing, whenever made, must be based on budgetary requirements for that period.

Furthermore there are many unanswered questions in connection with the projects authorized by the two bills I have just signed. These questions must be satisfactorily answered before the construction authorized is initiated. I do not intend to approve any requests for appropriations or allocations of funds for the construction of any of those projects until all the important questions concerning them have been satisfactorily resolved, and until all of the Federal agencies directly concerned are substantially agreed upon the technical features involved.

With a shelf of projects that will take us many years to complete, it is obvious that we must give careful consideration to which projects are undertaken first. Accordingly, in connection with the preparation of budget estimates, the program must be reexamined annually so as to determine the present estimated cost and the present economic merits of the projects proposed for inclusion in that year's program.

For some years the majority of these authorized projects must be deferred. As to the more immediate future, I repeat what I said when the Director of the Office of War Mobilization and Reconversion issued his seventh report, that government expenditures will be reduced and deferrable construction and public works projects using Federal funds will be studied with a view to saving strategic materials and to diminishing inflationary pressures.

Concluding

The foregoing statement is of significance and importance in outlining the Federal flood-control program and reviewing its probable cost and scope. A heavy expenditure is contemplated extending over many years, and this is likely to exceed present cost estimates materially unless a rigid future standard of project justification is followed and undesirable projects are ruled out.

The author again accentuates the desirability of utilizing high-degree power storage, with flood relief as a by-product, whenever conditions make this feasible. This can often be done successfully by private corporations, as upon the Wisconsin River in Wisconsin and the Sacandaga River in New York State, described in Chap. 9, and other locations where excellent results have been obtained. The advantage over reservoirs for flood control alone lies both in lower cost and in the conserving of river flow, thus improving as well as protecting our rivers. As a long-time policy, this is of great importance in their comprehensive development, affecting, as it does, our national prosperity.

CHAPTER 20

1947

Manuscript for this book was completed in September, 1946. During the time that has since elapsed, further appropriations for flood-control work have been made and, also, some additional progress in the construction of projects. Floods have also occurred, especially in the Mississippi and Missouri valleys. This chapter has therefore been added to bring flood history up to the fall of 1947.

Appropriations for Flood Control—1947. The 80th Congress in Public Law 296 (H.R. Doc. 4002, 1st Sess., Chap. 411, approved July 31, 1947, appropriated funds for flood-control purposes as follows:

Summary of Flood-control Appropriations—1947

Flood control—general:		
Total construction	$231,800,825	
Advance planning	4,500,000	
Preliminary surveys, etc.	5,000,000	
Salaries, chief's office	497,000	
Emergency repairs (Sec. 5, Act of 1941)	1,000,000	
Transfer to Geological Survey	375,000	
Maintenance of completed works	2,400,000	
Emergency bank protection	500,000	$246,072,825
Mississippi River and tributaries		50,000,000
Emergency fund		500,000
Sacramento River, Calif.		1,750,000
Total		$298,322,825

A list is given at the end of the chapter of the construction projects, with location and cost. It shows most of those actually under way and is a breakdown of the item of $231,800,825 in the preceding table.

This will make a total funded amount to date as follows:

Appropriated to June 30, 1946 (p. 397)	$1,479,083,000
For 1947	298,322,825
Total funded to July 31, 1947	$1,777,405,825

The total cost of $4,086,175,000 for the approved flood-control program, as given in Chap. 19, page 396, remains substantially the same, since the 1947 appropriations will make up a part of the item of $1,386,557,000 entitled "Remaining portions of approved basin plans for which further increases in authorization will be required for completion."

Accordingly, to date, as noted in Chap. 19, page 397, about one-third of the total amount of about $4,100,000,000 for flood-control projects has been funded.

Floods upon Mississippi and Missouri Rivers—June and July, 1947.[1] During June and early July, because of heavy rainfall, Midwest floods occurred upon the Lower Missouri and the Mississippi between Hannibal, Mo., and Cairo, Ill. At St. Louis, the Mississippi on July 2 reached a 103-year high of 40.3 ft, or within about 1 ft of the all-time record of 41.4 ft reached in 1844.

Almost every Midwestern tributary was in flood, and many streams reached heights greater than any previously recorded. Thus the Raccoon River in Iowa reached stages of 22.5 and 21.2 ft at Jefferson and Van Meter, respectively, where previous high crests were 16.2 and 18.8.

The Des Moines River at Des Moines, Iowa, crested at 26.5 ft, or less than 1 ft below the all-time high of 1903. The Missouri River rose progressively from an all-time high of 20.1 ft at Nebraska City, Nebr.; to 28.1 ft at Lexington, Mo.; to 32 ft at Bonneville, Mo., on June 27.

The Grand River reached a high of 33.2 ft at Chillicothe, Mo. The The Blue River went over its banks at Beatrice, Nebr., as did the Platte River at Agency, Mo.

A preliminary estimate of total property damage in the St. Louis district, with an inundation of 213,000 acres, was about $12,000,000.

Total flood damage for the entire Mississippi Basin above Cairo, Ill., exclusive of the Missouri and Illinois Rivers, according to Col. Kittrell, division engineer Upper Mississippi Valley Division, Corps of Engineers, was as follows:

Estimated total tangible damage, exclusive of soil erosion, $62,000,000; 955,000 acres flooded, including 340,000 acres behind protective works of varying degrees of effectiveness—all deficient for this flood; 9 United States highways and 23 state highways blocked; 14 railroad lines interrupted; and two water-supply and four electric-power plants forced out of service. Over 500,000 acres, however, were protected from flooding by existing works, thus preventing an estimated direct property damage of over $120,000,000.

Flood Control, General—1947

Details of Projects

Arkansas:	
Blakeley Mountain Reservoir	$2,000,000
Bull Shoals Reservoir	6,700,000
Cardens Bottoms levee district No. 2	300,000
Conway County levee districts Nos. 1, 2, and 8	400,000

[1] *Eng. News-Record*, July 10 and Aug. 21, 1947.

Crawford County levee district	$ 400,000
Little Rock to Pine Bluff	500,000
McLean Bottom levee district No. 3	652,000
Narrows Reservoir	2,000,000
Near Dardanelle	221,500
Red River below Denison Dam, Okla., Tex., Ark. and La.	2,500,000
Roland drainage district	150,000
Van Buren	210,000
California:	
Big Dry Creek Reservoir and diversion	771,000
Burbank Western system (lower)	100,000
Isabella Reservoir	1,250,000
Kings River and Tulare Lake Basin, including Pine Flat Reservoir	1,750,000
Los Angeles River	4,016,000
Lower San Joaquin River	....
Merced County stream group	300,000
Pajaro River	500,000
Sacramento River levees and tributaries	500,000
San Gabriel River (Canyon to Santa Fe Reservoir)	1,085,000
Santa Fe Reservoir	730,000
Ventura River	500,000
Colorado:	
Cherry Creek Reservoir	4,500,000
John Martin Reservoir	550,000
Connecticut: Hartford	293,000
Georgia:	
Allatoona Reservoir	8,425,000
Clark Hill Reservoir, Ga. and S. C.	5,000,000
Idaho: Heise-Roberts area, Snake River	500,000
Illinois:	
Chouteau, Nameoki, and Venice	130,000
Clear Creek drainage and levee district	200,000
Columbia drainage and levee district	500,000
DeGognia and Fountain Bluff levee and drainage district	1,065,000
East St. Louis and vicinity	750,000
Farm Creek Reservoirs	2,000,000
Galena	465,000
Grand Tower drainage and levee district	500,000
Harrisonville and Ivy Landing drainage and levee district	260,000
Henderson County drainage district	47,000
Lacey, Langellier, West Matanzas, and Kerton valley drainage and levee districts	500,000
Mounds and Mound City	500,000
Mouth of Sangamon River	200,000
Prairie du Rocher	500,000
Preston drainage and levee district and vicinity	323,000

Sny Basin	$1,000,000
Stringtown–Fort Chartres and Ivy Landing drainage and levee district	700,000
Wilson and Wenkel and Prairie DuPont drainage and levee district	478,000
Wood River drainage and levee district	750,000
Indiana:	
Cagles Mill Reservoir	500,000
Cannelton	500,000
Delphi	65,000
Indianapolis, Fall Creek section	750,000
Jeffersonville-Clarksville	209,000
McGinnis levee	300,000
Muncie	60,000
New Albany	400,000
Shufflebarger levee	300,000
Iowa:	
Council Bluffs	500,000
Des Moines	350,000
Dry Run	531,000
Missouri River between Kensler's Bend, Nebr., and the combination bridge at Sioux City, Iowa	800,000
Missouri River levees, Sioux City, Iowa, to the mouth	6,000,000
Red Rock Reservoir (see advance planning.)	
Kansas:	
Fall River Reservoir	3,000,000
Kanopolis Reservoir	2,900,000
Kansas Citys, Mo. and Kans.	5,000,000
Wichita and Valley Centre	500,000
Kentucky:	
Covington	500,000
Dale Hollow Reservoir, Tenn. and Ky.	800,000
Dewey Reservoir	1,500,000
Louisville	1,650,000
Maysville	....
Newport	1,200,000
Taylorsville	240,000
Uniontown	504,000
Wolf Creek Reservoir	8,000,000
Louisiana:	
Bayou Bodeau, Red Chute, and Loggy Bayou	86,500
Bayou Bodeau Reservoir	800,000
Bayou Pierre in vicinity of Shreveport	129,000
Red River below Denison Dam, Okla., Tex., Ark., and La. (see Arkansas)	
Shreveport	1,000,000
Massachusetts:	
Holyoke and Springdale	70,000
Tully Reservoir	726,000

Michigan:	
Mount Clemens	$ 250,000
Sebewaing	100,000
Minnesota: Red Lake and Clearwater Rivers	400,000
Missouri:	
Chariton River	300,000
Clearwater Reservoir	2,000,000
Kansas Citys, Mo. and Kans. (see Kansas)	
Montana: Forsyth	171,000
Nebraska:	
Aten (bank protection)	500,000
Harlan County Reservoir	9,000,000
Indianola	75,000
Missouri River between Kensler's Bend, Nebr., and the combination bridge at Sioux City, Iowa (see Iowa)	
Omaha	1,500,000
New Hampshire: West Peterboro	500,000
New York:	
Almond Reservoir	1,600,000
East Sidney Reservoir	900,000
Elmira	1,900,000
Lancaster	230,000
Montour Falls	820,000
Mount Morris Reservoir	2,000,000
Olean	500,000
Owasco inlet and outlet and tributaries	235,500
Syracuse	500,000
North Carolina:	
Buggs Island Reservoir, Va. and N.C.	4,800,000
Goldsboro	85,200
North Dakota:	
Baldhill Reservoir	900,000
Garrison Reservoir	20,105,625
Park River Reservoir	300,000
Ohio:	
Burr Oak Reservoir	600,000
Cincinnati	1,000,000
Delaware Reservoir	3,200,000
Dillon Reservoir	2,000,000
Portsmouth–New Boston	2,000,000
West Fork Mill Creek Reservoir	500,000
Oklahoma:	
Canton Reservoir	2,200,000
Fort Gibson Reservoir	5,000,000

Hulah Reservoir	$1,000,000
Jenks	107,000
Polecat Creek	1,000,000
Red River below Denison Dam, Okla., Tex., Ark. and La. (see Arkansas)	
Tenkiller Ferry Reservoir	2,000,000
Wister Reservoir	2,950,000
Oregon:	
Arlington	156,000
Coquille River	131,000
Detroit Reservoir	1,250,000
Dorena Reservoir	3,750,000
Amazon Creek	305,000
Fern Ridge Reservoir	20,000
Lookout Point Reservoir	4,000,000
Mill Four district	116,000
Nehalem River	23,000
Pudding River	110,000
Umpqua River and tributaries	194,000
Willamette River (bank protection)	500,000
Pennsylvania:	
Conemaugh River Reservoir	5,200,000
East Branch, Clarion River Reservoir	1,000,000
Punxsutawney	400,000
Sunbury	1,369,000
Williamsport	2,236,000
South Carolina: Clark Hill Reservoir, Ga. and S.C. (see Georgia)	
South Dakota:	
Fort Randall Reservoir	9,000,000
Hot Springs	200,000
Oahe Reservoir (see advance planning)	
Tennessee:	
Center Hill Reservoir	6,500,000
Dale Hollow Reservoir, Tenn. and Ky. (see Ky.)	
Memphis	1,052,000
Texas:	
Buffalo Bayou	1,854,700
Hords Creek Reservoir	1,303,800
Red River below Denison Dam, Okla., Tex., Ark., and La. (see Arkansas)	
San Angelo Reservoir and floodway	1,000,000
Trinity River:	
Benbrook Reservoir	1,600,000
Lavon Reservoir	500,000
Grapevine Reservoir	500,000
Whitney Reservoir	5,200,000
Utah: Redmond and vicinity	583,000

Vermont: Union Village Reservoir	$ 100,000
Virginia:	
Buggs Island Reservoir (see North Carolina)	
Philpott Reservoir	250,000
Washington:	
Mud Mountain Reservoir	448,000
Tacoma	1,350,000
Yakima	202,000
West Virginia:	
Bluestone Reservoir	5,700,000
Elkins	628,000
Parkersburg	1,500,000
Point Pleasant	600,000
Total construction	$231,800,825

APPENDIX A

DISCHARGE OF RIVERS OF UNITED STATES

(From records of U.S. Geological Survey)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm		
			Avg. (4)	Max. (5)	Min. (6)
1. North Atlantic New England:					
St. John	Fort Kent, Me.	5,690	1.68	21.4	0.11
Penobscot	West Enfield, Me.	6,600	1.75	23.4	s0.22
Kennebec	Bingham, Me.	2,710	1.52	20.0	s0.28
Androscoggin	Rumford, Me.	2,067	1.72	36.0	s0.65
Saco	Cornish, Me.	1,298	2.02	39.5	0.070
Merrimack	Lowell, Mass.	4,635	1.50	37.0	s0.043
Connecticut	South Newbury, Vt.	2,825	1.76	27.6	0.079
	Thompsonville, Conn.	9,661	1.63	29.4	s0.13
Housatonic	Stevenson, Conn.	1,545	1.59	45.0	0.0000
		34,206	(1.62)		
Middle Atlantic					
Hudson	Gooley, N.Y.	419	2.00	33.4	0.10
	Mechanicville, N.Y.	4,500	1.64	26.8	0.000
Passaic	Paterson, N.J.	785	1.85	35.8	0.000
Delaware	Trenton, N.J.	6,780	1.71	34.0	0.18
Susquehanna	Marietta, Pa.	25,990	1.32	30.8	0.053
Potomac	Washington, D.C.	11,560	0.96	41.5	0.038
Rappahannock	Fredericksburg, Va.	1,599	1.03	87.0	0.003
		51,214	(1.28)		
		85,420	(1.43)		
2. South Atlantic and Eastern Gulf of Mexico:					
James	Cartersville, Va.	6,242	1.17	24.0	0.050
Roanoke	Clover, Va.	3,230	1.00	17.5	0.062
Tar	Nashville, N.C.	701	0.88	24.0	0.014
Neuse	Clayton, N.C.	1,140	1.18	24.6	0.040
Cape Fear	Fayetteville, N.C.	4,370	1.10	25.0	0.017
Pee Dee	Rockingham, N.C.	6,870	1.23	31.0	0.024
Santee	Ferguson, N.C.	14,800	1.28	24.8	0.17
Oconee	Dublin, Ga.	4,400	1.18	21.8	0.12
Suwannee	Ellaville, Fla.	6,580	0.98	11.0	0.15
Chattahoochee	West Point, Ga.	3,550	1.65	37.8	0.063
Apalachicola	River Junction, Fla.	17,100	1.28	17.2	0.30
Coosa	Wetumpka, Ala.	10,200	1.63	29.2	0.053
Alabama	Montgomery, Ala.	15,100	1.59	13.8	0.34
Tallapoosa	Tallassee, Ala.	3,320	1.49	35.0	0.003
Tombigbee	Columbus, Miss.	4,490	1.32	19.0	0.044
	Leroy, Ala.	19,100	1.42	10.0	
Pearl	Columbia, Miss.	5,690	1.25	13.0	0.13
		122,193	(1.28)		

Appendix A (continued)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm Avg. (4)	Max. (5)	Min. (6)
4. St. Lawrence:					
Niagara	Buffalo, N.Y.	263,500	0.72	0.92	0.44
St. Lawrence	Ogdensburg, N.Y.	298,100	0.74	0.99	0.51
Fox	Berlin, Wis.	1,430	0.69	4.48	0.17
	Wrightstown, Wis.	6,150	0.70	3.34	0.022
Wolf	New London, Wis.	2,240	0.82	6.90	0.11
Muskegon	Newaygo, Mich.	2,350	0.71	4.95	0.064
Black	Port Huron, Mich.	634	0.39	19.4	0.007
Maumee	Waterville, Ohio	6,314	0.64	7.84	0.005
Auglaize	Defiance, Ohio	2,329	0.67	16.7	0.003
Genesee	Rochester, N.Y.	2,467	1.07	14.4	0.009
Black	Watertown, N.Y.	1,876	2.02	18.0	0.007
Moose	McKeever, N.Y.	365	2.27	45.0	0.11
Oswegatchie	Heuvelton, N.Y.	973	1.70	16.0	0.20
Raquette	Piercefield, N.Y.	722	1.73	10.4	0.014
Ausable	Ausable Forks, N.Y.	448	1.47	54.0	0.000
Otter Creek	Middlebury, Vt.	628	1.38	17.6	0.15
Winooski	Essex Junction, Vt.	1,044	1.52	108	0.066
Lamoille	East Georgia, Vt.	686	1.74	120	0.090
Missisquoi	Richford, Vt.	479	1.72	94.0	0.020
		303,080	(0.75)		
3. Ohio:					
Ohio	Sewickley, Pa.	19,500	1.60	29.6	0.10
	Huntington, W.Va.	55,900	1.4	11.7	0.057
	Louisville, Ky.	91,170	1.2	12.2	0.045
	Metropolis, Ill.	203,000	1.4	8.88	0.10
Muskingum	McConnellsville, Ohio	7,411	0.96	36.2	0.030
Kanawha	Kanawha Falls, W.Va.	8,367	1.52	38.0	0.077
Levisa Fork	Paintsville, Ky.	2,143	0.98	32.6	0.003
Scioto	Higby, Ohio	5,129	0.74	34.5	0.003
Miami	Hamilton, Ohio	3,639	0.91	96.0	0.027
Kentucky	Lockport, Ky.	6,160	1.35	16.2	0.005
Green	Livermore, Ky.	7,580	1.31	27.4	0.037
Wabash	Mt. Carmel, Ill.	28,600	0.86	15.2	0.056
Cumberland	Clarksville, Tenn.	16,000	1.45	18.2	0.020
Tennessee	Chattanooga, Tenn.	21,400	1.75	19.3	0.15
	Johnsonville, Tenn.	38,520	1.60	12.0	0.092
		203,000	(1.40)		

Appendix A (continued)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm Avg. (4)	Max. (5)	Min. (6)
5. Upper Mississippi and Hudson Bay					
Mississippi:					
Mississippi	Royalton, Miss.	11,600	0.23	1.65	0.022
	St. Paul, Minn.	36,800	0.24	2.18	0.017
	LaCrosse, Wis.	62,008	0.35	2.00	0.052
	Keokuk, Iowa	119,000	0.51	2.64	0.042
Crow	Rockford, Minn.	2,520	0.18	4.25	0.0001
Minnesota	Mankato, Minn.	14,900	0.14	3.00	0.0002
St. Croix	St. Croix Falls, Wis.	5,930	0.60	6.00	0.0000
Chippewa	Durand, Wis.	9,010	0.81	10.4	0.15
Black	Neillsville, Wis.	2,120	0.84	23.0	0.0003
Wisconsin	Merrill, Wis.	2,780	0.96	17.8	0.0030
	Muscoda, Wis.	10,300	0.85	7.80	0.20
Rock	Como, Ill.	8,700	0.60	5.80	0.0051
Iowa	Wapello, Iowa	12,480	0.46	4.50	0.0032
Skunk	Augusta, Iowa	4,290	0.46	10.3	0.0002
Des Moines	Keosauqua, Iowa	13,900	0.34	6.90	1.0003
		119,000	(0.51)		
Hudson Bay:					
Red River of the North	Fargo, N.D.	6,840	0.0060	1.12	0.0010
	Grand Forks, N.D.	30,100	0.0070	1.45	0.0000
	Emerson, Man.	40,200	0.0048	1.15	0.0000
Red Lake	Crookston, Minn.	5,330	0.16	2.80	0.0030
Souris	Minot, N.D.	11,300	0.010	1.05	0.0004
		30,100	(0.007)		
		149,100	(0.41)		
6. Missouri:					
Missouri	Fort Benton, Mont.	24,600	0.33	5.70	0.025
	Fort Peck Dam, Mont.	57,800	0.10	0.57	*s*
	Bismarck, N.D.	186,400	0.10	1.50	0.010
	Omaha, Nebr.	322,800	0.082	0.62	0.0067
	Hermann, Mo.	528,200	0.12	1.10	0.0080
Yellowstone	Yellowstone Park	1,010	1.18	7.40	0.20
	Sidney, Mont.	67,450	0.16	1.90	0.012
Bighorn	Kane, Wyo.	15,900	0.14	1.59	0.011
Little Missouri	Watford City, N.D.	8,490	0.068	3.90	0.000
Cheyenne	Eagle Butte, S.D.	24,500	0.037	4.25	0.000
Belle Fourche	Elm Springs, S.D.	7,210	0.052	4.94	0.000

s = storage regulation.

Appendix A (continued)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm Avg. (4)	Max. (5)	Min. (6)
6. Missouri: (continued)					
Niobrara	Spencer, Nebr.	10,800	0.11	2.00	0.000
James	Scotland, S.D.	21,550	0.010	0.50	0.000
North Platte	Saratoga, Wyo.	2,880	0.45	6.25	0.013
South Platte	Henderson, Colo.	4,740	0.070	2.26	0.001
	Julesburg, Colo.	22,800	0.022	1.37	0.000
Platte	Agency, Mo.	1,760	0.42	21.8	0.000
Elkhorn	Waterloo, Nebr.	6,900	0.12	14.5	0.007
Nishnabotna	Hamburg, Iowa	2,800	0.25	8.80	0.002
Republican	Max, Nebr.	7,740	0.024	24.5	0.000
	Clay Center, Kans.	24,570	0.048	7.90	0.000
Kansas	Wamego, Kans.	55,240	0.068	3.21	0.000
Arikaree	Haigler, Nebr.	1,600	0.021	31.2	0.000
Smoky Hill	Lindsborg, Kans.	8,110	0.037	47.0	0.001
Big Blue	Randolph, Kans.	9,100	0.14	10.8	0.003
Grand	Gallatin, Mo.	2,250	0.45	25.8	0.001
Osage	Bagnell, Mo.	14,000	0.69	15.7	0.021
		528,200	(0.12)		
7. Lower Mississippi:					
Mississippi	St. Louis, Mo.	701,000	0.22	1.85	0.038
	Memphis, Tenn.	932,800	0.46	2.12	0.085
	Vicksburg, Miss.	1,144,500	0.46	1.82	0.085
Meramec	Eureka, Mo.	3,788	0.78	18.2	0.05
St. Francis	Patterson, Mo.	956	1.10	8.40	0.007
White	De Valls Bluff, Ark.	23,430	1.00	6.40	0.14
Arkansas	Canon City, Colo.	3,090	0.23	0.21	0.027
	La Junta, Colo.	12,200	0.023	16.5	0.000
	Holly, Colo.	25,000	0.015	5.40	0.000
	Arkansas City, Kans.	41,600	0.0020	2.00	
	Little Rock, Ark.	157,900	0.25	3.38	0.0005
Yazoo	Greenwood, Miss.	4,450	1.15	9.80	0.072
Red	Colbert, Okla.	38,330	0.14	5.30	0.0003
	Shreveport, La.	59,300	0.40	3.20	0.013
		1,244,000	(0.46)		
8. Western Gulf of Mexico:					
Sabine	Ruliff, Tex.	9,440	0.56	8.12	0.036
Neches	Evadale, Tex.	7,908	0.80	10.5	0.016
Angelina	Horger, Tex.	3,435	0.88	23.6	0.003
Trinity	Dallas, Tex.	6,001	0.25	30.7	0.001
	Romayor, Tex.	17,190	0.45	6.50	0.008

s = storage regulation.

Appendix A (continued)

River	Station	Drainage area, sq miles	Discharge, csm		
			Avg.	Max.	Min.
(1)	(2)	(3)	(4)	(5)	(6)
8. Western Gulf of Mexico: (continued)					
Brazos	Seymour, Tex.	14,490	0.036	6.60	0.0000
	Richmond, Tex.	44,050	0.18	2.68	0.0013
Colorado	Ballinger, Tex.	16,840	0.025	4.70	0.0000
	Columbus, Tex.	40,840	0.083	3.77	0.0002
Concho	Paint Rock, Tex.	5,538	0.048	5.40	0.0010
Neuces	Three Rivers, Tex.	15,500	0.055	5.50	0.0000
Rio Grande	Del Norte, Colo.	1,320	0.72	13.6	0.068
	Labatos, Colo.	7,700	0.10	0.81	0.0000
	San Felipe, N.M.	16,100	0.11	2.60	0.0000
	San Marcial, N.M.	27,700	0.058	2.00	0.0000
		171,592	(0.21)		
9. Colorado:					
Colorado	Granby, Colo.	322	1.01	12.3	0.063
	Cameo, Colo.	8,055	0.49	4.48	0.085
	Cisco, Utah	24,100	0.34	3.20	0.023
	Lees Ferry, Ariz.	107,900	0.16	2.80	0.011
	Parker Dam, Ariz.-Calif.	178,800	0.092	0.24	*s*0.008
Gunnison	Grand Junction, Colo.	8,020	0.35	4.45	0.013
Green	Daniel, Wyo.	468	1.04	16.50	0.007
	Linwood, Utah	14,300	0.12	1.08	0.013
	Green River, Utah	40,600	0.17	1.68	0.006
San Juan	Rosa, N.M.	1,990	0.63	8.70	0.004
	Farmington, N.M.	6,580	0.40	2.70	0.004
	Bluff, Utah	23,000	0.13	3.30	0.000
Paria	Lees Ferry, Ariz.	1,570	0.022	10.2	0.000
Little Colorado	Grand Falls, Ariz.	21,200	0.014	10.0	0.000
Virgin	Littlefield, Ariz.	5,090	0.055	4.30	0.010
Gila	Red Rock, N.M.	2,860	0.066	......	0.002
	Coolidge Dam, Ariz.	12,890	0.049	10.0	0.000
		180,000	(0.10)		
10. Great Basin:					
Bear	Alexander, Idaho	3,840	0.35	1.20	0.0074
Weber	Oakley, Utah	163	1.41	24.8	0.10
	Gateway, Utah	1,610	0.36	4.96	0.028
Jordan	Lehi, Utah	2,960	0.12	0.47	0.000
Sevier	Juab, Utah	5,120	0.049	0.42	0.000
Beaver	Minersville, Utah	512	0.080	1.43	0.001
Carson	Fort Churchill, Nev.	1,450	0.25	4.35	0.000
Humboldt	Palisade, Nev.	5,010	0.070	1.25	0.000
		20,502	(0.16)		

s = storage regulation.

Appendix A (continued)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm Avg. (4)	Max. (5)	Min. (6)
13. Snake:					
Snake	Moran, Wyo.	816	1.73	18.7	0.000
	Boise, Idaho	5,740	1.17	8.75	s0.21
	Murphy, Idaho	41,900	0.25	1.12	0.10
	Clarkston, Wash.	103,200	0.46	2.60	0.10
Henrys Fork	Ashton, Idaho	1,030	1.21	6.03	s0.43
	Rexburg, Idaho	3,010	0.63	3.05	s0.060
Big Wood	Hailey, Idaho	640	0.45	7.00	0.002
Owyhee	Owyhee Reservoir, Oreg.	10,400	0.060	1.55	0.010
Boise	Arrowrock, Idaho	2,220	1.00	8.50	0.001
Malheur	Drewsey, Oreg.	982	0.16	4.35	0.000
	Hope, Oreg.	3,030	0.075	2.68	0.001
Fayette	Horseshoe Bend, Idaho	2,230	1.32	10.0	0.16
Weiser	Wesier, Idaho	1,160	0.73	14.6	0.006
Powder	Robinette, Oreg.	1,710	0.70	2.44	0.010
Salmon	Stanley, Idaho	535	1.10	9.50	0.18
	Salmon, Idaho	3,600	0.50	4.60	0.0067
	Whitebird, Idaho	13,400	0.78	6.60	0.12
Grande Ronde	Rondowa, Oreg.	2,555	0.75	8.80	0.090
Clearwater	Kamiah, Idaho	4,850	1.63	16.8	0.042
	Spalding, Idaho	9,570	1.43	18.0	0.053
		103,200	(0.46)		
12, 14. Columbia and North Pacific:					
Columbia	Birchbank, B.C.	34,000	2.03	9.20	0.26
	Grand Coulee Dam, Wash.	74,100	1.41	6.65	s0.30
	The Dalles, Oreg.	237,000	0.82	4.95	0.15
Umatilla	Umatilla, Oreg.	2,290	0.22	8.60	0.000
John Day	McDonald Ferry, Oreg.	7,580	0.25	3.33	0.0001
Deschutes	Biggs, Oreg.	10,500	5.45	4.15	s0.32
Sandy	Bull Run, Oreg.	440	4.90	132	0.27
Willamette	Salem, Oreg.	7,280	2.96	69.0	0.34
Cowlitz	Castle Rock, Wash.	2,240	3.71	62.0	0.45
Umpqua	Elkton, Oreg.	3,680	1.90	47.0	0.17
Rogue	Central Point, Oreg.	2,020	1.35	45.0	0.30
Puyallup	Puyallup, Wash.	948	3.35	61.0	0.37
Snoqualmie	Tolt, Wash.	605	5.75	85	0.66
Skagit	Concrete, Wash.	2,700	5.05	55.0	s0.97
Kootenai	Newgate, B.C.	7,660	1.20	11.0	0.13
	Copeland, Idaho	13,400	1.01	6.7	0.10
Kettle	Laurier, Wash.	3,800	0.68	7.2	
Spokane	Spokane, Wash.	4,350	1.53	11.2	0.21
Okanogan	Tonasket, Wash.	7,250	0.37	3.50	0.017
Wenatchee	Peshastin, Wash.	1,000	2.68	20.4	0.18
Yakima	Cle Elum, Wash.	500	3.81	51.2	0.13
		246,950	(0.95)		

s = storage regulation.

Appendix A (continued)

River (1)	Station (2)	Drainage area, sq miles (3)	Discharge, csm Avg. (4)	Max. (5)	Min. (6)
11. California:					
Santa Ana	Mentone	189	0.50	276	0.000
San Gabriel	Azusa	211	0.57	310	0.000
Los Angeles	Downey	614	0.18	132	0.000
Santa Clara	Saugus	355	0.056	66.0	0.000
Ballona Creek	Culver City	111	0.34	172	0.000
Santa Ynez	Lompoc	790	0.30	41.0	0.000
Salinas	Spreckels	4,231	0.17	18.0	0.000
Kern	Bakersfield	2,420	0.41	9.0	0.023
San Joaquin	Friant	1,675	1.42	46.0	0.003
	Vernalis	14,010	0.40	3.68	0.013
Tuolumne	La Grange	1,540	1.68	40.0	0.000
Sacramento	Red Bluff	9,300	1.33	31.2	0.21
Pitt	Big Bend	4,920	0.58	6.8	s0.010
Feather	Oroville	3,611	1.63	52.0	0.083
American	Fair Oaks	1,921	1.94	73.5	0.002
Eel	Scotia	3,070	2.02	112	s0.0017
Klamath	Somesbar	8,480	0.77	8.80	0.026
Smith	Crescent City	613	5.30	147	0.27
		54,386	(0.98)		

s = storage regulation.

APPENDIX B

FORM FOR SUMMARY OF RESULTS OF FLOOD-DAMAGE SURVEYS

1. Losses and damages subject to direct valuation		
Industrial business, commercial, and institutional		$
Residential		$
Farms		
Buildings and contents, other structures, furnishings, supplies, and equipment	$	
Livestock	$	
Land and crops	$	
Levee and drainage district improvement	$	$
Miscellaneous private property (automobiles, boats, etc.)		$
Public-service facilities:		
Water-supply systems	$	
Gas- and electric-supply system	$	
Telephone and telegraph systems	$	
Sewerage systems	$	
Railroad, electric street railways, and interurban lines	$	
City streets and bridges	$	
Highways and bridges	$	
River-navigation facilities	$	$
Public property (not included above):		
Federal	$	
State	$	
County	$	
Municipal	$	$
Emergency relief costs:		
Federal	$	
State	$	
County and municipal	$	
Red Cross	$	
Other agencies	$	$
Total		$
2. Loss due to suspension of business		$
3. Depreciation in property value[1]		$
Grand total		$

4. Number of lives lost as a direct result of the flood
5. General
   a. Population affected:
      (1) Total in flood plain
      (2) Total, including population outside of flood plain, directly affected by interruption of gas, water, and electricity supply, or by business suspension

[1] Depreciation should not be included in the items of flood losses as a part of the total loss. This is explained upon p. 153 *et seq.*, where it is shown that depreciation losses are basically a result of direct and indirect losses that have already been included in the tabulation of losses. To include depreciation would therefore result in a duplication of estimated losses.

b. Number of persons forced to evacuate
c. Number of homes lost or rendered uninhabitable and number of persons affected thereby
d. Valuation of property affected—land and improvements:
(1) Urban
(2) Rural

Flood Damage—Form No. 1—Industrial Concerns, Business and Commercial Concerns, Institutions

Name or owner of establishment
Location
Nature of business
River stages
  Gauge
  Damage stage
  Flood crest
Value of property affected
  Land
  Improvements
Property loss and damages
  Buildings and other improvements
  Grounds
  Machinery, plant equipment
  Inventory
  Miscellaneous
  Cost of rehabilitation
  Emergency expense
  Total
Loss due to suspension of operations, based upon normal operation
    Days lost
    Wages unpaid
    Business lost
  Estimated net economic loss
    Wages
      Percentage of unpaid wages
      Amount
    Total business
      Percentage of business lost
      Amount
Remarks

Flood Damage—Form No. 2—Residential Property

Zone
River stages
  Gauge
  Damage stage
  Flood crest
Class of property
Number of units

Value of property affected
  Land
  Improvements
Representative damage per unit
  Buildings, furnishings, and fixtures
  Personal goods
  Grounds
  Total
Total damage for class
Remarks

Flood Damage—Form No. 3—Farm Property

Locality
River stages
  Gauge
  Damage stage
  Flood crest
Number of farms
Area (acres)
Value of property affected
  Land
  Improvements
Loss and damage
  Buildings and other structures
  Furnishings, supplies, and equipment
  Stored crops
  Growing crops
  Livestock
  Land
  Levees and drainage improvements
Total
Remarks

Flood Damage—Form No. 4—Miscellaneous Private Property

Locality of loss or damage
Nature of property
Number of units
Average loss per unit
Total
Remarks

Flood Damage—Form No. 5—Federal, State, County, and Municipal Loss and Damage

Locality
Agency and item
River stages
  Gauge
  Damage stage
  Flood crest

Loss and damage
- Buildings and contents
- Plant and equipment
- Supplies
- Roads and streets
- Bridges
- Sewerage and drainage systems
- Miscellaneous
- Total

Remarks

Flood Damage—Form No. 6—Waterworks

Ownership and location
River stage
- Gauge
- Flood crest

Pumping suspended
- Days
- River stage

Treatment suspended
- Days
- River stage

Direct loss and damage
- Pumping and treatment plant
- Other buildings and contents
- Meters and other equipment
- Mains
- Other property
- Emergency expense
- Total

Wages lost
Business lost
Remarks

Flood Damage—Form No. 7—Gas, Electric, Telephone, and Telegraph Utilities

Utility
Locality of damage
River stage
- Gauge
- Flood crest

Type of power plant
Service interrupted
- Days
- River stage

Direct loss and damage
- Plant
- Other buildings and contents
- Meters, transformers, and other equipment
- Lines or mains
- Other property

Emergency expense
Total
Wages lost
Business lost
Remarks

Flood Damage—Form No. 8—Railroad, Street Railways, and Interurban Lines

Name of line
Locality of damage
River stage
Gauge
Flood crest
Service interrupted
Days
River stage
Direct loss and damage
Buildings and contents
Roadway and track
Bridges and culverts
Rolling stock
Commodities in transit
Other property
Emergency expense
Total
Wages lost
Business lost
Remarks

Flood Damage—Form No. 9—Emergency Relief Expenditures

Locality of operations
Agency
Expenditures
Rescue work
Shelter
Clothing, food, and other supplies
Welfare
Maintenance of protection works
Unclassified
Total
Remarks

APPENDIX C

PROBLEMS

1. Fuller's Formula for Flood Flow. The maximum 24-hr floods in cfs on the Lamoille River at Johnson, Vt., where the drainage area is 310 sq miles, for the years 1932 to 1941, inclusive, were as follows: 4,840, 7,400, 5,840, 4,680, 10,700, 6,300, 6,890, 6,890, 9,930, 4,840. Using Fuller's formula, determine the maximum crest floods to be expected at Clarks Falls, where the drainage area is 726 sq miles for periods of 100, 500, and 1,000 years.

2. Boston Society of Civil Engineers Formula for Flood Flow. The crest flow of the Lamoille River at Milton, Vt., where the drainage area is 723 sq miles, was 23,200 cfs during the 1936 flood. The total flood period obtained from hydrographs is 6.6 days. Determine the flood coefficient for the flood formula developed by the Committee on Floods of the Boston Society of Civil Engineers. Determine the peak runoff in cfs at Milton for a flood runoff of 5.31 in.

3. Distribution Graph of Flood Flow. The following runoff data in cfs were observed on a river for an isolated 1-day storm occurring on Sept. 15.

Day	Runoff, cfs	Day	Runoff, cfs
Sept. 14	1,600	Sept. 18	4,900
15	4,200	19	3,100
16	14,300	20	2,100
17	7,600	21	1,600

Assuming that the ground-water flow was constant for the period, determine and plot the distribution graph for this storm.

4. Spillway Rise with Reservoir Control. The proposed reservoir at Westford, Vt., on Browns River will have a water area at spillway level (El. 475) of 2,000 acres. The spillway is to be 100 ft long with values of C as follows:

H, Ft	C
0–1	3.3
1–3	3.5
3–5	3.7
5–8	3.9
Above 8	4.0

A flood causes an inflow to this reservoir with a peak flow of 16,000 cfs and has a distribution of flow during the flood day according to the hydrograph used for Vermont streams as follows:[1] Starting at the beginning of the flood period with a flow equal to 2 per cent of the peak flow, rising uniformly to 100 per cent at the twelfth hour, and then decreasing uniformly to 20 per cent of the peak flow at the thirty-sixth hour.

Determine the maximum water-surface elevation for this flood, assuming water at

[1] BARROWS, H. K.: "Water Power Engineering," 3d ed., p. 197.

spillway elevation at the beginning of the flood. Use the mass-curve method, with the following reservoir capacities.

Range of Depth on Spillway, Ft	Capacity of Section, Acre-ft
0.0–0.5	1,080
0.5–1.0	1,090
1.0–2.0	2,190
2.0–3.0	2,220
3.0–4.0	2,270
4.0–5.0	2,320
5.0–6.0	2,390
6.0–7.0	2,460
7.0–8.0	2,530
8.0–9.0	2,600
9.0–10.0	2,670

Also determine the maximum spillway discharge and the number of hours between the start and the time that the reservoir reaches its maximum elevation.

5. Valley Storage—Delaware River at Riegelsville—September, 1938. In *Water Supply Paper* 867, page 332, are given data of hourly discharge of the Delaware River at Riegelsville, N.J., during the so-called "hurricane flood" of Sept. 18–23, 1938.

1. Plot the hydrograph of this flood for the period Sept. 20–28, using for each day discharges at 12 A.M., 6 A.M., 12 M., and 6 P.M.

2. Plot the curve of valley storage against discharge, taken from the descending limb of the hydrograph according to the method given in Chap. 5, page 57.

3. Plot and check the curve of valley inflow as explained in Chap. 5, and determine the total amount of runoff and valley inflow (which should be approximately the same), and of valley storage, all expressed in inches depth. Determine also the ratio of maximum valley inflow to maximum runoff.

6. Effect of Storage upon Willamette River. The Willamette River at Albany, Oreg. (4,840 sq miles), showed daily discharges as follows during the period Dec. 22, 1942, to Jan. 12, 1943 (*Water Supply Paper* 984, page 92).

Day	Discharge, cfs	Day	Discharge, cfs
Dec. 22	25,600	Jan. 1	157,000
23	28,800	2	210,000
24	38,200	3	155,000
25	41,700	4	105,000
26	50,700	5	73,600
27	52,300	6	57,400
28	61,500	7	48,100
29	77,200	8	40,600
30	98,300	9	34,800
31	92,000	10	30,600
		11	27,100
		12	24,600

Proposed storage reservoirs for flood control in the Willamette River above Albany include Lookout Point, Dorena, Cottage Grove, Quartz Creek, and Fern Ridge. See Fig. 15·1 for location of these reservoirs and Table 15·2 for drainage areas controlled and storage capacities.

Assume that full storage capacity is available except at Lookout Point and Quartz Creek power-use reservoirs, where half the storage capacity may be assumed available.

1. Make a tabular study as is done for the Connecticut River in Chap. 8, page 151, to determine the effect of storage in reducing the flood peak of 210,000 cfs on Jan. 2 and to ascertain the percentage of flood regulation.

2. Determine approximate gauge stages in 1943 with and without storage effect, making use of the following skeleton discharge rating at Albany:

Gauge Height, Ft	Discharge, Cfs
6	37,000
9	52,000
12	70,000
15	92,400
19	131,000
23	179,000
27	235,000
31	297,000

3. Make an approximation of reservoir effect as to both discharge and stage, with storage assumptions as in (1), for the flood of 1861, for which the peak discharge at Albany, based upon flood marks, was estimated at about 500,000 cfs.

7. Muskingum River Flood Control. Appended are maximum stages and discharges for the Muskingum River, Ohio; also data of direct flood damages in March, 1913, and August, 1935.

1. Construct a discharge-rating curve for Coshocton supplementing the given data with the appended skeleton rating curve taken from *Water Supply Paper* 873, page 112.

2. Construct a discharge-damage curve using data of direct damages in 1913 and 1935

Muskingum River—Maximum Stages and Discharge at Coshocton (4,847 Sq Miles), 1884–1936

Date	Max. stage, ft	Max. discharge, cfs
Mar. 26, 1913	30.5	202,000
March, 1898	26.4	
February, 1884	25.4	
Aug. 8, 1935	24.65	88,500
1910	23.8	
1907	23.7	
Feb. 28, 1936	20.8	
Mar. 22, 1927	20.1	
Feb. 27, 1929	20.0	
Jan. 23, 1927	19.2	

and further assuming the stage of immaterial damage at about gauge height 20 on the Coshocton gauge.

3. Construct the upper portion of a discharge-frequency curve, using a logarithmic plot based upon data in the table, and the discharge-rating curve in (1).

4. Combining the results of (2) and (3), determine by the tabular method the average yearly direct flood damages for the Muskingum Basin (see page 149).

5. Referring to the data of flood-control reservoirs for the Muskingum River as given upon page 240 for which the total estimated project cost is about \$43,000,000, discuss the economic feasibility of this project, assuming that total flood damages will be double the direct damages.

Since the reservoir system will have a flood capacity of 1,326,400 acre-ft, or about 274 acre-ft per square mile at Coshocton, it may further be assumed that the flood-control project will practically eliminate flood losses.

Flood Damages—Muskingum River Basin

Flood of March, 1913:

Item	Direct Damage
Bridges and roads	\$ 2,856,000
Railroads and public utilities	2,718,000
Private property	7,914,700
Navigation works	112,000
Total	\$13,600,700

Flood of August, 1935:

The *direct* losses to agriculture, railroads, and highways were about \$6,000,000.

Muskingum River Near Coshocton—Discharge-rating Curve

Gauge Height	Discharge, Cfs
6.0	9,100
9.0	17,300
12.0	27,100
14.0	34,700

8. Flood Routing—Connecticut River.

Data:

1. Total inflow and outflow in thousand cfs for the Montague-Thompsonville reach of the Connecticut River for each half day of the period Mar. 12–25, 1936.

Problem:

1. Determine (*a*) inflow minus outflow (or valley storage) for each half-day period, and (*b*) cumulative valley storage.

2. Determine (*a*) change in flow (or incremental flow) for each half-day period with a weighting of inflow of 0.3, and (*b*) cumulative incremental flow.

3. Make a plot of cumulative incremental flow (ordinates) against cumulative valley storage (abscissas), using a scale of 1 in. = 40,000 cfs for ordinates and 1 in. = 40,000 acre-ft for abscissas.

4. Determine from the plot the value of K in the valley-storage formula.

5. Using this value of K, determine by tabular computation values of $d_2 = i_1 - (i_2 - d_1)(K - T/K + T)$ for each of the half-day periods from Mar. 12–25, 1936 (see page 75).

6. Plot hydrographs of (*a*) inflow at Montague, (*b*) actual outflow at Thompsonville and (*c*) computed outflow at Thompsonville, with scales of 1 in. = 40,000 cfs for ordinates and 1 in. = 2 days for abscissas.

9. Flood Frequency—Connecticut River.

Data:

On appended sheets (prepared from U.S. Geological Survey records):

1. Peak yearly discharges in cfs per square mile for Lower Connecticut River (drainage area about 8,000 sq miles), for the period of 37 years, 1900–1934, inclusive).

Flood Control—Connecticut River—Montague-Thompsonville Reach—Flood Discharge—March, 1936, Thousands Cfs

Date March, 1936	Total inflow	Total outflow	Date March, 1936	Total inflow	Total outflow
12	26.7 94.6	14.0 47.2	19	299 293	198 261
13	125 133	83.2 92.2	20	280 260	280 280
14	93 108	114 118	21	233 218	268 249
15	113 101	123 112	22	210 197	236 222
16	93.9 86.4	110 101	23	183 166	206 190
17	81.2 82.9	90.4 85.0	24	147 122	173 156
18	103 222	86.8 118	25	110 102	136 119

Flood Control—Connecticut River, 1900-1934, Yearly Peak Flows, Csm*

Year	Discharge	Year	Discharge	Year	Discharge
1900	8.0	1912	10.56	1924	7.61
1901	8.2	1913	13.50	1925	10.86
1902	9.9	1914	11.25	1926	10.46
1903	9.8	1915	9.65	1927	20.62
1904	8.82	1916	9.36	1928	9.34
1905	11.92	1917	7.92	1929	9.46
1906	9.65	1918	8.78	1930	6.24
1907	9.30	1919	9.76	1931	8.73
1908	8.10	1920	12.12	1932	11.30
1909	12.03	1921	9.56	1933	17.50
1910	10.72	1922	12.96	1934	14.52
1911	7.40	1923	10.62		

* Based upon records at gauging stations as follows: Orford, 1900–1903; Sunderland, 1904–1932; Montague City, 1932–1934.

2. For use in extending the period of record, yearly maximum flood stages at Hartford, Conn., for the period 1843–1934.

Problem:

1. From data supplied, prepare a plot showing, for the period 1900–1934, the relation between flood discharge at Sunderland in cfs per square mile (csm) and the stage at Hartford.

2. Using the stages at Hartford, as appended and the plot from (1), estimate the corresponding flood flows in csm at Sunderland for each Hartford stage prior to 1900.

3. Considering the period 1843–1934, inclusive, tabulate by years all flows at Sunderland in csm. Arrange these flows in order of magnitude, and compute for each its frequency of occurrence per 100 years based on a period of record of 92 years (1843–1934, inclusive).

Flood Stages, Hartford, Conn., 1843-1934

Year	Max. stage, ft	Year	Max. stage, ft	Year	Max. stage, ft
1843	27.2	1874	23.9	1905	24.0
1844	19.5	1875	18.7	1906	20.1
1845	19.0	1876	22.0	1907	20.3
1846	18.8	1877	22.9	1908	18.5
1847	21.2	1878	24.5	1909	24.7
1848	16.0	1879	21.5	1910	18.6
1849	17.5	1880	15.4	1911	15.5
1850	21.3	1881	16.5	1912	21.2
1851	14.5	1882	14.8	1913	26.3
1852	23.1	1883	20.5	1914	21.9
1853	20.5	1884	21.6	1915	20.6
1854	29.8	1885	18.0	1916	20.8
1855	21.5	1886	21.8	1917	18.3
1856	23.3	1887	22.5	1918	18.8
1857	19.5	1888	19.4	1919	19.8
1858	12.2	1889	15.6	1920	22.5
1859	26.4	1890	16.0	1921	19.9
1860	16.0	1891	19.8	1922	24.5
1861	21.5	1892	18.3	1923	22.0
1862	28.7	1893	24.0	1924	20.7
1863	22.2	1894	13.8	1925	20.5
1864	17.2	1895	25.7	1926	20.8
1865	24.8	1896	26.5	1927	29.0
1866	20.8	1897	20.8	1928	18.6
1867	20.0	1898	21.2	1929	18.9
1868	21.5	1899	22.0	1930	14.2
1869	26.7	1900	23.4	1931	17.9
1870	21.3	1901	26.4	1932	20.5
1871	18.7	1902	25.5	1933	26.0
1872	21.0	1903	23.3	1934	23.1
1873	21.2	1904	21.4		

4. Plot the results of (3) on logarithmic paper as a frequency curve, noting on the plot the year of occurrence for each point above 15 csm (Codex Graph Paper No. 4123 may be used).

5. Determine and tabulate from the plotted curve and its extension the flood to be expected in cfs per square mile for periods of 10, 25, 50, 100, 200, 300, and 500 years, respectively.

6. Determine from your results the probable frequency of the March, 1936, flood for which the peak flow was 30 csm.

10. Flood Losses—Connecticut River.

Data:

On appended sheet:

1. Flood losses (direct) in the Connecticut River Basin for floods of 1927 and 1936.
2. Areas of four states included in the Connecticut River Basin.
3. Flood peak flows at three stations in 1927 and 1936 floods.
4. Discharge at each station at which flood damage becomes negligible.

Problem:

1. Tabulate losses in 1927 and 1936 floods, determining and including for each item the loss in dollars per square mile.

2. Make a plot on cross-section paper of flood peak flow in cfs per square mile (horizontal) against direct flood damage for the entire basin in million dollars (vertical), showing by different curves these relations for Vernon, Montague, and Thompsonville.

3. Using the results from the Montague curve in (2) and flood-frequency periods as obtained in Problem 9, determine by the discharge-damage increment method the average yearly direct flood damage for the Connecticut Basin.

Data Sheet Accompanying Problem 10

1. Flood losses (direct), in millions of dollars, Connecticut River Basin.

State	Urban	Rural	Industrial	Railroad	Highway	Total
November, 1927, Flood						
Vermont	1.78	0.169	1.181	2.891	4.960	10.981
New Hampshire	0.115	0.076	0.110	0.130	1.336	1.767
Massachusetts	0.505	0.275	0.507	0.575	0.295	2.157
Connecticut	0.275	0.145	0.126	0.075	0.0	0.621
Total	2.675	0.665	1.924	3.671	6.591	15.526
March, 1936, Flood						
Vermont	0.102	0.203	0.286	0.441	0.733	1.765
New Hampshire	0.060	0.156	0.517	0.167	1.368	2.268
Massachusetts	4.609	0.915	7.725	0.950	4.946	19.144
Connecticut	3.131	0.376	7.324	0.128	0.785	11.745
Total	7.902	1.650	15.852	1.686	7.832	34.922

2. State areas, Connecticut River Basin.

State	Sq Miles
Vermont	3,950
New Hampshire	3,050
Massachusetts	2,680
Connecticut	1,420
Total	11,100

3. Flood peak flows, cfs per square mile.

Station	Drainage area, sq miles	1927 flood	1936 flood
Vernon	6,100	25.4	29.8
Montague	7,840	21.8	28.9
Thompsonville	9,640	20.6	29.3

4. Discharge in cfs per square mile at which flood damage is negligible:

Vernon	18
Montague	15
Thompsonville	15

11. Flood Control—Connecticut River.

Data:

The tentative plan of a 20-reservoir system for flood control on the Connecticut River includes the following:

Total drainage area controlled by reservoirs = 2,266 sq miles
Total storage capacity = 644,500 acre-ft (about 5.3 in)
Cost of reservoirs = $65,000,000

In addition to the above proposed reservoir system, certain portions of the drainage area are now or will be controlled by power-storage reservoirs. These include, of storage effective in the March, 1936, flood, a total of about 380,000 acre-ft; in the November, 1927, flood, about 340,000 acre-ft. Additional power storage is contemplated of about 200,000 acre-ft, all above the 15 Mile Falls (Comerford) plant, where the drainage area is 1,650 sq miles. Note also that the 40,000 acre-ft of storage was added at the Comerford plant when it was built in 1930.

Appended are daily flood flows for portions of the 1936 and 1927 flood hydrographs.

Problems:

1. Determine by the tabular method and the use of the flood hydrographs, as further noted, the maximum stages that would have been reached in the floods of 1936 and 1927 at Sunderland (or Montague) assuming the full contemplated storage, for both power and flood control, available at each of these times.

2. Plot on the discharge-damage curve as constructed in Problem 10 a curve showing these relations as modified by reservoir effect as determined in (1).

3. Make a revised table, using the revised damages in (2), and determine by the discharge-damage increment method the yearly flood damages assuming storage effective as in (1) and (2).

4. From the results of (3) and the determination of yearly average direct flood losses and allowances for indirect damages in Problem 10, estimate the yearly flood benefits in the Connecticut River Basin due to the expenditure of $65,000,000 for reservoirs.

5. What would be the yearly cost of the reservoir project? Compare this with yearly flood benefits, and discuss the economic advisability of the reservoir project.

Daily Flood Flows of Connecticut River, Cfs

At Montague, 1936		At Sunderland, 1927	
Date	Discharge, cfs	Date	Discharge, cfs
Mar. 11	11,000	Nov. 3	11,600
12	45,000		
13	90,000	4	100,000
14	103,000		
15	85,000	5	160,000
16	70,000		
17	69,000	6	144,000
18	158,000		
19	233,000	7	104,000
20	216,000		
21	187,000	8	71,000
22	169,000		
23	144,000	9	51,000
24	112,000		
25	89,000	10	41,000

12. **Average Flood Losses in the United States.** Referring to Table 8·6, it will be seen that, for the period of about 33 years from 1912 to 1945, observations of total flood losses for eight of these years have been approximately as follows in order of magnitude:

Year	Flood Losses, Million Dollars
1937	615
1936	578
1913	250
1927	250
1938	140
1945	100
1935	70
1921	50

Make a semilogarithmic plot of flood losses against percentage of time, using a linear scale as abscissas for the latter, assuming that the above data apply to the higher portion of the curve and that minimum yearly damage is $10,000,000.

Using the curve thus obtained, estimate the average yearly flood damage in the United States.

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